CHAPTER 2

SITE CHARACTERISTICS

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CHAPTER 2

SITE CHARACTERISTICS

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

Insert this subsection (Final Safety Analysis Report [FSAR] Subsection 2.0) following the introduction to Chapter 2 of the DCD.

2.0 SITE CHARACTERISTICS

HAR SUP 2.0-1 Chapter 2 describes the characteristics and site-related design parameters of the proposed Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR). 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:

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

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

- **HAR site**. The HAR site is an irregularly shaped area comprised of the following site components: the plant site (area within the fence line), Harris Reservoir perimeter, the dam at Harris Reservoir, the pipeline corridor, and the intake structure and pumphouse. The HAR site is located within Wake and Chatham counties.

- **Vicinity**. The vicinity is considered to be a band or belt, 9.7 kilometers (km) (6 miles [mi.]) wide, that surrounds the HAR site. The vicinity includes a much larger tract of land than the HAR site. The vicinity is located within four counties: Wake, Chatham, Harnett, and Lee.

- **Region**. The region applies to the area within an 80-km (50-mi.) radius from the center point of the HAR power block footprint, including the site and vicinity. The following counties are included entirely within the region: Chatham, Durham, Harnett, Lee, Orange, and Wake. The following counties are located partially within the region: Alamance, Caswell,

Cumberland, Franklin, Granville, Guilford, Hoke, Johnston, Montgomery, Moore, Nash, Person, Randolph, Richmond, Robeson, Sampson, Scotland, Vance, Wayne, and Wilson. The region includes the economic centers of Raleigh, Durham, Fayetteville, Cary, and Chapel Hill.

Table 2.0-201 provides a comparison of site-related design parameters for which the Westinghouse's AP-1000 Reactor (AP1000) plant is designed and site characteristics specific to the HAR 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 HAR. The fourth column denotes the section or table in the HAR 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.

HAR SUP 2.0-1

Table 2.0-201 (Sheet 1 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Air Temperature				
Maximum Safety[(a)(h)]	115°F dry bulb / 86.1°F coincident wet bulb	106.6°F dry-bulb/73.6°F coincident wet-bulb (100-year return estimate of 2-hour duration, 0% exceedance value)	FSAR 2.3.1.2.7	Yes
	86.1°F wet bulb (noncoincident)	83.5°F wet bulb (noncoincident) (Frequency of occurrence 0% for period of available record)	FSAR 2.3.1.2.7	Yes
Minimum Safety[(a)]	-40°F	-11°F (100-year return period)	FSAR 2.3.1.2.7	Yes
Maximum Normal[(b)]	101°F dry bulb / 80.1°F coincident wet bulb	94°F dry bulb / 75°F coincident wet bulb (Charlotte)	FSAR 2.3.1.2.7	Yes
	80.1°F wet bulb (noncoincident)[(c)]	77°F wet bulb (noncoincident)	FSAR 2.3.1.2.7	Yes
Minimum Normal[(b)]	-10°F	14°F dry bulb (Greensboro)	FSAR 2.3.1.2.7	Yes
Wind Speed				
Operating Basis	145 mph (3 second gust); importance factor 1.15 (safety), 1.0 (nonsafety); exposure C; topographic factor 1.0	97 mph (3 second gust) (Maximum sustained wind speed 100 mph; importance factor 1.15; exposure C; topographic factor 1.0)	FSAR 2.3.1.2.2	Yes
Tornado	300 mph Maximum pressure differential of 2.0 lb/in^2	300 mph Maximum pressure differential of 2.0 lb/in^2	FSAR 2.3.1.2.2	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 2 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR 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. (d) 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.	Peak ground acceleration: (defined as GMRS acceleration at 100 Hz): HAR 2: 0.123 g HAR 3: 0.137g Ground Response Spectra: At HAR 2: The horizontal GMRS is not bounded by the CSDRS (Figure 2.5.2-306). However, the nuclear island FIRS is bounded by the CSDRS (Figure 3.7-201). At HAR 3: The horizontal GMRS is not bounded by the CSDRS (Figure 2.5.2-307). The nuclear island FIRS is bounded except between 33 and 35 Hz (Figure 3.7-202). The nuclear island FIRS exceeds the Westinghouse CSDRS in the frequency range of 33-35 Hz by a maximum value of 3 percent. This FIRS was further used to develop in-structure spectra that are enveloped by AP1000 design in-structure spectra.	FSAR 2.5.2.6, 3.7.1.1.1	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 3 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
	The hard rock high frequency (HRHF) envelope response spectra are shown in Figure 5.0-3 and Figure 5.0-4 defined at the foundation level for 5% damping. The HRHF envelope response spectra provide an alternative set of spectra for evaluation of site-specific GMRS. A site is acceptable if its site-specific GMRS 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 fps, while maintaining a shear wave velocity equal to or above 8,000 fps at the lower depths.			
Fault Displacement Potential	No potential fault displacement considered beneath the Seismic Category I and Seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the Seismic Category I and Seismic Category II structures.	The potential for tectonic deformation at the HAR site is judged to be negligible.	FSAR 2.5.3.8	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 4 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
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 8900 lb/ft^2 over the footprint of the nuclear island at its excavation depth.	HAR 2: 54,000 lb/in^2 HAR 3: 29,000 lb/in^2 These results are based on highly conservative estimates of rock mass strength, as described in FSAR Subsection 2.5.4.10.1.	FSAR 2.5.4.10.1.3	Yes
Dynamic Bearing Capacity for Normal Plus Safe Shutdown Earthquake (SSE)	The allowable bearing capacity, including a factor of safety appropriate for the design load combination, shall be greater than or equal to the maximum bearing demand of 35,000 lb/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.	HAR 2: 68,000 lb/in^2 HAR 3: 37,000 lb/in^2 These results are based on highly conservative estimates of rock mass strength, as described in FSAR Subsection 2.5.4.10.1.	FSAR 2.5.4.10.1.3	Yes
Shear Wave Velocity	Greater than or equal to 1000 ft/sec based on minimum low-strain soil properties over the footprint of the nuclear island at its excavation depth	Rock below nuclear island subgrades has V_s generally greater than 3500 ft/sec that increases to above 5000 ft/sec within 30 feet below the nuclear island foundation. Thin, isolated soil seams and weathered intervals have V_s less than 3500 ft/sec but greater than 3000 ft/sec. Inclusion of the isolated soil seams/weathered intervals in the site response was found to produce slightly larger motions at some frequencies. Therefore, they were included in the site response analyses used to develop the site ground motions.	FSAR 2.5.4.4.2	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 5 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Lateral Variability	Soils supporting the nuclear island should not have extreme variations in the 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 thicknesses or properties of the geologic units, or 2. Site-specific assessment of subsurface conditions demonstrates that the bearing pressures below the footprint of the nuclear island do not exceed 120% of those from the generic analyses of the nuclear island at a uniform site, or 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 firm rock. Significant variations in subgrade stiffness are not present.	FSAR 2.5.4.10.3	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 6 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Case 1: For a layer with a low strain shear wave velocity greater than or equal to 2500 ft/sec, 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.	The nuclear islands for HAR 2 and HAR 3 will be founded on firm rock with V_s predominantly greater than 3500 ft/sec. No V_s was measured to be 2500 ft/sec or less below the nuclear islands. HAR 2: Dip is less than 20 degrees, and V_s varies less than 20 percent from the average V_s in each layer below the nuclear island. HAR 3: Dip is approximately 20 degrees. V_s correlations indicate dip of 19.9 degrees and evaluation of acoustic logging surveys indicate localized dip measurements as high as 23.5 degrees. Dip along the north-south and east-west axes of the nuclear island are less than 20 degrees. One fractured rock interval includes material with V_s more than 20 percent below the average for the layer. This variation was encountered at Borehole BPA-25, at approximate elevation 194 to 196 ft. amsl. The V_s of this interval (3780 to 3870 ft/sec) is approximately 27 percent below the average for this layer among the three boreholes. This interval was included in the site response model as it led to slightly larger motions at some frequencies. The isolated intervals with lower V_s below nuclear island subgrade have been conservatively considered in preparation of the HAR 3 GMRS.	FSAR 2.5.4.1.1.2, 2.5.4.4.2 FSAR 2.5.2.6	HAR 2: Yes HAR 3: No, not clearly bounded; however, not considered significant for reasons stated at left.

HAR SUP 2.0-1

**Table 2.0-201 (Sheet 7 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics**

	AP 1000 DCD Site Parameters		HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Liquefaction Potential	No liquefaction considered beneath the Seismic Category I and Seismic Category II structures and immediate surrounding area. The immediate surrounding area includes the effective soil supporting media associated with the Seismic Category I and Seismic Category II structures.		None at the site-specific SSE. Foundations of Seismic Category I structures are on rock.	FSAR 2.5.4.8	Yes
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 present above sound rock will not be left in place under safety-related structures. Foundations of Seismic Category I structures are on rock. A waterproofing membrane will be located between the upper and lower layers of the mudmat, meeting AP1000 DCD requirements of ≥0.55 static coefficient of friction.	N/A	N/A
Limits Of Acceptable Settlement Without Additional Evaluation[i]	Differential Across Nuclear Island Foundation Mat	1/2 inch in 50 ft.	<1/4 inch in 50 ft. (projected)	FSAR 2.5.4.10.3	Yes (projected)
	Total for Nuclear Island Foundation Mat	6 inches	< 1 inch (projected)		
	Differential Between Nuclear Island and Turbine Building[j]	3 inches	< 3.0 inches (projected)		
	Differential Between Nuclear Island and Other Buildings[j]	3 inches	< 3.0 inches (projected)		

HAR SUP 2.0-1

**Table 2.0-201 (Sheet 8 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics**

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Missiles				
Tornado	4000 lb. automobile at 105 mph horizontal, 74 mph vertical 275 lb., 8 in. shell at 105 mph horizontal, 74 mph vertical 1 inch diameter steel ball at 105 mph in the most damaging direction	4000 lb. automobile at 105 mph horizontal, 74 mph vertical 275 lb., 8 in. shell at 105 mph horizontal, 74 mph vertical 1 inch diameter steel ball at 105 mph in the most damaging direction	DCD 3.5.1.4 DCD Section 3.5 APP-GW-GLR-020, “Wind and Tornado Site Interface Criteria,” Westinghouse Electric Company LLC.[e]	Yes
Flood Level	Less than plant elevation 100 ft.	DCD plant elevation of 100 ft. = 261 ft. NGVD29.	FSAR 2.4.1.1	
		The maximum water elevation obtained by the combined effect of the PMF, wave runup, and wind setup is 259.39 ft.	FSAR 2.4.3.6.8	Yes

HAR SUP 2.0-1

Table 2.0-201 (Sheet 9 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Ground Water Level	Less than plant elevation 98 ft.	DCD groundwater elevation of 98 ft. = 259 ft. NGVD29. Surficial aquifer monitoring wells MWA-3S (HAR 2) and MWA-8S (HAR 3) recorded groundwater elevations (August 2006, November 2006, February 2007, and May 2007), which ranged from 78.21 to 79.31 m (256.60 to 260.19 ft.) NGVD 29 and 77.86 to 78.36 m (255.45 to 257.08 ft.) NGVD 29, respectively. Site-specific groundwater elevations did exceed an elevation of 78.9 m (259 ft.) NGVD 29 during the February 2007 gauging event in monitoring well MWA-3S, which is located at the proposed location of HAR 2. The current average groundwater elevation within MWA-3S is 78.63 m (257.99 ft.) NGVD29. Grading of the plant site, including a series of stormwater drainage ditches, will result in a hydrologic alteration including the permanent change in groundwater elevations to less than 78.9 m (259 ft.) NGVD 29.	FSAR 2.4.12.5	Yes
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 elevation for the HAR site is 79.6 m (261 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. = 261 ft. NGVD29.	FSAR 2.4.1.1	Yes

HAR SUP 2.0-1

**Table 2.0-201 (Sheet 10 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics**

	AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No
Precipitation				
Rain	20.7 in/hr [1-hr 1-mi^2 PMP]	18.9 in/hr	FSAR 2.4.2.3	Yes
Snow / Ice	75 pounds per square foot on ground with exposure factor of 1.0 and important factor of 1.2 (safety) and 1.0 (non-safety)	17 lb/ft^2	FSAR 2.3.1.2.3	Yes
Atmospheric Dispersion Values X/Q[(f)]				
Site Boundary (0-2 hr)	≤ 5.1×10^{-4} sec/m^3	4.7×10^{-4} sec/m^3	Table 2.3.4-201	Yes
Site Boundary (annual average)	≤ 2.0×10^{-5} sec/m^3	6.0×10^{-6} sec/m^3	Table 2.3.5-201	Yes
Low population zone boundary				
0-8 hr	≤ 2.2×10^{-4} sec/m^3	9.1×10^{-5} sec/m^3	Table 2.3.4-201	Yes
8-24 hr	≤ 1.6×10^{-4} sec/m^3	6.3×10^{-5} sec/m^3	Table 2.3.4-201	Yes
24-96 hr	≤ 1.0×10^{-4} sec/m^3	2.8×10^{-5} sec/m^3	Table 2.3.4-201	Yes
96-720 hr	≤ 8.0×10^{-5} sec/m^3	9.0×10^{-6} sec/m^3	Table 2.3.4-201	Yes
Population Distribution				
Exclusion area (site)[(g)]	0.5 mi.	The minimum distance from the effluent release boundary to the exclusion area boundary is 1245 m (4085 ft. or 0.77 mi.) in the east, west, and northerly sectors and 1600 m (5249 ft. or 0.99 mi.) in the southerly sectors	FSAR 2.1.1.2	Yes

HAR SUP 2.0-1

**Table 2.0-201 (Sheet 11 of 11)
Comparison of AP1000 DCD Site Parameters
and Shearon Harris Nuclear Power Plant Units 2 and 3 Site Characteristics**

AP 1000 DCD Site Parameters	HAR Site Characteristics	HAR Site Characteristic Reference	Bounding Yes/No

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 noncoincident 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. Seismic input is defined at finished grade except for sites where the nuclear island is founded on hard rock.
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 specified for the site boundary applies whenever a discussion refers to the exclusion area boundary.
g) Exclusion area (site) for HAR is defined as two overlapping areas centered on the reactor building of each unit. The areas are defined by a circular distance of 1245 m (4085 ft.) in the east, west, and northerly sectors and 1600 m (5249 ft.) in the southerly sectors. The overall shape of the HAR EAB is defined by the outermost boundary of each unit’s defined area.
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.

°C = degrees Celsius
°F = degrees Fahrenheit
Amsl = above mean sea level
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
in/hr = inches per hour
lb. = pound
lb/ft^2 = pound per square foot
lb/in^2 = pound per square inch
m = meter
mi. = mile
mph = miles per hour
N/A = not applicable
NGVD29 = National Geodetic Vertical Datum of 1929
PMF = probable maximum flood
PMP = probable maximum precipitation
sec/m^3 = seconds per cubic meter
SSE = safe shutdown earthquake
V_S = shear wave velocity

HAR SUP 2.0-1

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

X/Q (sec/m³) at HVAC Intake for the Identified Release Points[(a)]

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	Ground Level Containment Release Points[(c)]	Ground Level Containment Release Points	PORV and Safety Valve Releases[(d)]	PORV and Safety Valve Releases	Condenser Air Removal Stack[(g)]	Condenser Air Removal Stack	Steam Line Break Releases	Steam Vent	Fuel Handling Area[(e)]	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	**HAR**	**HAR**	**DCD**	**HAR 2 and HAR 3**	**DCD**	**HAR**	**DCD**	**HAR 2 and HAR 3**	**DCD**	**HAR**	**DCD**	**HAR**	**HAR**
0 - 2 hours	**3.0E-03**	2.0E-03	1.6E-03	**6.0E-03**	4.2E-03	**2.0E-02**	1.2E-02	**6.0E-03**	1.5E-03	**2.4E-02**	1.4E-02	**6.0E-03**	1.4E-03	1.1E-03
2 - 8 hours	**2.5E-03**	1.4E-03	1.1E-03	**3.6E-03**	3.2E-03	**1.8E-02**	1.0E-02	**4.0E-03**	1.2E-03	**2.0E-02**	1.1E-02	**4.0E-03**	1.0E-03	8.3E-04
8 - 24 hours	**1.0E-03**	6.1E-04	4.9E-04	**1.4E-03**	1.2E-03	**7.0E-03**	4.2E-03	**2.0E-03**	5.1E-04	**7.5E-03**	4.6E-03	**2.0E-03**	4.5E-04	3.7E-04
1 - 4 days	**8.0E-04**	4.5E-04	3.4E-04	**1.8E-03**	1.2E-03	**5.0E-03**	3.1E-03	**1.5E-03**	3.4E-04	**5.5E-03**	3.4E-03	**1.5E-03**	3.2E-04	2.6E-04
4 - 30 days	**6.0E-04**	3.5E-04	3.0E-04	**1.5E-03**	1.0E-03	**4.5E-03**	2.6E-03	**1.0E-03**	2.6E-04	**5.0E-03**	2.8E-03	**1.0E-03**	2.5E-04	2.1E-04

X/Q (sec/m³) at Control Room Door for the Identified Release Points[(f)]

	Plant Vent or PCS Air Diffuser[(b)]	Plant Vent	PCS Air Diffuser	Ground Level Containment Release Points[(c)]	Ground Level Containment Release Points	PORV and Safety Valve Releases[(d)]	PORV and Safety Valve Releases	Condenser Air Removal Stack[(g)]	Condenser Air Removal Stack	Steam Line Break Releases	Steam Vent	Fuel Handling Area[(e)]	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	**HAR**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**HAR**
0 - 2 hours	**1.0E-03**	4.2E-04	4.3E-04	**1.0E-03**	3.6E-04	**4.0E-03**	8.6E-04	**2.0E-02**	3.4E-03	**4.0E-03**	8.4E-04	**6.0E-03**	3.4E-04	3.3E-04
2 - 8 hours	**7.5E-04**	3.2E-04	3.1E-04	**7.5E-04**	2.9E-04	**3.2E-03**	6.2E-04	**1.8E-02**	2.6E-03	**3.2E-03**	5.9E-04	**4.0E-03**	2.5E-04	2.4E-04
8 - 24 hours	**3.5E-04**	1.4E-04	1.4E-04	**3.5E-04**	1.4E-04	**1.2E-03**	2.6E-04	**7.0E-03**	1.2E-03	**1.2E-03**	2.5E-04	**2.0E-03**	1.1E-04	1.1E-04
1 - 4 days	**2.8E-04**	1.1E-04	1.0E-04	**2.8E-04**	1.0E-04	**1.0E-03**	2.0E-04	**5.0E-03**	7.1E-04	**1.0E-03**	1.9E-04	**1.5E-03**	8.2E-05	8.0E-05
4 - 30 days	**2.5E-04**	8.3E-05	8.4E-05	**2.5E-04**	8.9E-05	**8.0E-04**	1.6E-04	**4.5E-03**	6.0E-04	**8.0E-04**	1.5E-04	**1.0E-03**	6.7E-05	6.5E-05

HAR SUP 2.0-1

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

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

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

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

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

X/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

HAR COL 2.1-1

2.1.1.1 Specification of Location

The Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR) site is located in the southwest corner of Wake County, North Carolina, adjacent to the existing Shearon Harris Nuclear Power Plant Unit 1 (HNP). Portions of the site also lie in southeastern Chatham County (Figure 2.1.1-201). The City of Raleigh, North Carolina, is approximately 34.9 kilometers (km) (21.7 miles [mi.]) northeast of the site, and the City of Sanford, North Carolina, is approximately 26.5 km (16.5 mi.) southwest of the site. The Cape Fear River flows in a northwest-to-southeast direction approximately 11.3 km (7 mi.) south of the site.

The site is located just north and west of the HNP, on a peninsula that extends into Harris Reservoir from the northwest. As depicted in the Carolina Power and Light (CP&L) “Shearon Harris Nuclear Power Plant Safety Analysis Report, 1983, Amendment 48,” the Tom Jack Creek arm of the reservoir lies to the west; the Thomas Creek arm of the reservoir lies to the east. The reactor buildings and generating facilities lie within a nuclear exclusion area, access to which is controlled.

The location of each reactor is described in Table 2.1.1-201. The proposed Shearon Harris Nuclear Power Plant Unit 2 (HAR 2) is the southernmost reactor and the proposed Shearon Harris Nuclear Power Plant Unit 3 (HAR 3) is the northernmost reactor. The site is shown on the New Hill U.S. Geological Survey (USGS) quadrangle map 7.5-minute series (Reference 2.1-201). This and surrounding USGS quadrangle maps are listed in Table 2.1.1-202.

2.1.1.2 Site Area Map

The plant location is shown on Figure 2.1.1-202, Exclusion Area Boundary (EAB) and Site Boundary, and Figure 2.1.1-203, HAR Exclusion Boundary Plan. These maps 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 (other than those related to plant operation) are located on-site.

The HAR EAB, as shown on Figure 2.1.1-203, is defined as two overlapping areas centered on the reactor building of each unit. The areas are defined by a circular distance of 1600 m (5249 ft.) in the southerly sectors (east-southeast through west-southwest) and 1245 m (4085 ft.) in the east, west, and northerly sectors (west through east). The overall shape of the HAR EAB is defined by the outermost boundary of each unit's area as shown on Figure 2.1.1-203. The HAR site is located within a much larger tract of land that includes the HNP EAB, Harris Reservoir, and some surrounding lands. The HAR EAB is within the boundary of the HNP EAB. The HAR and HNP site includes the 74-m (243-ft.) contour of the Main Reservoir and the 79-m (260-ft.) contour of the Auxiliary Reservoir. The site boundary also defines the plant property line. The site totals 4348 ha (10,744 ac.). According to the NUREG-1038, this acreage is sometimes reported as "approximately" 4371 ha (10,800 ac.). For consistency, the acreage is reported as the larger number.

Of the 10,800 or so acres that comprise the HNP site, approximately 14.4 square kilometers (km^2) (5.6 square miles [mi^2]) were inundated over the 1980 to 1983 period with the creation of Harris Reservoir. A second, smaller impoundment, the Auxiliary Reservoir (also known as [aka] "West Auxiliary" Reservoir), was created when the Tom Jack Creek arm of Harris Reservoir was dammed. This 1.3 km^2 (0.5 $mi.^2$) reservoir, which lies immediately west of the generating facilities, was created to serve as a source of water for the emergency service water system for HNP.

The Harris Lake County Park operated by Wake County is located along County Park Drive approximately 3.2 km (2 mi.) east of the plant location. This park includes fishing facilities, primitive group camping, hiking and mountain biking trails, picnicking facilities, canoe and kayak launch areas, and a disc golf course. Park rules do not allow watercraft with trailers (Reference 2.1-202). There are two public boat ramps on Harris Lake. These boat ramps are located off Bartley Holleman Road in Wake County, and Crosspoint Road near Highway 42 in Chatham County (Reference 2.1-203). Other nearby recreation facilities include Jordan Lake, which is located 9 to 19 km (5 to 12 mi.) to the northwest and the North Carolina Wildlife Resources Commission (NCWRC) Game Lands, which are adjacent to Harris Lake. The recreation areas surrounding the site are shown on Figure 2.1.1-204.

The proposed project affects water level elevations in the Main Reservoir. This results in an elevation increase from 67.1 m (220 ft.) mean sea level (msl) to 73.2 m (240 ft.) msl. The 6-m (20-ft.) increased water level in the Main Reservoir affects the surrounding area, with possible impacts to the Harris Lake County Park, NCWRC Game Lands, public boat ramps, and local roads. A detailed evaluation of lake level impacts is provided in the Environmental Report.

Transportation routes to and from the plant include U.S. Highway 1, which passes north of the site, and several State-maintained roads that traverse the area. The CSX Railroad passes north of the plant, and the Norfolk Southern

Railroad crosses south of the Main Dam. Railway access to the plant is provided by a Progress Energy rail spur that connects to the CSX Railroad.

The Cape Fear River is 11.3 km (7 mi.) to the south of the site.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

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

The “Standards for Protection Against Radiation” 10 CFR 20 and Appendix I to 10 CFR 50 describe effluent release limits in order 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; 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 HAR 2 and HAR 3, and Table 2.1.1-203 shows the distance from the centerpoint of the HAR site to the HAR EAB for each major compass direction. Because of the shape of the overlapping EABs, the distance from the centerpoint of the HAR site to the outermost boundary of the HAR EAB ranges from approximately 1247 to 1749 m (4090 to 5738 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 EAB and low population zone (LPZ). The exclusion area boundary 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 exclusion area are owned by Progress Energy. Roads that existed within the exclusion area prior to the construction of the original plant have been abandoned by the State and are not open for public access or use. However, Shearon Harris Road (SR1134) is open in order to allow traffic to connect with access roads to the plant. Easements are in place to allow the State of North Carolina to maintain the road and to allow the local telephone provider to maintain communication lines to the plant.

A rail spur connects the plant with nearby commercial rail service. Rail carriers are allowed access to the plant over the spur rail line.

No mineral rights have been leased within the exclusion area, and there are no surface or subsurface rights for mineral mining associated with the plant location. According to the CP&L "Shearon Harris Nuclear Power Plant Safety Analysis Report," there is little potential for commercial exploitation of minerals within the exclusion area and leasing of mineral rights is not anticipated.

2.1.2.2 Control of Activities Unrelated to Plant Operation

The following activities are unrelated to plant operations but will be permitted within the exclusion area (aside from transit through the area):

- State Road Highway and Utility Maintenance: Activities along the State Road are limited to highway/utility maintenance. These activities may occur along the easement granted to the State. Emergency procedures for evacuating the exclusion area will include evacuation of road/utility maintenance personnel in accordance with Security Procedures. Signage is posted along the road at the exclusionary boundary. This signage states that the area is private property and advises persons therein that they are subject to evacuation. Based on the CP&L "Shearon Harris Nuclear Power Plant Safety Analysis Report," total estimated evacuation time for roadway maintenance personnel is 30 minutes.

- Railroad Activity: Use of the railway spur is limited to that which is related to delivery of railcars to the plant. Progress Energy owns the rail spur and is responsible for maintenance. According to the "Shearon Harris Nuclear Power Plant Safety Analysis Report," estimated evacuation time for railroad personnel in the exclusion area is 15 minutes.

- Telephone Service Repair: The activities of the local telephone service provider may be limited to activities that construct, modify, repair, and maintain telephone lines to the plant. Signage is posted in areas where the easement intersects the EAB. These signs state that the area is private property and advises persons therein that they are subject to evacuation. Based on the CP&L "Shearon Harris Nuclear Power Plant Safety Analysis Report," it is estimated that about 10 people would be involved in telephone maintenance/installation operations in the exclusion area at any one time and that they could be evacuated within 30 minutes.

- Recreation Use: The general public is allowed to participate in recreation activities within a portion of the exclusion area. Activities within the area are limited by signage that is posted at known points of entry on the EAB and buoyed in conspicuous locations within and on the boundary of the reservoir waters inside the exclusion area. Based on the "Shearon Harris Nuclear Power Plant Safety Analysis Report," it is anticipated that

persons within this area would be able to clear the exclusion area within 1 hour of notification.

- Firefighting and Firearms Training: The activities are limited to those associated with the HNP Nuclear Security Firearms Training Facility, the Cary Police Department Firing Range, and the Wake County Fire Department Training Facility. These areas are located within the Owner Controlled Exclusion Area. Based on the “Shearon Harris Nuclear Power Plant Safety Analysis Report,” estimated capacity of these facilities is approximately 75 people combined. Estimated evacuation time is 30 minutes.

2.1.2.3 Arrangement for Traffic Control

Based on the CP&L “Shearon Harris Nuclear Power Plant Safety Analysis Report,” the following measures will be implemented if it becomes necessary to control traffic into the exclusion area:

- Access control will be established by Plant Security/Local Law Enforcement personnel on Shearon Harris Road (SR1134) where it intersects with the EAB in order to limit access to the area to authorized personnel.

- In a similar manner, the rail spur will be closed to rail traffic. Scheduled rail deliveries may be cancelled or postponed.

- Telephone service activities along the easement in the exclusion area will be prohibited or postponed. If necessary, warning signs will be posted at the intersection of the easement and the EAB.

- Control of public access to recreational land and water areas will be exercised by motorized patrols at known or likely points of entry on land and by patrol boats on the reservoirs. Plant Security/Local Law Enforcement will provide traffic control. Assistance will be requested from the county sheriffs’ departments and the North Carolina Highway Patrol.

Additional information regarding evacuation management is provided in the 2009 publication of “Harris Nuclear Plant Development of Evacuation Time Estimates” (Reference 2.1-204), or updated information may be found in later revisions.

2.1.2.4 Abandonment or Relocation of Roads

The EABs for the proposed HAR 2 and HAR 3 lie within the EAB for HNP. Therefore, the EABs for the proposed HAR 2 and HAR 3 should not affect any additional roads or bridges. The impact on roads and bridges near the Main Reservoir due to the proposed increase in water level elevation will be discussed further in the Environmental Report.

2.1.3 POPULATION DISTRIBUTION

2.1.3.1 Population within 10 Miles

Based on the 2000 U.S. Census (Reference 2.1-205), the total residential population within 16 km (10 mi.) of the HAR site is 55,219 persons (as shown in Table 2.1.3-201). Figure 2.1.3-201 shows the significant population groupings (for example, cities and towns) within 16 km (10 mi.) of the site. The 16 km (10 mi.) population of the HAR site varies from the population reported in the Emergency Plan, because 10-mi. emergency planning zones and population radius methodologies differ slightly. Figure 2.1.3-201 also shows a sector chart divided into radii for 0 to 10 mi. The sector chart was used in determining population distribution as described in the following sections. The current proposed plan includes the installation of two AP1000 units. It was assumed that the center of the distance between the two reactor buildings would be used as the centerpoint for the radii and sector grid. The radii were expanded by half of the distance between the two reactor buildings for HAR 2 and HAR 3. The two proposed reactor buildings are centered at the following coordinates:

HAR 2 Latitude: 35° 38’ 15.39” Longitude: -78° 57’ 29.84”

HAR 3 Latitude: 35° 38’ 23.90” Longitude: -78° 57’ 34.71”

The distance between the centerpoint of the reactor buildings for HAR 2 and HAR 3 is 289.5 m (950 ft. or 0.2 mi.). 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 1.8 km (1.1 mi.) to provide adequate coverage of HAR 2 and HAR 3 while maintaining compliance with regulatory guidelines.

Residential and transient population distribution within the sectors, shown on Figure 2.1.3-204, has been summarized and provided in Table 2.1.3-201. The table indicates that a majority of the population lives in the northeast and east-northeast portion of the sectors, 8 to 16 km (5 to 10 mi.) from the site. The eastern sectors include the cities and towns of Apex (population of 20,212) located 13.9 km (8.6 mi.) northeast, Holly Springs (population of 9192) located 10.9 km (6.8 mi.) east, and Fuquay-Varina (population of 7898) located 15.7 km (9.8 mi.) east. Data from the 2000 U.S. Census (Reference 2.1-205) 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 33 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 were defined as follows:

- Seasonal Population: A GIS was used to collect information on seasonal and vacation home usage within the 16-km (10-mi.) radius.

- Transient Business Population: For businesses located within the 16-km (10-mi.) radius, it was assumed that the employees would be included in the transient population estimates for major employers. A list of the major employers and the total number of employees was obtained from the Economic Development office (Reference 2.1-206, Reference 2.1-207, Reference 2.1-208, Reference 2.1-209). Major employers were defined as those employers with greater than 100 employees.

- Hotel/Motel Population: Hotels and motels located within the 16-km (10-mi.) radius were identified using a GIS. The GIS data were sorted based on distance from the centerpoint of the two proposed reactor units (HAR 2 and HAR 3). Total room numbers were obtained by phone survey or by the hotels' websites. It was assumed that one person occupied each room on a given night (Reference 2.1-210, Reference 2.1-211, Reference 2.1-212).

- Recreation Areas: Figure 2.1.1-204 shows principal recreation areas within 16 km (10 mi.). Three major recreational areas were identified within the 16-km (10-mi.) radius of the site: the Jordan Lake State Park, the Wake County – Harris Lake County Park, and the NCWRC Game Lands (Reference 2.1-202, Reference 2.1-213). The NCWRC Game Lands include the Harris Game Lands (approximately 5 km [3 mi.] south-southeast) and the Chatham Game Lands (approximately 5 to 6 km [3 to 4 mi.] south-southwest). The NCWRC Game Lands do not employ measures for determining daily usage. Therefore, the NCWRC Game Lands were not included in the determination of transient population estimates.

- Special Populations (Schools, [Reference 2.1-214, Reference 2.1-215, Reference 2.1-216], Hospitals, Nursing Homes, and Correctional Facilities): A GIS was used to determine schools, hospitals, nursing homes, and correctional facilities located within the 16-km (10-mi.) radius.

- Festivals: There are no major festivals within the 16-km (10-mi.) radius that would affect the transient population estimates. The annual Progress Energy Lineman's Rodeo is held on Progress Energy's property and is attended by approximately 1300 people; however, this 1-day event is not included in transient population estimates because of its short duration.

- Migrant Workers: Migrant workers were calculated using average statewide statistical information supplied by the U.S. Department of Agriculture's (USDA's) 2002 Agricultural Census (Reference 2.1-217) related to the number of migrant workers.

Population projections for 10-year increments up to 80 years from the latest U.S. Census are included in Table 2.1.3-202. Projection information was collected from the North Carolina State Demographics unit web site for county projections. 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 2.1-218). The percent change was estimated for each county. It was assumed that the expected population change rate for the 10-year increments between 2030 and 2080 would be similar to the average estimated percent change 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 (Reference 2.1-205). Population projections for each sector were then 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 and 80 km (10 and 50 mi.) from the HAR site is 1,973,427 persons (as shown in Table 2.1.3-203) (Reference 2.1-205). Figure 2.1.3-202 shows the significant population groupings (for example, cities and towns) within the region (80 km or 50 mi.).

Residential and transient population distribution within the sectors for the 16- to 80-km (10- to 50-mi.) radii are shown on Figure 2.1.3-206 and have been summarized and provided in Table 2.1.3-203. The sector chart was used in determining population distribution as described below. Table 2.1.3-203 indicates that a majority of the residential population is concentrated in the north, northeast, and east-northeast sectors. There is also a significant portion of the resident population in the south sector. The U.S. Census Bureau data from the 2000 U.S. Census (Reference 2.1-205) and a GIS were used to determine the sector population distribution, as described in Subsection 2.1.3.1.

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 in Subsection 2.1.3.1 was used to determine seasonal population within the 16- to 80-km (10- to 50-mi.) radius.

- Transient Business Population: For businesses located within the 80-km (50-mi.) radius, it was assumed that there is no net change 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: A GIS was used to collect information on the location and number of hotels and motels within the 80-km (50-mi.)

radius. Because of the large area and based on reasonable judgment, it was assumed that the average hotels and motels contained 75 rooms and 25 rooms, respectively. For the purposes of determining transient population estimates, it was assumed that one person occupied each room on a given night.

- Recreation Areas: Recreation areas were defined as public recreation areas where usage patterns are tracked based on parking permits or other entrance fees. Figure 2.1.3-203 shows principal recreation areas within 16 to 80 km (10 to 50 mi.). Phone interviews were conducted to identify peak usage by visitors. (Reference 2.1-213)

- Special Populations (Schools, Hospitals, Nursing Homes, and Correctional Facilities): Because of the large area and using reasonable judgment, it was assumed that there would be no net change in special populations within the 80-km (50-mi.) radius. The U.S. Census Bureau includes 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. It was assumed that staff and residents temporarily placed in hospitals, nursing homes, and other institutions are likely to live within the 80-km (50-mi.) radial area, and therefore would not contribute to transient population estimates.

- Migrant Workers: The methodology described in Subsection 2.1.3.1 was used to determine migrant worker population within the 80-km (50-mi.) radius.

Population projections for 10-year increments up to 80 years from the latest U.S. Census are included in Table 2.1.3-204. 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. Population projections were obtained from the North Carolina State Demographics unit Website (Reference 2.1-218). The same methodology as described in 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

The transient population is important to quantify in order to determine the number of people in the vicinity of the site who would not normally be included in census counts. The transient population may include those using recreational facilities, seasonal residents, special populations (for example, schools, hospitals, nursing homes, and correctional facilities), and business and migrant workers who do not normally live in the area. The assumptions used to estimate transient populations are described in Subsection 2.1.3.1 and Subsection 2.1.3.2. As noted in these subsections, significant variations due to transient land use are not expected. Further, the significant transient population in the region is a result of recreational

travel to State parks in the area. Table 2.1.3-201 and Table 2.1.3-203 include the transient population for the year 2000 and Table 2.1.3-202 and Table 2.1.3-204 include the year 2010 and 10-year incremental population projections to 2080 of the transient population surrounding the HAR.

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 proposed HAR 2 and HAR 3. 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. Population distribution was determined based on a single radius of 5 km (3.1 mi.) to include the distance between HAR 2 and HAR 3.

The LPZ was selected to provide reasonable probability that appropriate protective measures could be taken on behalf of the permanent and transient residents. 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. The determination of the LPZ is further explained in FSAR Section 2.3.

Facilities and institutions were identified with the LPZ (Reference 2.1-206, Reference 2.1-207, Reference 2.1-208, and Reference 2.1-209). The Harris Lake County Park is located approximately 3.2 km (2 mi.) southeast of the site. The Harris Energy and Environmental Center is located approximately 3.4 km (2.1 mi.) east northeast of the site. Two private nursing homes were identified between 3.2 and 4.8 km (2 and 3 mi.) to the northeast and 3.2 and 4.8 km (2 to 3 mi.) to the north northeast. No other facilities or institutions such as schools, hospitals, or prisons are within the LPZ. Nearby industrial, transportation, and military facilities are further described in FSAR Section 2.2.

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.

The average daily recreational users at recreational facilities in the vicinity of the HAR site, including Harris Lake County Park, are shown in Table 2.1.3-205.

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 Cary, North Carolina, which is located 21 km (13 mi.) northeast of the site. The land use between the site and Cary is primarily rural with some scattered residential land use. In 2000, its population was 94,536 (Reference 2.1-219). 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 plant is projected to change significantly through 2080.

2.1.3.6 Population Density

The current and projected residential population densities in the vicinity and in the regional area surrounding the HAR site are presented on Figures 2.1.3-204, 2.1.3-205, 2.1.3-206, and 2.1.3-207. Most of the area within a 16-km (10-mi.) radius of the site is rural, with a population density in 2000 of 173 people per square mile (ppsm). The area between 16 and 80 km (10 and 50 mi.) of the site is the most densely populated. In 2000, the 0 to 80 km (0 to 50 mi.) residential and transient population was 2,097,567 persons, with an average population density of 263 ppsm (as shown in Tables 2.1.3-201, 2.1.3-203, and 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 population distribution and population density for the year 2010 and 2015 are shown in Table 2.1.3-207 and on Figures 2.1.3-208, 2.1.3-209, 2.1.3-210, and 2.1.3-211.

U.S. Nuclear Regulatory Commission (NRC) Standard Review Plan (NUREG-0800), Section 2.1.3, III, 5, notes that if the population density of the proposed site exceeds but is not well in excess of 500 ppsm over a radial distance out to 32 km (20 mi.), then the analysis of alternative sites should evaluate alternative sites having lower population density. However, consideration could be given to other elements such as safety, environmental, or economic factors that may result in the site with the higher population density being found acceptable. The population density projected for the HAR site at the time of initial site approval and 5 years thereafter exceeds 500 ppsm but is not well in excess of this value, as shown on Table 2.1.3-207. Safety, environmental, and economic factors were taken into account when identifying this site as the preferred alternative. The HAR site is identified as a superior site because there is an existing facility on site that provides already existing safety infrastructure of a skilled labor force with 20 years of operational experience, an existing Emergency Plan, and a surrounding population that is educated and informed about nuclear safety due to having lived with the HNP for 20 years. Environmental factors, superior seismic characteristics, existing transmission corridors, and proximity to the load center, as well as worker and transportation infrastructure, have already been established for the HNP and will be available to the HAR.

STD DEP 1.1-1 2.1.4 COMBINED LICENSE INFORMATION FOR GEOGRAPHY AND DEMOGRAPHY

HAR COL 2.1-1 This COL item is addressed in Subsection 2.1.

2.1.5 REFERENCES

2.1-201 North Carolina Department of Environment and Natural Resources (NCDENR), "Geologic Map of the New Hill 7.5-Minute Quadrangle, Wake and Chatham Counties, North Carolina," Draft, 2003.

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Table 2.1.1-201
Coordinates of Proposed Reactors

Reactor Unit	Latitude	Longitude	State Plane Northing	State Plane Easting	UTM Zone 17N Northing	UTM Zone 17N Easting
2	35 38 15.39	-78 57 29.84	686990.26	2012392.80	3945674.61	684865.21
3	35 38 23.90	-78 57 34.71	687851.26	2011991.31	3945934.53	684737.50

Source: Reference 2.1-201

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

USGS Quad ID	USGS Quad Name	State Name
35079-G1	Farrington	North Carolina
35078-G8	Green Level	North Carolina
35078-G7	Cary	North Carolina
35079-F2	Pittsboro	North Carolina
35079-F1	Merry Oaks	North Carolina
35078-F8	New Hill	North Carolina
35078-F7	Apex	North Carolina
35079-E2	Colon	North Carolina
35079-E1	Moncure	North Carolina
35078-E8	Cokesbury	North Carolina
35078-E7	Fuquay-Varina	North Carolina
35079-D1	Broadway	North Carolina
35078-D8	Mamers	North Carolina

Source: Reference 2.1-223

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

	Distance	
Sector	**Meters**	**Feet**
N	1378	4522
NNE	1342	4402
NE	1291	4234
ENE	1247	4090
E	1303	4275
ESE	1707	5600
SE	1739	5705
SSE	1749	5738
S	1733	5686
SSW	1697	5568
SW	1646	5400
WSW	1602	5256
W	1303	4275
WNW	1352	4436
NW	1384	4540
NNW	1394	4572

Note: The distances reported in this table represent the approximate distances from the midpoint between Units 2 and 3 to the outermost boundary of the exclusion area boundaries at the centerpoint of each sector as shown in Figure 2.1.1-203.

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Table 2.1.3-201 (Sheet 1 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
North-Residential	0	22	82	89	119	999	**1311**
North-Transient	0	0	0	0	0	3	**3**
North North East-Residential	0	20	121	166	168	7755	**8230**
North North East-Transient	0	0	0	0	0	898	**898**
North East-Residential	0	23	81	90	138	12,619	**12,951**
North East-Transient	0	0	0	0	0	8845	**8845**
East North East-Residential	0	5	20	24	23	6999	**7071**
East North East-Transient	0	0	0	0	0	219	**219**
East-Residential	0	3	11	14	106	9006	**9140**
East-Transient	0	0	0	0	0	3224	**3224**
East South East-Residential	1	3	4	29	52	8183	**8272**
East South East-Transient	0	78	63	0	0	4053	**4194**
South East-Residential	0	3	3	14	52	2238	**2310**
South East-Transient	0	77	9	0	0	529	**615**
South South East-Residential	0	0	3	2	22	898	**925**
South South East-Transient	0	12	0	0	1	4	**17**
South-Residential	0	0	0	1	2	283	**286**
South-Transient	0	0	0	1	1	6	**8**
South South West-Residential	0	0	1	14	30	611	**656**
South South West-Transient	0	0	0	1	1	5	**7**
South West-Residential	0	0	5	19	22	303	**349**
South West-Transient	0	0	0	1	1	476	**478**
West South West-Residential	0	0	6	20	62	961	**1049**
West South West-Transient	0	0	0	1	1	1467	**1469**
West-Residential	0	0	29	66	73	1049	**1217**
West-Transient	0	0	165	1	1	225	**392**

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Table 2.1.3-201 (Sheet 2 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
West North West-Residential	0	7	35	58	78	257	**435**
West North West-Transient	0	0	0	1	1	3	**5**
North West-Residential	2	19	37	51	51	439	**599**
North West-Transient	0	0	0	1	1	4	**6**
North North West-Residential	2	35	49	24	34	274	**418**
North North West-Transient	0	0	0	0	0	3985	**3985**
Residential Total	5	140	487	681	1032	52,874	**55,219**
Cumulative Total (Residential plus Transient)	5	307	724	688	1040	76,820	**79,584**

Note: To account for the difference in distance between each HAR unit and the HAR 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.

Source: Reference 2.1-205

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Table 2.1.3-202 (Sheet 1 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
North-Residential							
2010 Population	1	31	115	123	163	1300	**1733**
2020 Population	1	40	149	158	209	1601	**2157**
2030 Population	1	50	184	195	257	1914	**2600**
2040 Population	1	65	241	254	333	2386	**3281**
2050 Population	1	85	316	331	432	2981	**4146**
2060 Population	2	112	413	432	560	3730	**5249**
2070 Population	2	146	541	563	728	4675	**6656**
2080 Population	3	192	709	735	946	5871	**8455**
North-Transient							
2010 Population	0	0	0	0	0	4	**4**
2020 Population	0	0	0	0	0	5	**5**
2030 Population	0	0	0	0	0	6	**6**
2040 Population	0	0	0	0	0	8	**8**
2050 Population	0	0	0	0	0	10	**10**
2060 Population	0	0	0	0	0	12	**12**
2070 Population	0	0	0	0	0	16	**16**
2080 Population	0	0	0	0	0	20	**20**
North North East-Residential							
2010 Population	0	28	168	231	235	10,828	**11,490**
2020 Population	0	36	218	299	304	13,996	**14,852**
2030 Population	0	44	270	370	376	17,351	**18,412**
2040 Population	0	58	353	485	493	22,721	**24,110**
2050 Population	0	76	463	635	646	29,753	**31,573**
2060 Population	0	99	606	832	845	38,962	**41,345**
2070 Population	0	130	794	1089	1107	51,021	**54,141**
2080 Population	0	170	1039	1427	1450	66,812	**70,898**

HAR COL 2.1-1

Table 2.1.3-202 (Sheet 2 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
North North East-Transient							
2010 Population	0	0	0	0	0	1254	**1254**
2020 Population	0	0	0	0	0	1621	**1621**
2030 Population	0	0	0	0	0	2009	**2009**
2040 Population	0	0	0	0	0	2631	**2631**
2050 Population	0	0	0	0	0	3445	**3445**
2060 Population	0	0	0	0	0	4512	**4512**
2070 Population	0	0	0	0	0	5908	**5908**
2080 Population	0	0	0	0	0	7736	**7736**
North East-Residential							
2010 Population	0	33	113	125	193	17,618	**18,083**
2020 Population	0	42	147	162	250	22,773	**23,374**
2030 Population	0	52	182	201	310	28,232	**28,977**
2040 Population	0	69	238	263	406	36,970	**37,946**
2050 Population	0	90	312	345	531	48,413	**49,690**
2060 Population	0	118	408	451	696	63,396	**65,069**
2070 Population	0	154	534	591	911	83,018	**85,209**
2080 Population	1	202	700	774	1193	108,712	**111,581**
North East-Transient							
2010 Population	0	0	0	0	0	12,350	**12,350**
2020 Population	0	0	0	0	0	15,963	**15,963**
2030 Population	0	0	0	0	0	19,789	**19,789**
2040 Population	0	0	0	0	0	25,914	**25,914**
2050 Population	0	0	0	0	0	33,935	**33,935**
2060 Population	0	0	0	0	0	44,438	**44,438**
2070 Population	0	0	0	0	0	58,191	**58,191**
2080 Population	0	0	0	0	0	76,201	**76,201**

HAR 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
East North East-Residential							
2010 Population	1	7	28	33	33	9772	**9874**
2020 Population	1	9	36	43	42	12,631	**12,763**
2030 Population	1	12	45	53	52	15,659	**15,822**
2040 Population	2	15	58	70	68	20,506	**20,719**
2050 Population	2	20	76	92	89	26,852	**27,132**
2060 Population	3	26	100	120	117	35,163	**35,529**
2070 Population	3	34	131	157	153	46,046	**46,526**
2080 Population	4	45	172	206	201	60,298	**60,926**
East North East-Transient							
2010 Population	0	0	0	0	0	306	**306**
2020 Population	0	0	0	0	0	395	**395**
2030 Population	0	0	0	0	0	490	**490**
2040 Population	0	0	0	0	0	642	**642**
2050 Population	0	0	0	0	0	840	**840**
2060 Population	0	0	0	0	0	1100	**1100**
2070 Population	0	0	0	0	0	1441	**1441**
2080 Population	0	0	0	0	0	1887	**1887**
East-Residential							
2010 Population	0	4	16	20	148	12,574	**12,761**
2020 Population	0	5	21	25	191	16,252	**16,494**
2030 Population	0	6	26	31	237	20,148	**20,448**
2040 Population	0	7	34	41	310	26,385	**26,777**
2050 Population	0	10	44	54	406	34,551	**35,064**
2060 Population	0	13	58	70	532	45,244	**45,917**
2070 Population	0	17	75	92	696	59,247	**60,128**
2080 Population	1	22	99	121	912	77,585	**78,738**

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Table 2.1.3-202 (Sheet 4 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
East-Transient							
2010 Population	0	0	0	0	0	4501	**4501**
2020 Population	0	0	0	0	0	5818	**5818**
2030 Population	0	0	0	0	0	7213	**7213**
2040 Population	0	0	0	0	0	9446	**9446**
2050 Population	0	0	0	0	0	12,369	**12,369**
2060 Population	0	0	0	0	0	16,198	**16,198**
2070 Population	0	0	0	0	0	21,211	**21,211**
2080 Population	0	0	0	0	0	27,776	**27,776**
East South East-Residential							
2010 Population	0	5	5	40	72	11,425	**11,547**
2020 Population	0	6	7	52	93	14,767	**14,925**
2030 Population	0	8	8	64	115	18,308	**18,503**
2040 Population	0	10	11	84	151	23,974	**24,230**
2050 Population	0	13	14	110	198	31,394	**31,729**
2060 Population	0	17	18	144	259	41,111	**41,549**
2070 Population	0	23	24	188	339	53,835	**54,409**
2080 Population	0	30	32	247	444	70,497	**71,249**
East South East-Transient							
2010 Population	0	109	88	0	0	5659	**5856**
2020 Population	0	140	114	0	0	7314	**7568**
2030 Population	0	176	141	0	0	9068	**9385**
2040 Population	0	230	185	0	0	11,874	**12,288**
2050 Population	0	301	242	0	0	15,549	**16,092**
2060 Population	0	394	317	0	0	20,362	**21,073**
2070 Population	0	515	415	0	0	26,664	**27,593**
2080 Population	0	675	543	0	0	34,916	**36,134**

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Table 2.1.3-202 (Sheet 5 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
South East-Residential							
2010 Population	0	4	4	19	72	2879	**2979**
2020 Population	0	6	5	25	93	3570	**3699**
2030 Population	0	7	7	31	114	4291	**4450**
2040 Population	0	9	9	41	149	5349	**5556**
2050 Population	0	12	11	53	194	6677	**6947**
2060 Population	0	15	15	70	253	8346	**8699**
2070 Population	0	20	20	91	330	10,449	**10,910**
2080 Population	0	26	26	120	430	13,102	**13,703**
South East-Transient							
2010 Population	0	108	13	0	0	701	**822**
2020 Population	0	140	16	0	0	883	**1039**
2030 Population	0	173	20	0	0	1074	**1267**
2040 Population	0	227	26	0	0	1365	**1618**
2050 Population	0	297	35	0	0	1738	**2069**
2060 Population	0	388	45	0	0	2215	**2649**
2070 Population	0	509	59	0	0	2828	**3396**
2080 Population	0	666	78	0	0	3614	**4358**
South South East-Residential							
2010 Population	0	1	4	2	27	1111	**1145**
2020 Population	0	1	5	3	33	1348	**1390**
2030 Population	0	1	6	3	39	1592	**1643**
2040 Population	0	1	8	5	48	1928	**1989**
2050 Population	0	2	11	6	58	2333	**2409**
2060 Population	0	3	14	8	70	2824	**2918**
2070 Population	0	3	18	10	84	3419	**3535**
2080 Population	0	4	24	13	102	4138	**4282**

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Table 2.1.3-202 (Sheet 6 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
mi.	**0-1**	**1-2**	**2-3**	**3-4**	**4-5**	**5-10**	**Sector**
South South East-Transient							
2010 Population	0	17	0	0	1	5	**23**
2020 Population	0	22	0	0	2	6	**29**
2030 Population	0	27	0	0	2	7	**36**
2040 Population	0	35	0	0	2	9	**46**
2050 Population	0	46	0	0	3	10	**59**
2060 Population	0	61	0	0	3	13	**76**
2070 Population	0	79	0	0	4	15	**99**
2080 Population	0	104	0	0	5	18	**127**
South-Residential							
2010 Population	0	0	0	1	2	349	**353**
2020 Population	0	0	0	1	3	420	**424**
2030 Population	0	0	0	2	3	492	**497**
2040 Population	0	0	1	2	4	593	**599**
2050 Population	0	0	1	2	4	713	**721**
2060 Population	0	0	1	3	5	859	**868**
2070 Population	0	0	1	3	7	1034	**1046**
2080 Population	0	0	2	4	8	1246	**1259**
South-Transient							
2010 Population	0	0	0	1	1	7	**10**
2020 Population	0	0	0	2	1	9	**12**
2030 Population	0	0	0	2	2	10	**14**
2040 Population	0	0	0	2	2	13	**17**
2050 Population	0	0	0	3	3	15	**20**
2060 Population	0	0	0	3	3	18	**24**
2070 Population	0	0	0	4	4	22	**29**
2080 Population	0	0	0	4	4	27	**36**

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Table 2.1.3-202 (Sheet 7 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
South South West-Residential							
2010 Population	0	0	2	17	38	729	**786**
2020 Population	0	0	2	20	45	846	**914**
2030 Population	0	0	3	24	53	968	**1048**
2040 Population	0	0	3	29	64	1130	**1226**
2050 Population	0	0	4	35	77	1319	**1434**
2060 Population	0	0	4	42	92	1540	**1678**
2070 Population	0	0	5	50	111	1798	**1965**
2080 Population	0	0	6	60	134	2100	**2300**
South South West-Transient							
2010 Population	0	0	0	1	1	6	**8**
2020 Population	0	0	0	2	1	7	**10**
2030 Population	0	0	0	2	2	8	**11**
2040 Population	0	0	0	2	2	9	**13**
2050 Population	0	0	0	3	3	11	**16**
2060 Population	0	0	0	3	3	13	**19**
2070 Population	0	0	0	4	4	15	**22**
2080 Population	0	0	0	4	4	17	**26**
South West-Residential							
2010 Population	0	0	6	24	27	364	**421**
2020 Population	0	0	7	28	33	425	**493**
2030 Population	0	0	9	33	38	488	**568**
2040 Population	0	0	10	40	46	573	**669**
2050 Population	0	0	12	48	55	673	**788**
2060 Population	0	0	15	58	67	790	**929**
2070 Population	0	0	18	69	80	928	**1095**
2080 Population	0	0	22	83	97	1090	**1292**

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Table 2.1.3-202 (Sheet 8 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
South West-Transient							
2010 Population	0	0	0	1	1	567	**569**
2020 Population	0	0	0	2	1	657	**660**
2030 Population	0	0	0	2	2	751	**754**
2040 Population	0	0	0	2	2	874	**879**
2050 Population	0	0	0	3	3	1018	**1023**
2060 Population	0	0	0	3	3	1186	**1192**
2070 Population	0	0	0	4	4	1382	**1389**
2080 Population	0	0	0	4	4	1610	**1618**
West South West-Residential							
2010 Population	0	0	8	25	78	1152	**1263**
2020 Population	0	0	10	30	93	1341	**1473**
2030 Population	0	0	11	35	108	1537	**1691**
2040 Population	0	0	13	42	130	1798	**1984**
2050 Population	0	1	16	51	157	2104	**2328**
2060 Population	0	1	19	61	188	2464	**2733**
2070 Population	0	1	23	73	227	2885	**3209**
2080 Population	0	1	28	88	273	3379	**3770**
West South West-Transient							
2010 Population	0	0	0	1	1	1751	**1753**
2020 Population	0	0	0	2	1	2032	**2035**
2030 Population	0	0	0	2	2	2325	**2329**
2040 Population	0	0	0	2	2	2713	**2717**
2050 Population	0	0	0	3	3	3165	**3170**
2060 Population	0	0	0	3	3	3694	**3700**
2070 Population	0	0	0	4	4	4311	**4319**
2080 Population	0	0	0	4	4	5034	**5043**

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Table 2.1.3-202 (Sheet 9 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
West-Residential							
2010 Population	0	1	36	83	91	1305	**1515**
2020 Population	0	1	43	99	108	1554	**1805**
2030 Population	0	1	50	115	126	1809	**2102**
2040 Population	0	1	61	139	152	2170	**2523**
2050 Population	0	1	73	167	183	2603	**3028**
2060 Population	0	2	88	201	220	3124	**3635**
2070 Population	0	2	106	242	265	3749	**4364**
2080 Population	0	2	127	291	319	4500	**5240**
West-Transient							
2010 Population	0	0	203	1	1	278	**483**
2020 Population	0	0	248	2	1	330	**580**
2030 Population	0	0	285	2	2	383	**672**
2040 Population	0	0	345	2	2	458	**807**
2050 Population	0	0	413	3	3	547	**965**
2060 Population	0	0	495	3	3	655	**1156**
2070 Population	0	0	600	4	4	783	**1390**
2080 Population	0	0	720	4	4	936	**1665**
West North West-Residential							
2010 Population	0	9	44	73	98	321	**544**
2020 Population	0	11	52	87	117	383	**650**
2030 Population	0	14	61	101	136	447	**759**
2040 Population	0	17	74	122	164	538	**914**
2050 Population	0	22	89	147	197	647	**1102**
2060 Population	0	28	107	177	237	779	**1327**
2070 Population	1	35	128	213	285	937	**1599**
2080 Population	1	45	155	256	343	1128	**1927**

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Table 2.1.3-202 (Sheet 10 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
West North West-Transient							
2010 Population	0	0	0	1	1	4	**6**
2020 Population	0	0	0	2	1	4	**7**
2030 Population	0	0	0	2	2	5	**9**
2040 Population	0	0	0	2	2	6	**10**
2050 Population	0	0	0	3	3	8	**13**
2060 Population	0	0	0	3	3	9	**15**
2070 Population	0	0	0	4	4	11	**18**
2080 Population	0	0	0	4	4	13	**22**
North West-Residential							
2010 Population	2	25	46	64	63	548	**749**
2020 Population	3	31	55	76	76	655	**896**
2030 Population	4	37	64	89	88	764	**1046**
2040 Population	5	47	77	107	106	919	**1262**
2050 Population	7	60	92	129	128	1106	**1522**
2060 Population	9	76	111	155	154	1332	**1837**
2070 Population	11	97	134	187	185	1603	**2216**
2080 Population	15	123	161	225	222	1929	**2675**
North West-Transient							
2010 Population	0	0	0	1	1	5	**7**
2020 Population	0	0	0	2	1	6	**9**
2030 Population	0	0	0	2	2	7	**10**
2040 Population	0	0	0	2	2	8	**13**
2050 Population	0	0	0	3	2	10	**15**
2060 Population	0	0	0	3	3	12	**18**
2070 Population	0	0	0	4	4	15	**22**
2080 Population	0	0	0	4	4	18	**26**

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Table 2.1.3-202 (Sheet 11 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
mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
North North West-Residential							
2010 Population	2	48	64	30	42	342	**528**
2020 Population	3	61	79	36	50	409	**638**
2030 Population	4	76	94	42	59	477	**751**
2040 Population	5	98	118	50	71	574	**915**
2050 Population	7	128	147	60	85	690	**1117**
2060 Population	9	167	184	73	102	831	**1365**
2070 Population	11	217	231	87	123	1000	**1670**
2080 Population	15	283	290	105	148	1203	**2045**
North North West-Transient							
2010 Population	0	0	0	0	0	4979	**4979**
2020 Population	0	0	0	0	0	5947	**5947**
2030 Population	0	0	0	0	0	6937	**6937**
2040 Population	0	0	0	0	0	8348	**8348**
2050 Population	0	0	0	0	0	10,047	**10,047**
2060 Population	0	0	0	0	0	12,091	**12,091**
2070 Population	0	0	0	0	0	14,552	**14,552**
2080 Population	0	0	0	0	0	17,512	**17,512**

2010 Population							
Residential Total	6	194	660	911	1382	72,617	**75,771**
Cumulative Total (Residential plus Transient)	6	428	963	920	1392	104,993	**108,703**
2020 Population							
Residential Total	8	249	835	1145	1739	92,970	**96,946**
Cumulative Total (Residential plus Transient)	8	551	1213	1156	1751	133,967	**138,645**
2030 Population							
Residential Total	10	307	1019	1391	2113	114,477	**119,318**
Cumulative Total (Residential plus Transient)	10	683	1465	1403	2127	164,561	**170,250**

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Table 2.1.3-202 (Sheet 12 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
	mi.	0-1	1-2	2-3	3-4	4-5	5-10	Sector
2040 Population								
Residential Total		13	400	1308	1774	2694	148,512	**154,700**
Cumulative Total (Residential plus Transient)		13	891	1864	1788	2711	212,830	**220,097**
2050 Population								
Residential Total		17	520	1680	2265	3440	192,809	**200,731**
Cumulative Total (Residential plus Transient)		17	1163	2369	2282	3460	275,528	**284,820**
2060 Population								
Residential Total		23	676	2162	2896	4398	250,493	**260,648**
Cumulative Total (Residential plus Transient)		23	1520	3019	2917	4423	357,020	**368,921**
2070 Population								
Residential Total		30	880	2784	3708	5632	325,643	**338,678**
Cumulative Total (Residential plus Transient)		30	1983	3858	3734	5662	463,006	**478,273**
2080 Population								
Residential Total		39	1146	3589	4755	7222	423,588	**440,340**
Cumulative Total (Residential plus Transient)		39	2591	4930	4786	7258	600,925	**620,529**

Note: To account for the difference in distance between each HAR unit and the HAR 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.

Source: Reference 2.1-205

HAR 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	26,833	150,895	25,818	13,975	**217,521**
North-Transient	1941	3286	938	134	**6299**
North North East-Residential	22,862	45,342	20,709	12,057	**100,970**
North North East-Transient	2724	2646	410	177	**5957**
North East-Residential	107,335	150,622	34,588	19,172	**311,717**
North East-Transient	3514	2582	385	357	**6838**
East North East-Residential	75,967	108,363	32,014	19,983	**236,327**
East North East-Transient	1242	2334	110	327	**4013**
East-Residential	27,829	32,145	23,381	18,594	**101,949**
East-Transient	21	588	82	152	**843**
East South East-Residential	16,905	15,620	22,132	13,936	**68,593**
East-South-East-Transient	174	62	1,015	72	**1323**
South East-Residential	12,282	26,062	15,429	9791	**63,564**
South East-Transient	13	305	719	111	**1148**
South South East-Residential	6903	8949	10,377	17,340	**43,569**
South South East-Transient	395	37	240	117	**789**
South-Residential	4777	18,020	138,693	134,349	**295,839**
South-Transient	37	102	1,803	2727	**4669**
South South West-Residential	7886	11,707	10,206	18,915	**48,714**
South South West-Transient	96	40	103	227	**466**
South West-Residential	25,432	10,090	20,419	32,649	**88,590**
South West-Transient	706	122	551	2570	**3949**
West South West-Residential	5718	4275	7311	9829	**27,133**
West South West-Transient	15	670	99	156	**940**
West-Residential	2490	6695	8460	44,116	**61,761**
West-Transient	22	30	25	458	**535**
West North West-Residential	4246	8651	13,768	30,994	**57,659**
West North West-Transient	42	151	3	24	**220**

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

km	16-32	32-48	48-64	64-80	Total for
mi.	10-20	20-30	30-40	40-50	Sector
North West-Residential	6672	7698	42,007	74,026	**130,403**
North West-Transient	42	98	1103	816	**2059**
North North West-Residential	40,738	38,246	29,766	10,368	**119,118**
North North West-Transient	351	476	395	56	**4508**
Residential Total	**394,875**	**643,380**	**455,078**	**480,094**	**1,973,427**
Cumulative Total (Residential plus Transient)	**409,440**	**656,909**	**463,059**	**488,575**	**2,017,983**

Note: To account for the difference in distance between each HAR unit and the HAR 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.

Source: Reference 2.1-205

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**Table 2.1.3-204 (Sheet 1 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
North-Residential					
2010 Population	31,174	174,193	29,690	15,383	**250,440**
2020 Population	35,615	198,108	33,697	17,015	**284,434**
2030 Population	40,156	222,577	37,729	18,548	**319,010**
2040 Population	45,981	253,399	42,824	20,391	**362,594**
2050 Population	52,681	288,500	48,610	22,419	**412,210**
2060 Population	60,394	328,474	55,183	24,653	**468,705**
2070 Population	69,285	373,999	62,649	27,114	**533,047**
2080 Population	79,543	425,848	71,130	29,826	**606,347**
North-Transient					
2010 Population	2255	3793	1079	148	**7275**
2020 Population	2576	4314	1224	163	**8277**
2030 Population	2905	4847	1371	178	**9301**
2040 Population	3326	5518	1556	196	**10,596**
2050 Population	3811	6283	1766	215	**12,075**
2060 Population	4369	7153	2005	236	**13,763**
2070 Population	5012	8144	2276	260	**15,692**
2080 Population	5754	9274	2584	286	**17,898**
North North East-Residential					
2010 Population	31,480	55,126	24,834	14,220	**125,661**
2020 Population	40,362	65,163	29,155	16,509	**151,189**
2030 Population	49,762	75,652	33,437	18,743	**177,594**
2040 Population	64,703	90,492	39,330	21,733	**216,257**
2050 Population	84,203	108,719	46,327	25,209	**264,458**
2060 Population	109,664	131,214	54,652	29,251	**324,781**
2070 Population	142,921	159,103	64,582	33,951	**400,556**
2080 Population	186,375	193,835	76,455	39,417	**496,082**
North North East-Transient					
2010 Population	3751	3217	492	209	**7669**
2020 Population	4809	3803	577	242	**9431**
2030 Population	5929	4415	662	275	**11,281**
2040 Population	7709	5281	779	319	**14,088**
2050 Population	10,033	6344	917	370	**17,664**
2060 Population	13,066	7657	1082	429	**22,234**
2070 Population	17,029	9285	1279	498	**28,091**
2080 Population	22,207	11,312	1514	579	**35,612**

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Table 2.1.3-204 (Sheet 2 of 10)
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 East-Residential					
2010 Population	149,864	210,302	47,070	24,008	**431,244**
2020 Population	193,707	271,827	59,926	28,969	**554,430**
2030 Population	240,145	336,993	73,452	34,048	**684,638**
2040 Population	314,471	441,293	94,721	41,335	**891,820**
2050 Population	411,801	577,876	122,273	50,224	**1,162,173**
2060 Population	539,255	756,730	157,991	61,069	**1,515,045**
2070 Population	706,156	990,942	204,326	74,303	**1,975,727**
2080 Population	924,715	1,297,642	264,472	90,458	**2,577,288**
North East-Transient					
2010 Population	4906	3605	524	447	**9482**
2020 Population	6342	4660	667	539	**12,208**
2030 Population	7862	5777	818	634	**15,091**
2040 Population	10,295	7565	1054	770	**19,684**
2050 Population	13,482	9906	1361	935	**25,684**
2060 Population	17,654	12,972	1759	1137	**33,522**
2070 Population	23,119	16,987	2274	1384	**43,764**
2080 Population	30,274	22,245	2944	1684	**57,147**
East North East-Residential					
2010 Population	106,067	151,299	44,588	24,373	**326,327**
2020 Population	137,098	195,562	57,566	28,861	**419,087**
2030 Population	169,964	242,444	71,325	33,437	**517,170**
2040 Population	222,569	317,482	93,282	40,038	**673,371**
2050 Population	291,455	415,743	122,010	48,130	**877,338**
2060 Population	381,662	544,418	159,597	58,077	**1,143,754**
2070 Population	499,788	712,917	208,778	70,337	**1,491,820**
2080 Population	654,474	933,568	273,132	85,485	**1,946,659**
East North East-Transient					
2010 Population	1734	3259	153	399	**5545**
2020 Population	2241	4212	198	472	**7123**
2030 Population	2779	5222	245	547	**8793**
2040 Population	3639	6838	321	655	**11,453**
2050 Population	4765	8955	419	788	**14,927**
2060 Population	6240	11,726	548	950	**19,464**
2070 Population	8171	15,355	717	1151	**25,394**
2080 Population	10,700	20,108	938	1399	**33,145**

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Table 2.1.3-204 (Sheet 3 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
East-Residential					
2010 Population	38,854	44,674	32,442	24,192	**140,162**
2020 Population	50,221	57,791	41,980	30,058	**180,050**
2030 Population	62,263	71,922	52,313	36,354	**222,852**
2040 Population	81,533	94,167	68,490	46,050	**290,241**
2050 Population	106,768	123,293	89,669	58,625	**378,356**
2060 Population	139,813	161,428	117,398	74,956	**493,596**
2070 Population	183,086	211,358	153,701	96,197	**644,342**
2080 Population	239,751	276,731	201,231	123,854	**841,567**
East-Transient					
2010 Population	29	817	114	198	**1158**
2020 Population	38	1057	147	246	**1488**
2030 Population	47	1316	183	297	**1843**
2040 Population	62	1723	240	376	**2401**
2050 Population	81	2255	314	479	**3129**
2060 Population	106	2953	412	613	**4084**
2070 Population	138	3866	539	786	**5329**
2080 Population	181	5062	706	1012	**6961**
East South East-Residential					
2010 Population	23,284	21,661	30,709	17,774	**93,428**
2020 Population	29,922	28,021	39,737	21,826	**119,506**
2030 Population	36,972	34,910	49,519	26,167	**147,568**
2040 Population	48,092	45,691	64,831	32,862	**191,476**
2050 Population	62,584	59,803	84,879	41,556	**248,823**
2060 Population	81,480	78,275	111,126	52,864	**323,745**
2070 Population	106,125	102,455	145,490	67,588	**421,658**
2080 Population	138,275	134,106	190,480	86,783	**549,644**
East South East-Transient					
2010 Population	240	86	1408	92	**1826**
2020 Population	308	111	1822	113	**2354**
2030 Population	381	139	2271	135	**2926**
2040 Population	495	181	2973	170	**3819**
2050 Population	644	237	3893	215	**4989**
2060 Population	839	311	5096	273	**6519**
2070 Population	1092	407	6672	349	**8520**
2080 Population	1423	532	8736	448	**11,139**

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**Table 2.1.3-204 (Sheet 4 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
South East-Residential					
2010 Population	15,216	32,699	19,286	11,296	**78,497**
2020 Population	18,477	40,012	23,560	13,116	**95,165**
2030 Population	21,846	47,631	28,099	15,056	**112,633**
2040 Population	26,480	58,337	34,534	17,420	**136,772**
2050 Population	32,100	71,507	42,570	20,184	**166,362**
2060 Population	38,918	87,726	52,631	23,423	**202,698**
2070 Population	47,190	107,718	65,263	27,228	**247,399**
2080 Population	57,227	132,392	81,161	31,711	**302,491**
South East-Transient					
2010 Population	16	383	899	128	**1426**
2020 Population	20	468	1098	149	**1735**
2030 Population	23	557	1309	171	**2060**
2040 Population	28	683	1609	197	**2517**
2050 Population	34	837	1984	229	**3084**
2060 Population	41	1027	2453	266	**3787**
2070 Population	50	1261	3041	309	**4661**
2080 Population	61	1549	3782	360	**5752**
South South East-Residential					
2010 Population	8538	10,718	10,882	18,531	**48,668**
2020 Population	10,357	12,736	11,728	20,295	**55,115**
2030 Population	12,236	14,803	12,518	22,036	**61,593**
2040 Population	14,810	17,600	13,369	23,963	**69,742**
2050 Population	17,927	20,971	14,303	26,110	**79,311**
2060 Population	21,700	25,035	15,332	28,508	**90,574**
2070 Population	26,266	29,938	16,468	31,189	**103,861**
2080 Population	31,793	35,855	17,727	34,193	**119,569**
South South East-Transient					
2010 Population	489	44	252	125	**910**
2020 Population	593	53	271	137	**1054**
2030 Population	700	61	290	149	**1200**
2040 Population	847	73	309	162	**1391**
2050 Population	1026	87	331	176	**1620**
2060 Population	1242	104	355	192	**1893**
2070 Population	1503	124	381	210	**2218**
2080 Population	1818	148	408	230	**2604**

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Table 2.1.3-204 (Sheet 5 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
South-Residential					
2010 Population	5908	21,609	142,896	141,150	**311,564**
2020 Population	7167	25,700	151,670	152,172	**336,709**
2030 Population	8467	29,894	159,341	162,470	**360,172**
2040 Population	10,249	35,570	166,905	174,610	**387,335**
2050 Population	12,406	42,412	174,833	188,746	**418,397**
2060 Population	15,017	50,663	183,144	205,427	**454,251**
2070 Population	18,177	60,619	191,856	225,379	**496,030**
2080 Population	22,002	72,636	200,991	249,554	**545,183**
South-Transient					
2010 Population	46	122	1858	2865	**4891**
2020 Population	56	145	1972	3089	**5262**
2030 Population	66	169	2071	3298	**5604**
2040 Population	79	201	2170	3544	**5994**
2050 Population	96	240	2273	3831	**6440**
2060 Population	116	287	2381	4170	**6954**
2070 Population	141	343	2494	4575	**7553**
2080 Population	170	411	2613	5065	**8259**
South South West-Residential					
2010 Population	9432	14,347	11,628	26,688	**62,094**
2020 Population	11,029	17,238	13,217	34,608	**76,093**
2030 Population	12,693	20,223	14,803	43,299	**91,018**
2040 Population	14,889	24,284	16,953	57,166	**113,293**
2050 Population	17,472	29,171	19,536	75,489	**141,667**
2060 Population	20,510	35,051	22,656	99,701	**177,917**
2070 Population	24,086	42,128	26,448	131,698	**224,360**
2080 Population	28,297	50,648	31,084	173,986	**284,015**
South South West-Transient					
2010 Population	115	49	117	320	**601**
2020 Population	134	59	133	415	**741**
2030 Population	155	69	149	520	**893**
2040 Population	181	83	171	686	**1121**
2050 Population	213	100	197	906	**1416**
2060 Population	250	120	229	1197	**1796**
2070 Population	293	144	267	1581	**2285**
2080 Population	344	173	314	2088	**2919**

HAR COL 2.1-1

Table 2.1.3-204 (Sheet 6 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
South West-Residential					
2010 Population	30,078	11,892	23,880	38,222	**104,073**
2020 Population	34,721	13,693	27,338	43,793	**119,545**
2030 Population	39,576	15,540	30,725	49,263	**135,104**
2040 Population	45,866	17,950	35,212	56,541	**155,569**
2050 Population	53,157	20,733	40,355	64,913	**179,158**
2060 Population	61,606	23,948	46,249	74,551	**206,354**
2070 Population	71,398	27,663	53,004	85,653	**237,718**
2080 Population	82,746	31,956	60,746	98,452	**273,900**
South West-Transient					
2010 Population	835	144	644	3009	**4632**
2020 Population	964	166	738	3447	**5315**
2030 Population	1099	188	829	3878	**5994**
2040 Population	1273	217	950	4451	**6891**
2050 Population	1476	251	1089	5110	**7926**
2060 Population	1710	290	1248	5868	**9116**
2070 Population	1982	334	1430	6742	**10,488**
2080 Population	2297	386	1639	7750	**12,072**
West South West-Residential					
2010 Population	6810	5133	8568	11,033	**31,543**
2020 Population	7896	5981	9826	12,318	**36,021**
2030 Population	9028	6846	11,065	13,600	**40,539**
2040 Population	10,517	8018	12,710	15,190	**46,435**
2050 Population	12,254	9396	14,601	16,984	**53,234**
2060 Population	14,280	11,014	16,777	19,010	**61,080**
2070 Population	16,644	12,917	19,280	21,299	**70,140**
2080 Population	19,403	15,155	22,160	23,889	**80,607**
West South West-Transient					
2010 Population	18	804	116	175	**1113**
2020 Population	21	937	133	196	**1287**
2030 Population	24	1073	150	216	**1463**
2040 Population	28	1257	172	241	**1698**
2050 Population	32	1473	198	270	**1973**
2060 Population	37	1726	227	302	**2292**
2070 Population	44	2024	261	338	**2667**
2080 Population	51	2375	300	379	**3105**

HAR COL 2.1-1

Table 2.1.3-204 (Sheet 7 of 10)
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					
2010 Population	3105	8365	9810	49,551	**70,832**
2020 Population	3705	9992	11,273	55,933	**80,903**
2030 Population	4318	11,656	12,769	62,458	**91,202**
2040 Population	5191	14,027	14,680	70,134	**104,032**
2050 Population	6239	16,881	16,895	78,753	**118,769**
2060 Population	7500	20,316	19,465	88,432	**135,714**
2070 Population	9016	24,450	22,452	99,302	**155,220**
2080 Population	10,840	29,425	25,926	111,507	**177,698**
West-Transient					
2010 Population	27	37	29	514	**607**
2020 Population	33	45	33	581	**692**
2030 Population	38	52	38	648	**776**
2040 Population	46	63	43	728	**880**
2050 Population	55	76	50	818	**999**
2060 Population	66	91	58	918	**1133**
2070 Population	80	110	66	1031	**1287**
2080 Population	96	132	77	1158	**1463**
West North West-Residential					
2010 Population	5306	10,764	15,868	34,781	**66,719**
2020 Population	6337	12,833	18,196	39,211	**76,577**
2030 Population	7392	14,953	20,597	43,742	**86,685**
2040 Population	8896	17,957	23,599	49,066	**99,518**
2050 Population	10,707	21,566	27,062	55,038	**114,373**
2060 Population	12,885	25,904	31,062	61,736	**131,588**
2070 Population	15,507	31,119	35,686	69,251	**151,562**
2080 Population	18,662	37,386	41,038	77,680	**174,766**
West North West-Transient					
2010 Population	52	188	3	27	**270**
2020 Population	63	224	4	30	**321**
2030 Population	73	261	4	34	**372**
2040 Population	88	313	5	38	**444**
2050 Population	106	376	6	43	**531**
2060 Population	127	452	7	48	**634**
2070 Population	153	543	8	54	**758**
2080 Population	185	653	9	60	**907**

HAR COL 2.1-1

Table 2.1.3-204 (Sheet 8 of 10)
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
mi.	10-20	20-30	30-40	40-50	Sector
North West-Residential					
2010 Population	8322	8785	47,529	83,629	**148,265**
2020 Population	9931	10,026	54,224	95,203	**169,385**
2030 Population	11,576	11,300	61,387	107,507	**191,769**
2040 Population	13,917	12,853	69,662	121,753	**218,185**
2050 Population	16,733	14,627	79,053	137,889	**248,302**
2060 Population	20,121	16,653	89,710	156,168	**282,651**
2070 Population	24,196	18,968	101,803	176,873	**321,840**
2080 Population	29,098	21,617	115,527	200,328	**366,569**
North West-Transient					
2010 Population	52	112	1248	922	**2334**
2020 Population	63	128	1424	1049	**2664**
2030 Population	73	144	1612	1185	**3014**
2040 Population	88	164	1829	1342	**3423**
2050 Population	105	186	2076	1520	**3887**
2060 Population	127	212	2356	1721	**4416**
2070 Population	152	241	2673	1950	**5016**
2080 Population	183	275	3033	2208	**5699**
North North West-Residential					
2010 Population	46,707	43,157	33,628	11,459	**134,951**
2020 Population	53,009	48,481	38,027	12,787	**152,304**
2030 Population	58,918	53,334	42,356	14,124	**168,732**
2040 Population	66,741	59,590	47,648	15,688	**189,668**
2050 Population	75,668	66,580	53,605	17,443	**213,295**
2060 Population	85,866	74,389	60,310	19,411	**239,975**
2070 Population	97,528	83,114	67,857	21,620	**270,120**
2080 Population	110,882	92,863	76,354	24,101	**304,200**
North North West-Transient					
2010 Population	4106	537	446	62	**5151**
2020 Population	4660	603	505	69	**5837**
2030 Population	5179	664	562	76	**6481**
2040 Population	5867	742	632	85	**7326**
2050 Population	6651	829	711	94	**8285**
2060 Population	7548	926	800	105	**9379**
2070 Population	8573	1034	900	117	**10,624**
2080 Population	9747	1156	1013	130	**12,046**

HAR COL 2.1-1

**Table 2.1.3-204 (Sheet 9 of 10)
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
2010 Population					
Residential Total	520,145	824,723	533,307	546,291	**2,424,467**
Cumulative Total (Residential plus Transient)	583,816	841,920	542,689	555,931	**2,479,357**
2020 Population					
Residential Total	649,554	1,013,164	621,120	622,675	**2,906,512**
Cumulative Total (Residential plus Transient)	672,475	1,034,149	632,066	633,612	**2,972,301**
2030 Population					
Residential Total	785,312	1,210,678	711,435	700,852	**3,408,277**
Cumulative Total (Residential plus Transient)	812,645	1,235,632	724,000	723,999	**3,485,369**
2040 Population					
Residential Total	994,906	1,508,712	838,751	803,940	**4,146,308**
Cumulative Total (Residential plus Transient)	1,028,957	1,539,614	853,564	817,900	**4,240,034**
2050 Population					
Residential Total	1,264,155	1,887,779	996,581	927,711	**5,076,225**
Cumulative Total (Residential plus Transient)	1,306,765	1,926,214	1,014,166	943,710	**5,190,854**
2060 Population					
Residential Total	1,610,670	2,371,239	1,193,281	1,077,237	**6,252,427**
Cumulative Total (Residential plus Transient)	1,664,208	2,419,246	1,214,297	1,095,662	**6,393,413**
2070 Population					
Residential Total	2,057,367	2,989,408	1,439,642	1,258,983	**7,745,401**
Cumulative Total (Residential plus Transient)	2,124,899	3,049,610	1,464,920	1,280,318	**7,919,748**
2080 Population					
Residential Total	2,634,084	3,781,664	1,749,614	1,481,222	**9,646,584**
Cumulative Total (Residential plus Transient)	2,719,575	3,857,455	1,780,224	1,506,058	**9,863,312**

HAR COL 2.1-1

Table 2.1.3-204 (Sheet 10 of 10)
Resident and Transient Population Projections between 16 and 80 km (10 and 50 mi.)

Note: To account for the difference in distance between each HAR unit and the HAR 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.

Source: Reference 2.1-205

HAR COL 2.1-1

Table 2.1.3-205
Recreational Areas within 8 km (5 mi.) of the HAR

Area	Average Daily Attendance	Approximate Distance and Direction
Harris Lake County Park	239	3 km (2 mi.) ESE
NCWRC Game Lands[a]	NA	5 km (3 mi.) SE
Jordan Lake State Park	3982	8 to 19 km (5 to 12 mi.) NW
TOTAL	4221	

a) NCWRC Game Lands do not have controls in place to keep track of daily attendance.

Note: NCWRC = North Carolina Wildlife Resources Commission

HAR COL 2.1-1

Table 2.1.3-206 (Sheet 1 of 4)
2000 Population of Cities and Communities within an 80-km (50-mi.) Radius

Place Name	Total Population in 2000	Distance		Direction
		Kilometers	Miles	
Holly Springs	9192	10.93	6.79	E
Apex	20,212	13.86	8.61	NE
Fuquay-Varina	7898	15.74	9.78	ESE
Cary	94,536	20.99	13.04	NE
Broadway	1015	21.63	13.44	SSW
Pittsboro	2226	22.05	13.70	WNW
Fearrington	903	22.13	13.75	NNW
Angier	3419	24.04	14.94	SE
Morrisville	5208	24.66	15.32	NNE
Sanford	23,220	26.49	16.46	SW
Lillington	2915	28.78	17.88	SSE
Garner	17,757	30.63	19.03	ENE
Buies Creek	2215	31.53	19.59	SE
Chapel Hill	48,715	33.49	20.81	NNW
Carrboro	16,782	33.72	20.95	NNW
Goldston	319	34.26	21.29	W
Raleigh	276,093	34.94	21.71	NE
Coats	1845	36.08	22.42	SE
Durham	187,035	38.83	24.13	N
Erwin	4537	43.03	26.74	SE
Cameron	151	43.60	27.09	SW
Knightdale	5958	45.40	28.21	ENE
Clayton	6973	45.53	28.29	E
Linden	127	46.04	28.61	SSE
Benson	2923	46.27	28.75	SE
Gorman	1002	46.96	29.18	NNE
Siler City	6966	47.12	29.28	WNW
Dunn	9196	47.22	29.34	SE

HAR COL 2.1-1

Table 2.1.3-206 (Sheet 2 of 4)
2000 Population of Cities and Communities within an 80-km (50-mi.) Radius

Place Name	Total Population in 2000	Distance		Direction
		Kilometers	Miles	
Saxapahaw	1418	48.14	29.91	NW
Spring Lake	8098	50.44	31.34	S
Hillsborough	5446	50.49	31.37	NNW
Vass	750	51.43	31.96	SW
Pope AFB	2583	51.63	32.08	S
Four Oaks	1424	52.38	32.55	ESE
Godwin	112	52.50	32.62	SSE
West Smithfield	59	53.41	33.19	E
Wake Forest	12,588	53.91	33.50	NE
Wilson's Mills	1291	54.36	33.78	E
Carthage	1871	54.38	33.79	SW
Rolesville	907	54.56	33.90	NE
Fort Bragg	29,183	55.33	34.38	S
Wendell	4247	55.47	34.47	ENE
Plain View	1820	55.81	34.68	SE
Wade	480	55.96	34.77	SSE
Falcon	328	56.29	34.98	SSE
Smithfield	11,510	56.38	35.03	ESE
Swepsonville	922	56.94	35.38	NW
Whispering Pines	2090	57.02	35.43	SW
Staley	3053	57.10	35.48	WNW
Butner	5792	57.95	36.01	NNE
Mebane	7284	58.56	36.39	NNW
Creedmoor	2232	59.22	36.80	NNE
Eastover	1376	60.48	37.58	SSE
Liberty	2661	60.91	37.85	WNW
Youngsville	651	61.19	38.02	NE
Selma	5914	61.28	38.08	E

HAR COL 2.1-1

Table 2.1.3-206 (Sheet 3 of 4)
2000 Population of Cities and Communities within an 80-km (50-mi.) Radius

Place Name	Total Population in 2000	Distance Kilometers	Distance Miles	Direction
Zebulon	4046	61.44	38.18	ENE
Robbins	1195	61.53	38.23	WSW
Graham	12,833	61.78	38.39	NW
Woodlawn	833	61.83	38.42	NNW
Fayetteville	121,015	62.65	38.93	S
Haw River	1908	62.76	39.00	NW
Southern Pines	10,918	63.94	39.73	SW
Ramseur	1588	64.07	39.81	W
Spivey's Corner	448	64.58	40.13	SE
Green Level	2042	64.68	40.19	NNW
Alamance	310	65.23	40.53	NW
Pine Level	1313	65.68	40.81	ESE
Stem	229	66.16	41.11	NNE
Burlington	44,917	67.061	41.67	NW
Pinehurst	9706	67.25	41.79	SW
Taylortown	845	67.51	41.95	SW
Franklinville	1258	67.80	42.13	W
Vander	1204	68.20	42.38	SSE
Micro	454	68.41	42.51	E
Newton Grove	606	68.91	42.82	SE
Franklinton	1745	68.99	42.87	NE
Aberdeen	3400	69.99	43.49	SW
Middlesex	838	70.02	43.51	ENE
Seven Lakes	3214	70.34	43.71	SW
Glen Raven	2750	71.23	44.26	NW
Ashley Heights	341	71.47	44.41	SSW
Elon College	6738	71.81	44.62	NW
Vann Crossroads	324	71.86	44.65	SE

HAR COL 2.1-1

Table 2.1.3-206 (Sheet 4 of 4)
2000 Population of Cities and Communities within an 80-km (50-mi.) Radius

Place Name	Total Population in 2000	Distance		Direction
		Kilometers	Miles	
Rockfish	2353	72.21	44.87	S
Stedman	664	72.87	45.28	SSE
Bunn	357	72.95	45.33	ENE
Hope Mills	11,237	73.87	45.90	S
Gibsonville	4372	74.11	46.05	NW
Kenly	1569	74.21	46.11	E
Princeton	1066	74.34	46.19	ESE
Whitsett	686	74.51	46.30	NW
Pinebluff	1109	75.00	46.60	SW
Silver City	1146	75.00	46.60	SSW
Seagrove	246	75.33	46.81	W
Foxfire	474	75.85	47.13	SW
Autryville	196	76.22	47.36	SSE
Raeford	3386	76.67	47.64	SSW
Five Points	306	77.26	48.01	SSW
Sedalia	618	77.72	48.29	NW
Bailey	670	77.81	48.35	ENE
Altamahaw-Ossipee	996	78.07	48.51	NW
Louisburg	3111	78.28	48.64	NE
Asheboro	21,672	78.49	48.77	W
Forest Oaks	3241	78.58	48.83	WNW
Star	807	79.65	49.49	WSW
Randleman	3557	79.78	49.57	WNW
Kittrell	148	79.92	49.66	NE
Pleasant Garden	4714	81.24	50.48	WNW

Source: Reference 2.1-219

HAR COL 2.1-1

Table 2.1.3-207
Estimated and Projected Residential and Transient Population Density within 80 km (50 mi.) of the HAR (People per Square Mile)

	People per Square Mile		
Year	**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	249	383	266
Year 2010	340	511	328
Year 2015	384	574	361
Year 2020	433	640	395

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

Source: Reference 2.1-205

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.

Subsection 2.2.1 of the DCD is renumbered as Subsection 2.2.4 and moved to the end of Section 2.2. This is being done to accommodate the incorporation of Regulatory Guide 1.206 numbering conventions for Section 2.2.

STD DEP 1.1-1

2.2.1 LOCATIONS AND ROUTES

HAR COL 2.2-1
HAR COL 3.3-1
HAR COL 3.5-1

Information was compiled on all significant 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. 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. Information was also gathered on military firing or bombing ranges and any nearby flight, holding, and landing patterns.

The proposed site is located in the southwest corner of Wake County near the cities and towns of Apex, Fuquay-Varina, Holly Springs, and Moncure in Wake County, North Carolina. The City of Sanford is located in adjacent Lee County.

The area immediately adjacent to the proposed Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR) site is primarily State forest lands, which are undeveloped and used periodically for recreation. Some recreational boating occurs on Harris Lake as a result of nearby boat ramps associated with the Wake County Government Harris Lake County Park and the nearby Shearon Harris Game Lands, which are State Wildlife Management Areas. (Reference 2.2-201) The Harris Lake County Park and nearby North Carolina Wildlife Resources Commission (NCWRC) Game Lands will be affected by the proposed increased lake level in the main reservoir, but this will have no new effect related to off-site hazards.

Industrial development in the immediate vicinity of the plant site is limited. The majority of the industrial development in an 80-kilometer (km) (50-mile [mi.]) radius of the plant site is located in the urbanized areas of Durham, Guilford, Alamance, and Orange Counties. Industrial development within a 16-km (10-mi.) radius of the plant site is primarily concentrated in the immediate vicinity of the cities and towns of Apex, Holly Springs, Fuquay-Varina, and Moncure, and is further discussed in Subsection 2.2.2.8, Projections of Industrial Growth. Four active quarrying and mining facilities are located in the vicinity of the project (as shown in Table 2.2.1-201 and Figure 2.2.1-201) (Reference 2.2-202).

The Raleigh-Durham area is a major transportation hub for central North Carolina. Both cities are served by rail lines and major interstate highways that serve both local and interstate traffic. Transportation corridors near the plant site, which include U.S. Highway 1, Old U.S. Highway 1, and State Road 42, are described in further detail in Subsection 2.2.2.5, Description of Highways.

Airports in the area include the Sanford-Lee County Regional Airport located approximately 14.5 km (9 mi.) southwest of the plant location, the Raleigh-Durham International Airport (RDU) located approximately 32 km (20 mi.) north-northeast of the plant site, and three private airstrips in the area surrounding the site. Information on the operations at these airports is provided in Subsection 2.2.2.7, Description of Airports. No significant military facilities are located within a 40-km (25-mi.) radius of the plant site (Reference 2.2-203). The nearest active military facility is Fort Bragg, located 56 km (35 mi.) to the south (Figure 2.1.1-201). Fort Bragg is a support base for Army Training Operations. Pope Air Force Base is co-located with Fort Bragg and is a support base for the 43rd Airlift Wing.

Railways in the area include the Norfolk Southern Railroad and the CSX Railroad. Three railroad segments are located within 8 km (5 mi.) of the plant location, including a spur line that connects to the plant site. These segments are described in further detail in Subsection 2.2.2.6, Description of Railways.

The Cape Fear River runs southwest of the plant site, but this portion of the river is not used for commercial traffic.

2.2.2 DESCRIPTIONS

According to the Carolina Power and Light (CP&L) "Shearon Harris Nuclear Power Plant Safety Analysis Report," 1983, Amendment 54, the Research Triangle and Raleigh/Wake County areas, including Apex, Fuquay-Varina, and Holly Springs, support light industry such as electronic component manufacturing, electronic research, fiber chemistry research, pharmaceutical research, health statistics studies, and air pollution research. Industries in the surrounding area include manufacturers of wood products and building materials such as bricks.

2.2.2.1 Description of Facilities

As shown in Table 2.2.2-201, industries within an 8-km (5-mi.) radius of the plant site include the existing Shearon Harris Nuclear Power Plant Unit 1 (HNP), a brick manufacturing facility, a wood chipping recycling facility, and a combined heavy truck repair facility and chip/mulch hauling facility.

2.2.2.2 Description of Products and Materials

There are no manufacturing facilities in the vicinity that utilize or store products that are considered hazardous. Therefore, they pose no safety hazard to the

plant site. Materials stored on-site at HNP are described in Section 6.4 of the Unit 1 FSAR. Chemicals stored on-site for HAR are described in DCD Table 6.4-1.

2.2.2.3 Description of Pipelines

The Dixie Pipeline Company owns an 8-in. liquefied petroleum gas (LPG) pipeline, which is located 2.5 km (8214 feet [ft.] or 1.6 mi.) from the plant site. This pipeline runs in a northeast-southwest direction and is buried at a depth of approximately 0.9 meters (m) (3 ft.) below the surface. The pipeline carries approximately 1600 barrels per hour at peak flow, at a maximum pressure of 1440 pounds per square inch (psi). The line terminates at Apex, North Carolina, where the fuel is stored and distributed for local use. The pipeline is not used for storage of gas at higher than normal pressures. There are no plans to carry any product other than liquefied propane in the pipeline.

2.2.2.4 Description of Waterways

The Cape Fear River north of Fayetteville is not navigable by barges or large boats. Therefore, river traffic on the Cape Fear River near the plant site is limited to recreational pleasure boats. Some recreational fishing boats use Harris Lake.

2.2.2.5 Description of Highways

The Raleigh-Durham area is a major transportation hub for central North Carolina. Both cities are served by rail lines and major interstate highways that carry both local and interstate traffic. The nearest interstate (Interstate 40) is located approximately 16 km (10 mi.) north-northeast of the plant site. Transportation corridors near the plant site include U.S. Highway 1, Old U.S. Highway 1, and State Road 42. U.S. Highway 1 is approximately 2 km (1.3 mi.) from the center of the plant site at its nearest point, and the Average Annual Daily Traffic (AADT) near the plant site in North Carolina is 18,000 vehicles. Old U.S. Highway 1 is approximately 3.2 km (2 mi.) from the center of the plant site at its nearest point. The AADT for Old U.S. Highway 1 near the plant site is 1800 vehicles. The nearest local roads are New Hill Holleman Road (AADT is 3800 vehicles), which is approximately 3.2 km (2 mi.) from the plant site at its nearest point, and Bartley Holleman Road, which is approximately 3.6 km (2.2 mi.) from the plant site at its nearest point (Reference 2.2-204 and Reference 2.2-205).

2.2.2.6 Description of Railways

The two primary railways in the area are the Norfolk Southern Railroad and the CSX Railroad. Three railroad segments are located within 8 km (5 mi.) of the plant location, including a spur line that connects to the plant site.

- The Bonsal – Durham segment, which is 4 km (2.5 mi.) northwest of the plant site.

- The Fuquay-Varina – Brickhaven segment, which is 6.9 km (4.3 mi.) south of the plant site.
- The Raleigh – Moncure segment, which is 3 km (1.9 mi.) northwest of the plant site.

The CSX Railroad passes north of the plant, and the Norfolk Southern Railroad crosses south of the Main Dam. Railway access to the plant is provided by a Progress Energy rail spur that connects to the CSX Railroad. The spur is used to transport spent fuel shipments. These shipments do not present a hazard to HAR 2 and HAR 3.

The New Hope Valley Railway (NHVRy) operates as a living history tourist attraction near the plant site. The NHVRy operates along the former Southern Railway System between Bonsal and New Hill (Reference 2.2-206). The NHVRy has been in operation since April 1984. The railway currently operates 1 or 2 days per month for approximately 6 months per year. Each trip lasts approximately 1 hour and occurs up to five times a day when the railway is open. Current hours of operation span May to December and include 8 scheduled days (40 1-hour trips) between August and December.

2.2.2.7 Description of Airports

There are no airports located within an 8-km (5-mi.) radius of the plant site (Figure 2.2.2-201). There are 62 airports located within the region (80 km [50 mi.] radius) of the HAR site (Figure 2.2.2-201). Of the 62 airports located within the region, 12 airports are within the immediate proximity of less than 32 km (20 mi.) from the plant site, which include one major public airport (RDU), two general public aviation airports (Sanford-Lee County Regional Airport and Triple W Airport), and nine privately owned airports. The nine privately owned airports are shown below (Reference 2.2-216):

- Bagwell
- Barclaysville Field
- CAG Farms
- Cox
- Deck Airpark
- Eagles Landing
- Fuquay/Angier Field
- Moretz Riverside Landing
- Womble Field

There are 50 airports located within the range of 32 km (20 mi.) and 80 km (50 mi.) from the project area, which includes two military airports (Pope Air Force

Base and Simmons Army Airfield), 11 public airports, and 37 privately owned airports. The 11 public airports are shown below (Reference 2.2-216) and the 37 privately owned airports are shown on Figure 2.2.2-201:

- Burlington-Alamance Regional
- Causey
- Fayetteville Regional
- Franklin County
- Harnett Regional
- Horace Williams
- Hurdle Field
- Johnston County
- Moore County
- Person County
- Southeast Greensboro

Only the RDU Airport is expected to grow substantially in the near future. A summary of operations data for the public airports within 32 km (20 mi.) of the plant site is provided in Table 2.2.2-202 and a summary of aircraft operations per year for the RDU airport is provided in Table 2.2.2-203.

Table 2.2.2-202 describes the types of aircraft and flying patterns for aircraft-associated public airports within 32 km (20 mi.) of the plant location. Figure 2.2.2-202 shows the Federal Aviation Administration (FAA) Temporary Flight Restrictions Map for the area. This figure shows nearby airports, as well as those around the plant location. The outer boundary of four airways are routed within 2 miles of the HAR site: IR718, V3-66-155, J207, and J52-55 (shown on Figure 2.2.2-202).

The Sanford-Lee County Regional Airport is located approximately 14.5 km (9 mi.) southwest of the plant location. This general aviation airport is accessed via Runway 3/21, which is 2 km (6500 ft.) by 30 m (100 ft.) and in good condition. There are approximately 89 aircraft based at the field (86 single-engine and three multi-engine), with approximately 129 aircraft operations a day (85 percent local general aviation – 110 flights, 13 percent transient general aviation – 17 flights, and 2 percent military – two flights). The Sanford-Lee County Regional Airport will be constructing new hanger storage and anticipates a 1 percent growth for 2007.

RDU is located approximately 31 km (19 mi.) northeast of the project. The majority of the aircraft operations at RDU in 2005 were commercial air carrier flights (31 percent), general aviation-itinerant (22 percent), and air taxi/commuter (45.2 percent). Only 1.7 percent of aircraft operations in 2005 were military

operations (Reference 2.2-207 and Reference 2.2-208). RDU has three major airways that pass within 16.1 km (10 mi.) of the site. But flight holding and landing patterns do not affect the plant location.

In 2006, 9.4 million passengers traveled through RDU, averaging approximately 25,000 passengers per day. From 2005 to 2006 the usage of RDU increased by one percent. The RDU Authority began major construction on the redevelopment and expansion of Terminal C in 2006. Terminal C will expand from 330,000 square feet (sq. ft.) to 890,000 sq. ft. and have a total of 32 gates. The Terminal C redevelopment and expansion will be conducted in two phases. Phase one is scheduled for completion in summer 2008, while phase two completion is projected for late 2010. The expansion will allow for accommodation of up to 12 million passengers per year. The budgeted cost of the expansion is $570 million.

Currently, Terminal A has 23 gates and Terminal C has 26 gates that service the airport for a total of 49 gates. As shown in Table 2.2.2-203 total operations in 2006 were 245,099. Following completion of the Terminal C expansion, RDU will have a total of 55 gates. Assuming 245,099 total operations at 49 gates, 5002 operations per gate per year were calculated. Using this assumption, the redevelopment and expansion of Terminal C has the potential to increase operations by approximately 30,012 operations (based on six gates multiplied by 5002 operations) or 12 percent in 2010.

2.2.2.8 Projections of Industrial Growth

The HNP is located in New Hill, Wake County, North Carolina. The HAR is located immediately north of the existing plant and south of U.S. Highway 1. An investigation was undertaken to identify industrial growth, which may be planned within an 8-km (5-mi.) radius of the plant site. Industrial growth patterns in the area were identified through a series of Internet searches and personal communications with representatives of local entities, as described in the following subsections. Additionally, existing plant documents, including the previous CP&L 1983 “Shearon Harris Nuclear Power Plant Safety Analysis Report,” were comprehensively reviewed.

The HNP is located in the southwest corner of Wake County, North Carolina, and the southeast corner of Chatham County, North Carolina. Industrial development within a 16-km (10-mi.) radius of the plant site is primarily concentrated in the immediate vicinity of the cities and towns of Apex, Holly Springs, Fuquay-Varina, and Moncure (Figure 2.2.1-201). The Southwest Wake Area Land Use Plan: Land Use Classification Map shows the westernmost portion of Wake County as primarily residential, with some office/research park and industrial uses along U.S. Highway 1 (Reference 2.2-209). Other large land areas include the State-managed Shearon Harris Game Lands, which are classified as forestry/light industry. The area west of the plant site is located in Chatham County. The Chatham County zoning categories for this area include heavy industrial use and office and institutional use along U.S. Highway 1 and Old U.S.

Highway 1, surrounded by low-density residential/agricultural use (Reference 2.2-210). Aerial photography indicates that the area immediately south of the plant site that is located in Harnett County is primarily rural and undeveloped.

No new industrial development is anticipated within the immediate vicinity of the plant site. Contacts have been made with the industrial development authorities of Chatham, Harnett, and Wake Counties, and there are no plans for any future significant industrial development within an 8-km (5-mi.) radius of the plant site.

The North Carolina Department of Commerce (NCDOC) tracks economic data for the State of North Carolina. NCDOC has divided the State into seven regional partnerships for economic development (Reference 2.2-211). The plant site is located within the Research Triangle Region, which is defined as Chatham, Durham, Franklin, Granville, Harnett, Johnston, Lee, Moore, Orange, Person, Vance, Wake, and Warren Counties. The plant site is located in the southwestern corner of Wake County and is immediately adjacent to Harnett and Chatham Counties. Table 2.2.2-204 shows the largest non-government employers in Chatham, Harnett, and Wake Counties. The majority of the jobs listed are in the industrial or manufacturing sector (Reference 2.2-212).

Between the years 1999 and 2003, the Research Triangle Region has experienced a robust increase in industrial investment, as shown in Table 2.2.2-205. In 2003, new and expanded industry investment in the region reached over $856 million and resulted in an estimated 5038 jobs (Reference 2.2-212).

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

Potential accidents in nearby transportation, industrial, and military activities are reviewed in this subsection to evaluate whether their effects might be of serious consequence to nuclear safety at the HAR site, with an annual probability of occurrence exceeding 10^{-7}. For events such as rail or truck accidental explosions or pipeline failure, where a probability has not been estimated, it is shown that, should the event occur, no consequence of a critical magnitude with respect to nuclear safety would be induced at the locations of the proposed Shearon Harris Nuclear Power Plant Unit 2 (HAR 2) and the proposed Shearon Harris Nuclear Power Plant Unit 3 (HAR 3) critical structures.

2.2.3.1 Determination of Design-Basis Events

As presented in Subsection 2.2.2, there are no industrial facilities within 8 km (5 mi.) of the plant that could affect the safety of the nuclear facility. A review of the materials transported or stored within 8 km (5 mi.) indicates that the only sources that present a potential hazard are rail and road transportation of high explosives, the nearby LPG pipeline, and the accidental release of toxic materials due to transportation accidents.

Design-basis events are those accidents that have a probability of occurrence on the order of 10^{-7} 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. Based on the NUREG-0800, the expected rate of occurrence exceeding the guidelines in 10 CFR 100 (on the order of 10^{-6} per year) is acceptable if, when combined with reasonable qualitative evaluations, the realistic probability can be shown to be lower.

2.2.3.1.1 Transportation of Explosives

Potential sources of explosions from nearby activities described in Subsection 2.2.2 would include an explosion in highway or railway transport. There is no indication that explosives are regularly transported past the plant site by highway or rail.

Other corridors, such as water and air, do not pose a potential hazard to the HAR facilities. Water traffic is presently limited to pleasure and/or fishing boats in the Harris Reservoir. There are no navigable connections to the Cape Fear River or related ports. There are no military facilities within an 8-km (5-mi.) radius of the plant site. A small general aviation airport is located southwest of the plant site, outside the 8-km (5-mi.) radius of the plant site.

According to the Nuclear Regulatory Commission's (NRC) "Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants," an explosion near the plant site may be considered a critical hazard, if it creates a blast overpressure exceeding the design-basis tornado dynamic wind pressure, or a peak incident overpressure of approximately 1 psi.

The maximum probable hazardous solid cargo for a single highway truck is 22,680 kilograms (kg) (50,000 pounds [lb.]). Assuming a Trinitrotoluene (TNT) equivalency ratio of 1, the distance from an explosion that would produce a peak incident pressure of 1 psi is 518 m (1700 ft.). The closest roadway to the HAR site is U.S. Highway 1. It is located approximately 2 km (1.3 mi.) from a critical plant structure. The estimated peak incident overpressure at this distance is approximately 0.4 psi relative to Westinghouse's AP-1000 Reactor (AP1000) plant design-basis value of 2.9 psi. Therefore, no adverse effects are anticipated due to the transport of explosives via roadway.

The maximum explosive in a single railroad boxcar is approximately 59,874 kg (132,000 lb.). Assuming a TNT equivalency ratio of 1, the distance from this explosive that would produce a 1 psi overpressure is approximately 698 m (2291 ft.) The nearest major railroad corridor is 3058 m (10,032 ft.) north of the plant site at its closest distance to a safety-related structure. The estimated peak incident overpressure at this distance is approximately 0.12 psi relative to the plant design-basis value of 2.9 psi. Therefore, no adverse effects on safety-related structures are anticipated due to a railway explosive accident.

2.2.3.1.2 Nearby Gas Pipeline

The Dixie Pipeline Company owns an 8-in LPG pipeline that is located 2.5 km (8214 ft. or 1.6 mi.) from critical plant structures. This pipeline, which runs in a northeast-southwest direction, is buried at a depth of approximately 0.9 m (3 ft.) below the surface. The pipeline carries approximately 1600 barrels per hour at peak flow at a maximum pressure of 1440 psi.

The effect of a postulated rupture of this LPG pipeline and a related explosion and fire hazard analysis was originally evaluated and documented in the HNP FSAR. This analysis, which represents the design-basis analysis for the HNP site, evaluated the effect of the pipeline break and the explosive overpressure on safety-related structures, fire hazard, seismic shock, and missile generation. The analysis for HAR 2 and HAR 3 uses the previously developed methods and assumptions to evaluate the effect of the line rupture and subsequent detonation of the released propane on the proposed AP1000 safety-related structures. This is accomplished by extrapolating and updating the results of the previous analyses to reflect the HAR 2 and HAR 3 site development features and considers the AP1000 design criteria for safety-related facilities.

The evaluation concludes that a pipeline failure, should it occur, would not result in damage to the HAR 2 and HAR 3 critical facilities that could impede the continued safe operation or prevent safe shutdown of the plant. This includes damage due to explosive impact, flying debris, thermal radiation, or the intrusion of an un-ignited or the delayed ignition of an LPG gas cloud. The updated analysis includes the following significant assumptions and results:

- The long-term break flow release rate due to the pipeline rupture is conservatively estimated at a constant rate of 1314 standard cubic feet per second (scfs). This value assumes that all of the released propane gas vaporizes.

- The maximum downwind extent and associated volume of the detonable propane-air cloud mixture is 1753 m (5750 ft.) and 453,070 cubic meters (m^3) (16 million cubic feet [ft^3]).

- The energy released is estimated at 162.5 tons TNT equivalent based on a 50-percent yield and assuming an average detonable mixture of 4.9-percent propane.

- The closest point of approach of the detonable propane-air cloud mixture to the plant critical structures is estimated at 751 m (2464 ft.) and is the value used for determining overpressure.

- The blast and seismic parameters for shock waves are as follows:

  - Peak overpressure = 1.3 psi

- Peak dynamic pressure = 0.041 psi
- Peak reflected pressure = 2.57 psi
- Positive phase duration = 330 milliseconds
- Peak acceleration = 0.131 g
- Peak velocity = 0.175 ft/sec
- Peak displacement = 0.015 in.

- The mass of debris ejected due to the explosion is estimated at 3.1 kg (6.8 lb.). The maximum impact due to a missile of this size is estimated at 627.0 Newton-meter (N-m) (462.5 ft.-lb.).

- Flammability hazards were not identified that could affect any of the new or existing safety-related structures based on a review of site plant developments with the placement of HAR 2 and HAR 3.

2.2.3.1.3 Toxic Chemicals

Potential sources of toxic chemicals that might reach the critical HAR facilities are those due to accidental releases from nearby mobile and stationary sources and from on-site storage facilities. Based on data given in Subsections 2.2.1 and 2.2.2, toxic chemical sources may be considered limited to road or rail transport carriers.

There are no industries in the area whose classification would be expected to result in the storage or consumption of products that are considered hazardous. There are no toxic gas release event hazards identified for the Harris site from hazardous chemicals that are outside the scope of the DCD identified in Table 6.4-201.

2.2.3.1.3.1 Releases of Toxic Materials Resulting From a Roadway Accident

As noted in Subsection 2.2.1, the closest major roadway located within the 8-km (5-mi.) radius of the plant site that may be used to transport hazardous materials is U.S. Highway 1. The number of toxic material truck shipments on this roadway that have the potential to affect the plant is not known. However, significant and frequent traffic of this nature is not considered likely, since none of the regional industries in the 8-km (5-mi.) zone use or store hazardous materials. Truck traffic for the transport of hazardous materials to support regional industries is expected to use the nearby interstate routings that are located approximately 16 km (10 mi.) from the plant site. Therefore, a hazardous material release due to a roadway accident is not considered to be a design-basis event.

2.2.3.1.3.2 Releases of Toxic Materials Resulting from a Railroad Accident

Three railroad segments come within 8 km (5 mi.) of the plant location. These segments are:

- The Bonsal – Durham segment, which is 4 km (2.5 mi.) northwest of the plant site.
- The Fuquay-Varina – Brickhaven segment, which is 6.9 km (4.3 mi.) south of the plant site.
- The Raleigh – Moncure segment, which is 3 km (1.9 mi.) northwest of the plant site.

Only the Raleigh – Moncure segment of railroad traffic within the vicinity of the HAR site carries hazardous materials on a regular basis.

A current breakdown of the hazardous materials transported on the rail segment near the plant site was evaluated. None of the hazardous materials evaluated are considered a threat to the HAR Control Room habitability based upon a review of the number of annual shipments, the material physical and chemical characteristics, toxicity, or the proximity of the rail segment (3 km [1.9 mi.]) to the location of Control Room intakes. Therefore, a railway hazardous chemical release is not considered to be a design-basis event.

To further demonstrate this conclusion, the annual probability of an accidental release of toxic chemicals resulting from a railroad accident in the vicinity of the plant site was determined. A conservative estimate of the likelihood of toxic chemical releases occurring near the plant includes consideration of releases from rail transport accidents and plant site meteorology. The probability of a toxic chemical release is calculated from the frequency of the rail transport accidents, conditional probabilities of accident severity, and the rail travel distance near the plant. The probability of adverse meteorological conditions is calculated from the annual frequencies of atmospheric stability class and the wind direction. Therefore, the probability of a toxic chemical release adversely affecting the plant is:

$$AP = P \times N \sum_i M_i \times \sum_j (D_j \times F_{ji})$$

For:

i = A, …, G stability classes, and
j = 1, …, n wind direction sectors affecting plant.

Where:

AP = Annual probability of a design-basis event from a hazardous chemical release from transport near the plant site.

P = Average annual probability of a railcar containing hazardous chemicals will have a severe accident per unit length of travel.

M_i = Annual probability of an atmospheric stability class.

D_j = Length of segment, in the direction of concern.

F_{ji} = Wind frequency from the sector j to outside air intake of the Control Room for stability class i.

N = number of unknown toxic chemical railcar loads ≤ 1.

From NUREG/CR-6624, the statistics on the number of severe events involving the release of hazardous materials per railcar mile per year is estimated to be on average about 1.4×10^{-7}. The total length of railway from which an airborne cloud of hazardous materials might be released from railroad segments in the direction of the HAR 2 or HAR 3 Control Room intake location is approximately 13.2 km (8.2 mi.). This distance is broken down into six segments from west to north-northeast of the plant site. Based upon the railway-segment lengths, atmospheric stability, and wind sector frequency data as given in FSAR Section 2.3, the annual probability of severe hazardous chemicals from railcar shipments reaching HAR 2 or HAR 3 is determined to be less than 8.6×10^{-8} per year.

The accidental release of the chemicals shipped on the rail line near the plant will not have toxic consequences that can adversely affect the plant's safety features. In addition, the annual probability of a significant accidental release from an unknown but toxic shipment on the line, is not greater than about 10^{-7} per year. Therefore, accidental releases from rail shipments of chemicals near HNP are not design-basis events that could lead to significant risks to the public.

2.2.3.1.4 Fires

The only fire hazard in the vicinity of the plant site is the potential delayed ignition of flammable vapor clouds associated with the potential for a rupture in the LPG pipeline that passes within approximately 2.5 km (1.6 mi.) of the plant site. This information is discussed in Subsection 2.2.3.1.2, Nearby Gas Pipeline.

2.2.3.1.5 Collision with the Intake Structure

This subsection is not applicable, as the plant site is not located on a navigable waterway.

2.2.3.1.6 Liquid Spills

No storage facilities for corrosive, cryogenic, or coagulant oil or liquids were identified. Therefore, these materials are not anticipated to be drawn into the intake structures or affect the plant's safe operation.

2.2.3.2 Effects of Design-Basis Events

There are no design-basis events identified in Subsection 2.2.3.1 that require mitigating actions to eliminate or lessen the likelihood and severity of potential accidents. The consequences of the events in Subsection 2.2.3.1 do not cause design-basis events that could result in significant risks to the public. In addition, the rate of occurrence of all transportation, industrial and military hazards near the plant causing design-basis events is approximately 10^{-7} per year, which is acceptably low.

STD DEP 1.1-1 2.2.4 COMBINED LICENSE INFORMATION FOR IDENTIFICATION OF SITE SPECIFIC POTENTIAL HAZARDS

HAR COL 2.2-1 This COL item is addressed in Subsection 2.2.

2.2.5 REFERENCES

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HAR COL 2.2-1

Table 2.2.1-201
Active Facilities in Chatham, Harnett, and Wake Counties

Mines and Quarries	County	Operating Status	Commodity
Brickhaven Mine No. 2 (Cherokee Sanford Group)	Chatham	Active	Brick Clay
Holly Springs Quarry (Hanson Aggregates Carolina)	Wake	Active	Crushed Stone
Merry Oaks Site #1 (Triangle Brick Company)	Chatham	Active	Brick Clay
Merry Oaks Site #2 (Triangle Brick Company)	Chatham	Active	Brick Clay

Source: Reference 2.2-202

HAR COL 2.2-1

Table 2.2.2-201
Industries within an 8-km (5-mi.) Radius of HAR Site

Company	Location	Description	Distance to HAR Site
B&B Companies of NC, Inc.	6991 Old U.S. Highway 1, New Hill, NC 27562	Heavy Truck Repair, Chip Hauling, Mulch Sales	7.45 km (4.63 mi.)
McGill Environmental	634 Christian Chapel Church Road, New Hill, NC 27562	Wood Chipping, Recycling Facilities	4.7 km (2.92 mi.)
Triangle Brick Company	294 Kings Road, Moncure, NC 27562	Brick Manufacturer	4.5 km (2.8 mi.)

Sources: Reference 2.2-213, Reference 2.2-214, and Reference 2.2-215

HAR COL 2.2-1

Table 2.2.2-202
Public Airports within 32 km (20 mi.) of HAR Site

Airport	Distance to Site	Operations	Length and Orientation of Runway	Types of Aircraft Using the Facility	Flying Patterns Associated with the Airport
Raleigh-Durham International Airport	31 km (19 mi.)	245,099 per year	1) 3048 m (10,000 ft.) Oriented north 2) 2286 m (7500 ft.) Oriented north 3) 1088 m (3570 ft.) Oriented east	Commercial, commuter, and general aviation aircraft	Straight in, left turn patterns, and right turn patterns
Sanford-Lee County Regional Airport	14.5 km (9 mi.)	47,085 per year	1981 m (6500 ft.)	Mainly small single-engine and some larger multi-engine aircraft	Standard left traffic
Triple W Airport	23.3 km (14.5 mi.)	21,535 per year	916 m (3004 ft.)	Strictly light aircraft	Standard left and right traffic

Sources: Reference 2.2-207, Reference 2.2-208, and Reference 2.2-218

HAR COL 2.2-1

Table 2.2.2-203
Aircraft Operations — Raleigh-Durham International Airport

Actual Year	Operations Per Year				
	Air Carrier	General Aviation	Air Taxi	Military	Total
1976	30,826	147,861	9365	9568	197,620
1977	33,608	152,229	11,462	9059	206,358
1978	34,145	154,476	13,153	7470	209,244
1979	39,929	146,203	14,889	6720	207,741
1980	40,225	130,079	24,382	7487	202,173
1985	55,648	111,138	31,299	10,609	208,694
1990	124,113	83,041	67,113	8683	282,950
1995	90,976	69,007	38,865	6041	204,889
2000	152,817	67,325	71,434	5103	296,679
2005	77,059	54,964	112,353	4135	248,511
2006	76,917	59,112	104,501	4569	245,099

Source: Reference 2.2-208

HAR COL 2.2-1

**Table 2.2.2-204 (Sheet 1 of 2)
Largest Companies in Chatham, Harnett, and Wake Counties
(Government/Public Employers Not Included)**

Company	Specialization	Employment
Chatham County		
1. Townsends	Poultry Processing	1375
2. GoldKist	Poultry Processing	800
3. Joann Fabrics	Upholstery Fabrics for Auto and Home	700
4. Performance Fibers	Fiber Manufacturer	525
5. Acme-McCrary Corporation	Women's Hosiery	365
6. Carolina Meadows	Health Services	250
7. Wal-Mart	Retail and Distribution	250
8. ATC Panels	Panel Manufacturer	220
9. Weyerhaeuser	Woodworking	175
10. Performance Bicycle Shop	Bicycle Manufacturer	100
Harnett County		
1. Food Lion Distribution Center	Grocery Warehouse and Shipping	760
2. Campbell University	University/Education	600
3. Betsy Johnson Hospital	Medical Center	550
4. Energy Conversion Systems	Carbon Brushes	485
5. Wal-Mart	Mass Retail	420
6. Edwards Brothers	Hard and Soft Bound Books	269
7. Good Hope Hospital	Medical Center	250
8. Machine & Welding Supply	Industrial Gases and Supplies	200
9. Champion Homes	Manufactured Mobile Homes	175
10. Godwin Mfg. Company	Truck Bodies, Hydraulics	170
Wake County		
1. WakeMed	Medical Center	6739
2. SAS Institute	Computer Software Developer	4143
3. Rex Healthcare	Medical Center	3870
4. Progress Energy	Utility Company	3400
5. Cisco Systems	Digital Switching Equipment	2850
6. Eaton Corporation	Integrated Power Systems	2600

HAR COL 2.2-1

Table 2.2.2-204 (Sheet 2 of 2)
Largest Companies in Chatham, Harnett, and Wake Counties
(Government/Public Employers Not Included)

Wake County		
Company	**Specialization**	**Employment**
7. Waste Industries	Waste Management	2000
8. Verizon Wireless	Telecommunication	1600
9. First Citizens Bank & Trust	Banking (Financial)	1574
10. Food Lion Stores	Grocery Distribution	1500
11. Longistics	Warehouse and Distribution	1500
12. Misys Healthcare Systems	Software Development and Marketing	1500

Source: Reference 2.2-212

HAR COL 2.2-1

Table 2.2.2-205
Industrial Investment in Chatham, Harnett, and Wake Counties and in the Research Triangle Region

Area	1999	2000	2001	2002	2003
Chatham County	$143,593,049	$5,500,000	$1,985,000	$15,177,554	$50,220,000
Harnett County	$9,500,000	$2,300,000	$6,100,000	$30,386,000	$34,300,000
Wake County	$401,480,000	$1,278,600,066	$138,280,000	$742,058,441	$204,003,000
Research Triangle Region	$1,109,454,357	$1,913,528,266	$1,160,511,129	$1,481,120,427	$856,662,500

Source: Reference 2.2-212

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 project site and the site itself, as described within this section, are bounded by the site parameters specified in DCD Table 2-1 and as compared in Section 2.0 of this FSAR.

2.3.1 REGIONAL CLIMATOLOGY

HAR COL 2.3-1 This subsection describes the general climate surrounding the proposed Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR). 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 proposed Shearon Harris Nuclear Power Plant Unit 2 (HAR 2) and the proposed Shearon Harris Nuclear Power Plant Unit 3 (HAR 3). 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 Charlotte, Greensboro, Raleigh-Durham, and Wilmington, North Carolina. Figure 2.3.1-201 shows the locations of these meteorological observation stations with respect to the HAR 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 HAR site is located near the geographical north central portion of North Carolina in the transition zone of the Coastal Plain and Piedmont regions. Four first-order meteorological observation stations are located within the general area surrounding the HAR site. The locations of these stations, which are all in North Carolina, and their distances from the HAR site are presented in Table 2.3.1-201. The Raleigh-Durham station is approximately 30 kilometers (km) (19 miles [mi.]) to the north-northeast of the HAR site; the Charlotte station is 188 km (117 mi.) to the west-southwest; the Greensboro station is 111 km (69 mi.) to the west-northwest; and the Wilmington station is 179 km (111 mi.) to the south-southeast 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 HAR site is located in the Central Piedmont state climate division of the NCDC (Reference 2.3-230).

Climatological data for the general area surrounding the 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:

- “Climates of the States,” Third Edition (Reference 2.3-201).
- “Weather of U.S. Cities,” Fourth Edition (Reference 2.3-202).
- “Local Climatological Data (LCD) Annual Summaries with Comparative Data” for Charlotte, Greensboro, Raleigh-Durham, and Wilmington, North Carolina, 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, and 2.3-206).

The topography of North Carolina is comprised of three physiographic divisions: the Coastal Plain, Piedmont, and Mountains. As illustrated on Figure 2.3.1-202, the Coastal Plain division is the largest of the State, comprising approximately half of the area of the state. The Coastal Plain division is subdivided into the tidewater area and the interior portion. The slope ranges from 61 meters (m) (200 feet [ft.]) at the fall line to approximately 15 m (50 ft.) in the tidewater area. The fall line represents the dividing line between the Coastal Plain and the Piedmont divisions. The Wilmington observation station is located in the tidewater section of the Coastal Plain region (Reference 2.3-206). The Piedmont represents about one-third of the area of North Carolina, and ranges in slope from 61 m (200 ft.) at the fall line to 457 m (1500 ft.) at the Mountains. The Piedmont is characterized primarily by gently rolling hills with some areas of steep hills. The HAR site is located in the transition zone between the Coastal Plain and Piedmont regions, as is the Raleigh-Durham observation station (Reference 2.3-205). The Charlotte and Greensboro stations are located in the Piedmont region (References 2.3-203 and 2.3-204).

The climatology of North Carolina is largely dependent on the elevation above sea level and the distance from the Atlantic Ocean. The climate of the Piedmont and Coastal Plain regions is typically humid, subtropical. With a humid, continental climate, the Appalachian Mountain Range in western North Carolina is typically much cooler than other areas in the state because of its higher elevation. Proximity to the Atlantic Ocean influences the winter weather in the eastern portion of the state by having a moderating effect on summer and winter temperatures. The Appalachian Mountains act as a barrier to cold polar air masses originating from the northwest, which tend to be stopped or deterred by the mountains. Deeper air masses are lifted by the mountains, resulting in a slight warming of the air and a loss of moisture during descent (References 2.3-203, 2.3-204, 2.3-205, and 2.3-206). During winter, cold air can wedge southward from the northeastern United States, east of the Appalachians, resulting in a relatively high frequency of freezing rain and sleet in the Piedmont. Summer weather is affected by tropical airstreams originating in the Gulf of Mexico. The higher temperatures and humidity of this tropical air affects the central and eastern portions of the state.

Table 2.3.1-202 presents a summary of historical climatological observations from the Charlotte, Greensboro, Raleigh-Durham, and Wilmington 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 LCD summaries for the cities in the area surrounding the HAR site indicate that thunderstorms have been observed on an average of 40.6 days per year in Charlotte (67-year period of record [POR]), 45.1 days per year in Greensboro (78-year POR), 44.0 days per year in Raleigh-Durham (61-year POR), and 47.5 days per year in Wilmington (54-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 four locations. Charlotte averaged 7 days of thunderstorms in both June and August and 9 days in July. Greensboro averaged 8 days in June and August and 10 days in July. Raleigh-Durham averaged 7 days, 11 days, and 8 days in June, July, and August, respectively. Wilmington averaged 8 days, 12 days, and 9 days in June, July, and August, respectively. Charlotte, Greensboro, and Raleigh-Durham averaged three or more thunderstorm days per month from April through September; Wilmington averaged three or more days per month from March through September. The Charlotte, Greensboro, and Raleigh-Durham stations each averaged two or less thunderstorm days per month from October through March, and the Wilmington station averaged two or less days from October through February. 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 2.3-207). Otherwise, special observations are recorded as a thunderstorm whenever thunder is heard.

A severe thunderstorm is defined in NOAA Technical Memorandum NWS SR-145, entitled “A Comprehensive Glossary of Weather Terms for Storm Spotters,” as a thunderstorm that possesses one or more of the following characteristics (Reference 2.3-208):

- Winds of 50 knots (58 miles per hour [mph]) or more.
- Hail 1.91 centimeters (cm) (0.75 inch [in.]) or more in diameter.
- Thunderstorms that produce tornadoes.

Severe thunderstorms producing hail events with hail greater than 1.91 cm (0.75 in.) or more in diameter were recorded during the period from 1950 to 2006. A total of 182 hail events were reported in Wake County, North Carolina, during the period from January 1, 1950 to July 31, 2006. Only one storm resulted in reported property damage (Reference 2.3-209). It is noted that there has been

a significant increase in the reported number of hail events over time, primarily as a result of increased reporting efficiency and confirmation skill and the fact that many storms may have been overlooked in the early data collection years. 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 182 hailstorms were reported in Wake County over the period 1950 to 2006, 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 2.3-210):

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

where

N = Number of flashes to earth per thunderstorm day per km^2

θ = Geographical latitude

For the HAR site, which is located at 35.64° north latitude, the frequency of lightning flashes (N), is predicted to range from 0.061 to 0.182 flashes per thunderstorm day per km^2. The value 0.182 is used as the most conservative estimate of lightning frequency in the calculations that follow.

The average annual number of thunderstorm days in the area (i.e., as reported at the Charlotte, Greensboro, Raleigh-Durham, and Wilmington observation stations) is 44.3. This results in a predicted mean frequency of 8.1 lightning flashes per km^2 per year, as calculated below:

$$\frac{0.182\,flashes}{(thunderstorm-day)(km^2)} \times \frac{44.3\,thunderstorm-days}{year} = \frac{8.1\,flashes}{(km^2)(year)}$$

The total owned area of the HAR site is approximately 4371 hectares (ha) (10,800 acres [ac.]). Hence, the predicted frequency of lightning flashes within the area of the property owned by the existing Shearon Harris Nuclear Power Plant Unit 1 (HNP) is 354 per year, as calculated below:

$$\frac{8.1\,flashes}{(km^2)(year)} \times 43.7\,km^2 = \frac{354\,flashes}{(year)}$$

The exclusion area for HAR 2 and HAR 3 is a radius of 1245 m (4085 ft.) around each unit. This is considered as the approximate operational area of the HAR. The predicted frequency of lightning flashes in the HAR exclusion area of a single reactor can be calculated as follows:

$$\frac{8.1\,flashes}{(km^2)(year)} \times 4.9km^2 = \frac{40\,flashes}{(year)}$$

Therefore, the predicted number of lightning flashes in the immediate vicinity of HAR 2 and HAR 3 is predicted to be 40 per year.

2.3.1.2.2 Tornadoes and Severe Winds

North Carolina ranks 20th in the United States in average annual number of tornadoes, based on a 52-year POR from 1953 to 2004 (Reference 2.3-211). Table 2.3.1-203 summarizes, by tornado intensity, all tornadoes reported in North Carolina during the period January 1, 1950 to July 31, 2006 (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 North Carolina were as follows:

- 372 (F0)
- 419 (F1)
- 172 (F2)
- 45 (F3)
- 27 (F4)
- 0 (F5)

These totals equate to an average of seven F0, seven F1, three F2, less than one F3, less than one F4, and zero F5 tornadoes reported in North Carolina per year.

During the same period (1950 to 2006), a total of 28 tornadoes were reported in Wake County. The number of reported tornadoes for Wake County and seven adjacent counties surrounding the HAR site are summarized in Table 2.3.1-204 using the original Fujita scale. A total of 83 tornadoes were reported during the POR for the eight counties (Wake, Alamance, Chatham, Durham, Harnett, Johnston, Lee, and Orange) surrounding the HAR site (Reference 2.3-212). The largest reported tornado, an F4, occurred in November 1988 in Wake and Nash

counties. This tornado resulted in four fatalities and approximately $250 to $285 million in damage. Table 2.3.1-205 summarizes the number of tornadoes in North Carolina by year and the (original) Fujita Tornado Scale Category for the period 1950 to 2006.

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 HAR 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 1.46 for the eight-county area containing and surrounding the HAR site, as calculated from Table 2.3.1-204).

a = Average individual tornado area, equal to 1.093 km^2 (0.422 mi.^{2}) for the HAR site. This calculation weights the expected individual tornado area from NUREG/CR-4461 Rev. 2, Table 2-14 by the tornado occurrences in Table 2.3.1-204.

A = Total area of concern (i.e., eight-county area equal to 11,283 km^2 (4356.5 mi.^{2}).

Using this equation, the tornado strike probability (for a tornado of any intensity) for the HAR site, Ps, is estimated to be 1.41E-4 per year, which corresponds to a return frequency of once in 7071 years.

Waterspouts, which are similar to tornadoes, have been observed to occur only over very large bodies of water, such as the ocean, the Great Lakes, the Great Salt Lake, and other similar sized large bodies of water, and are not expected to occur in the vicinity of the HAR site.

Design-basis tornado parameters have traditionally been based on 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. 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 (ALWRs) 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.

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 recent draft guidance, as published in NRC Draft Guide 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 AP1000 meet these criteria, as noted in subsection 3.3.2.1 of Westinghouse Electric Company, LLC's (Westinghouse's) "AP1000 Design Control Document" (AP1000 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, 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 RG 1.76 guidance is as follows:

- Rotational velocity = 82 m/sec (184 mph).
- Maximum translational velocity = 21 m/sec (46 mph).
- Maximum wind speed = 103 m/sec (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/sec (0.5 psi/sec).

These parameters are NRC's published design-basis tornado parameters for the region surrounding the HAR site. They are less stringent than the proposed design criteria for the AP1000 units that will be used for HAR 2 and HAR 3. However, since the maximum site characteristics for wind speed and pressure drop associated with the guidance in NRC Draft Regulatory Guide DG-1143 are higher than those in Regulatory Guide 1.76, Revision 1, those values will be used as the maximum site characteristics for comparison with the DCD site parameters in FSAR Table 2.0-201.

Peak observed wind speeds at the Charlotte, Greensboro, Raleigh-Durham, and Wilmington stations were previously identified in Table 2.3.1-202. As indicated in the table, the peak observed wind speeds at those four stations were 140 km/h (87 mph), 156 km/h (97 mph), 100 km/h (62 mph), and 126 km/h (78 mph), respectively. An importance factor of 1.15 is applied to this wind speed in the design of safety-related structures (Reference 2.3-216). Therefore, the maximum sustained wind speeds based on regional observations would be 161 km/h (100 mph), 180 km/h (112 mph), 114 km/h (71 mph), and 145 km/h (90 mph) for Charlotte, Greensboro, Raleigh-Durham, and Wilmington, respectively. These values are substantially less than NRC's design-basis tornado wind speeds for the region.

In addition to the maximum observed wind speeds in the region, a site characteristic 3-second gust wind speed that represents a 100-year return period for the HAR site has been established at 154 km/h (96 mph). The 3-second gust wind speed is based on the Structural Engineering Institute/American Society of Civil Engineers (SEI/ASCE) 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) CD published by NOAA for the Charlotte, Greensboro, and Raleigh-Durham weather stations (Reference 2.3-217). The published 3-second gust wind speed for these stations is 145 km/h (90 mph); this is represented as the nominal design 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 the nominal design 3-second gust wind speed of 154 km/h (96 mph).

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. Research conducted by Fuhrmann and Konrad of the Department of Geography at the University of North Carolina at Chapel Hill provides information on winter weather events observed at 18 first-order weather stations during the period from 1948 to 2003 (Reference 2.3-218). The North Carolina State Climate Office (SCO) reports that winter weather precipitation typically occurs in the state as a result of cold continental polar air masses from Canada mixing with moist air originating over the Gulf of Mexico. The moist air may be displaced by a cold dome that is formed when air masses from the New England area entrain the polar air masses from Canada, creating a wedge of cold air near the earth's surface. The moist air can migrate upward over the cold dome, resulting in mixed precipitation.

Annual precipitation distributions and mean recurrence intervals were determined for the State of North Carolina. According to the SCO, frozen precipitation totals for the Piedmont region of North Carolina are between 2.54 cm (1 in.) and 5.08 cm (2 in.) liquid equivalent per year, with liquid equivalent snowfall between 1.27 cm (0.5 in.) and 3.81 cm (1.5 in.) per year, sleet is at least 0.25 cm (0.1 in.)

per year, freezing rain averages 1.52 cm (0.6 in.), and 7.62 cm (3 in.) to 9.14 cm (3.6 in.) of cold rain per year.

Heavy freezing rain events have occurred in and across portions of the Piedmont of North Carolina, including a December 2002 event where 3.61 cm (1.42 in.) of freezing rain was recorded at Raleigh-Durham (Reference 2.3-218). Mean annual occurrences of measurable winter weather precipitation in Raleigh-Durham are 1.27 cm (0.5 in.), 3.30 cm (1.3 in.), and 1.78 cm (0.7 in.) for freezing rain, sleet, and snowfall, respectively. The probability of occurrence of measurable precipitation in Raleigh-Durham is 100 percent, 77 percent, and 100 percent for freezing rain, sleet, and snowfall, respectively (Reference 2.3-218). Although there have been some events that have caused traffic problems (such as the January 2005 snow event), there have been no significant impacts on HNP operations as a result of these types of events.

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. The normal winter precipitation event was determined by taking the highest ground-level weight (in pounds per square foot [psf]) associated with the projected 100-year return period snowpack, the historical maximum snowpack, the projected 100-year return period snowfall event, or the historical maximum snowfall event in the site region.

The 100-year return period snowpack was based on the published 50-year recurrence Ground Snow Load for Charlotte, Greensboro, Raleigh-Durham, and Wilmington using Figure 7-1, “Ground Snow Loads, p_g, for the United States (lb/ft^2),” from American Society of Civil Engineers (ASCE) 7-05, “Minimum Design Loads for Buildings and Other Structures” (Reference 2.3-216). The 100-year return period snowpack was estimated to be 48.8 kilograms per square meter (kg/m^2) (10 pounds per square foot [psf]), 73.2 kg/m^2 (15 psf), 73.2 kg/m^2 (15 psf), and 48.8 kg/m^2 (10 psf) for Charlotte, Greensboro, Raleigh-Durham, and Wilmington, respectively. (Reference 2.3-216). Using the conversion factor of 1.22, the 100-year recurrent ground snow load is calculated to be 59.6 kg/m^2 (12.2 psf), 89.3 kg/m^2 (18.3 psf), 89.3 kg/m^2 (18.3 psf), and 59.6 kg/m^2 (12.2 psf) for Charlotte, Greensboro, Raleigh-Durham, and Wilmington, respectively.

The historical maximum snowfall event and the projected 100-year return period snowfall event were obtained from the National Climatic Data Center’s (NCDC’s) Snow Climatology website (Reference 2.3-230) and converted to ground snow loads using guidance provided in DC/COL ISG-07. The historical maximum snowfall event for Charlotte, Greensboro, Raleigh-Durham, and Wilmington was 48.3 kg/m^2 (9.9 psf), 90.8 kg/m^2 (18.6 psf), 77.1 kg/m^2 (15.8 psf), and 40.5 kg/m^2 (8.3 psf), respectively. The projected 100-year return period snowfall event for Charlotte, Greensboro, Raleigh-Durham, and Wilmington was 63.0 kg/m^2 (12.9

psf), 66.4 kg/m^2 (13.6 psf), 69.8 kg/m^2 (14.3 psf), and 49.8 kg/m^2 (10.2 psf), respectively.

The historical maximum snowpack for Charlotte, Greensboro, Raleigh-Durham, and Wilmington was 44.4 kg/m^2 (9.1 psf), 61.0 kg/m^2 (12.5 psf), 68.8 kg/m^2 (14.1 psf), and 29.3 kg/m^2 (6.0 psf), respectively. The site characteristic ground snow load is selected as the greater of the above values or 18.6 psf (Greensboro, NC).

ISG DC/COL ISG-07 presents the methodology for converting the normal and extreme winter precipitation events into their resulting normal and extreme winter precipitation roof loads. Normal winter precipitation roof load was determined using the ASCE 7-05 standard for converting ground snow from a normal winter precipitation event to snow load on the roof (Reference 2.3-216). The resulting normal winter precipitation roof load is 20.5 psf. Extreme winter precipitation event roof load is based on the roof load due to the normal winter precipitation event plus any accumulation of water associated with an additional antecedent extreme winter precipitation event. While the design of the AP1000 structures may allow for the accumulation of snowfall on the roofs, the potential for additional accumulation of precipitation from rainwater ponding during an extreme winter PMP rainfall event is not considered to be significant, due to the design of the AP1000 structures. Therefore, no significant additional roof loadings are assumed to occur beyond the weight of pre-existing snowpack.

2.3.1.2.4 Hurricanes

Hurricanes have been observed in coastal and inland areas of North Carolina. While sustained hurricane force winds (greater than 119 km/h [74 mph]) have not been recorded at the Raleigh-Durham weather station, climatological and storm-event records indicate that a number of hurricane tracks have passed within 100 nautical miles of the HAR site. 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 HAR site is located approximately 225.3 km (140 mi.) inland from the Atlantic coast. The major effect from hurricanes on the area is heavy precipitation.

The State Climate Office of North Carolina reports that 48 reported hurricanes and tropical storms have made direct landfall in North Carolina during the period 1857 to 2005, which corresponds to an annual average frequency of occurrence of 0.32 storms per year (Reference 2.3-233). The NOAA Coastal Services Center reports that only four hurricanes rated Category 2-5 have passed within 50 nautical miles of Wake County and that only 11 hurricanes rated Category 2-5 have passed within 100 nautical miles of Wake County during the same period (Reference 2.3-234).

2.3.1.2.5 Normal Operating Heat Sink Design Parameters

Two natural draft cooling towers will be used to provide a heat sink during normal operation of HAR Units 2 and 3. The AP1000 reactor does not rely on site service water as a safety grade ultimate heat sink (UHS); therefore, this section establishes the meteorological design parameters for the natural draft cooling towers during normal operation, including any extreme meteorological conditions that could be encountered during operation of the plant. The controlling meteorological parameters for a natural draft cooling tower based system are wet and dry bulb temperatures. Table 2.3.1-206 provides a summary of statistically significant dry- and wet-bulb temperatures that are used to define the design temperatures at the HAR site, as obtained from the Charlotte, Greensboro, and Raleigh-Durham meteorological observing stations. These data were obtained from the 30-year (1961–1990) Solar and Meteorological Surface Observation Network (SAMSON) database (Reference 2.3-220).

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 2.3.1-206, the site characteristic maximum 30-day running average wet-bulb temperatures are 22.8 degrees Celsius (°C) (73 degrees Fahrenheit [°F]), 23.1°C (73.6°F), and 23.5°C (74.3°F), respectively, for the Charlotte, Greensboro, and Raleigh-Durham meteorological observing stations. The coincident 30-day average dry-bulb temperatures for the same period are 27°C (80.6°F), 26.7°C (80.1°F), and 26.3°C (79.3°F), respectively.

As also discussed in NRC’s Regulatory Guide 1.27, the meteorological conditions resulting in minimal water cooling would be the worst 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 temperatures for the same period. Based on an evaluation of the historical meteorological data presented in Table 2.3.1-206, the site characteristic maximum 5-day running average temperatures for the 30-year period from 1961 to 1990 are 23.9°C (75°F), 24.6°C (76.3°F), and 24.9°C (76.8°F), for the Charlotte, Greensboro, and Raleigh-Durham meteorological observing stations, respectively. The coincident 5-day running average temperatures are 28.8°C (83.8°F), 28.4°C (83.1°F), and 29.3°C (84.7°F), respectively. The site characteristic maximum 1-day running average wet-bulb temperatures are 24.9°C (76.8°F), 25.4°C (77.7°F), and 26°C (78.8°F), respectively, and the coincident 1-day running average dry-bulb temperatures for the same period are 29.2°C (84.6°F), 29.6°C (85.3°F), and 30.7°C (87.3°F), respectively.

The site characteristic wet-bulb temperatures that were exceeded less than 1 percent of the time were 24.4°C (76°F), 24.4°C (76°F), and 25°C (77°F) for the Charlotte, Greensboro, and Raleigh-Durham stations, respectively. The maximum wet-bulb temperatures recorded for Charlotte, Greensboro, and

Raleigh-Durham during this period were 27.2°C (81°F), 27.2°C (81°F), and 28.3°C (83°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 will be considered in designing the natural draft cooling towers to minimize any recirculation of air and vapor exiting the towers and to provide adequate cooling capacity should any recirculation occur.

2.3.1.2.6 Inversions and High Air Pollution Potential

Weather records from many United States weather stations have been analyzed by Hosler (Reference 2.3-221), Holzworth (Reference 2.3-222), and Holzworth (Reference 2.3-223) with the objective of characterizing atmospheric dispersion potential. The expected seasonal frequencies of inversions based below 152 m (500 ft.) for Greensboro, North Carolina, which is 103 km (69 mi.) to the west-northwest of the HAR site, are shown in Table 2.3.1-207. 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 2.3.1-207, the inversion frequency in Greensboro averaged 33 percent in summer season and 43 percent in winter season (Reference 2.3-221).

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 Greensboro are shown in Table 2.3.1-208. The lowest mean monthly mixing depth occurs in January (390 m [1280 ft.]) and the greatest mean mixing depth occurs in June (1790 m [5873 ft.]) (Reference 2.3-222).

The HAR site is located in Wake County, which is currently designated by the U.S. Environmental Protection Agency (USEPA) as a maintenance area for the ozone standard (8-hour) and carbon monoxide (CO) standards (1- and 8-hour), is an attainment area for all other pollutants and air quality standards (Reference 2.3-224). The North Carolina Department of Environment and Natural Resources (NCDENR) operates a network of ambient air quality monitoring stations throughout the state. The NCDENR separates the state into seven regions. The HAR site is located in the Raleigh region, which includes 13 monitoring locations. Three of the monitoring stations are located within Wake County. These stations monitor for various National Ambient Air Quality Standards (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 CO) (References 2.3-225 and 2.3-226).

Although Wake County is currently designated by USEPA and NCDENR to be a maintenance area for ozone, it is noted that the operation of the HNP facility (including the proposed HAR units) should not result in an increase in ozone levels at any location, since there will be no significant emissions of any ozone forming pollutants from the facility.

2.3.1.2.7 Ambient Air Temperatures

A summary of the ambient air temperatures at the major meteorological observing stations surrounding the HAR site (i.e., Charlotte/Douglas, Greensboro, and Raleigh-Durham) is provided in Table 2.3.1-209 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 for Raleigh-Durham in Table 2.3.1-209 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 2.3-220), and the ASHRAE fundamentals handbook (Reference 2.3-227). 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 pair for Raleigh-Durham (the closest and most representative station) is shown in Table 2.3.1-209 to be 106.6/73.6°F. When compared with the DCD site parameter data pair of 115/86.1°F, these values are seen to be bounded by the DCD site parameter and well below the Maximum Safety DCD limits. Although not calculated for the Charlotte/Douglas and Greensboro stations, a comparison of other temperature measurements in Table 2.3.1-209 from those stations with the Raleigh-Durham data indicates that estimated Maximum Safety values at those stations would also be well below the DCD Maximum Safety limit.

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 for Raleigh-Durham (Table 2.3.1-209) is 83.5°F and is well below the DCD site parameter value of 86.1°F.

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 2.3.1-209, 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 94°F dry bulb/75°F wet bulb (Charlotte).

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 2.3.1-209 is 77°F (Raleigh-Durham).

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 2.3.1-209 are well above the DCD site parameter, with the lowest estimated Minimum Safety dry bulb temperature being only -11°F (Greensboro).

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 2.3.1-209, which are based on 99.6 percent annual exceedance temperatures, are well above the DCD site

parameter of -10°F dry bulb. The lowest Minimum Normal dry bulb temperature shown in the table is only 14°F (Greensboro).

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, since 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, since 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 section of the Final Safety Analysis Report (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 plant.

2.3.2 LOCAL METEOROLOGY

HAR COL 2.3-2 Local meteorological conditions are characterized by data obtained from an on-site meteorological monitoring system that was installed and began operation at the HNP facility in March 1973. The on-site tower is located approximately 1.8 km (1.1 mi.) to the northeast of the HNP and consists of a 61.4-m (201.4-ft.) guyed, open-latticed design. The base of the tower is at approximately plant grade elevation of 79.2 m (260 ft.) above mean sea level (msl). The system datalogger and remote access instrumentation used to interrogate the system are housed in an environmentally controlled shelter located approximately 12 m (40 ft.) to the northwest of the tower. Based on the meteorological tower's proximity to the HAR site, the meteorological parameters that are monitored by the HNP monitoring station are considered to be representative of the HAR site and are therefore appropriate for use in characterizing local meteorological conditions. 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 Subsection 2.3.3.

The POR used to characterize the local meteorological conditions representative of the HAR site is the 5-year period from March 1, 1994 to February 28, 1999. The data from this period were determined to be the most recent contiguous 5-year period of data representative of the HAR site.

During the analysis and evaluation of the data available from the on-site monitoring station (i.e., data are available from January 14, 1976, to present), it was noted that, beginning in March 1999, there was a sharp increase in the observed frequency of calm winds for the lower-level wind measurement system. Subsequent analysis of the data indicated that the most probable cause of this increase was a wind speed monitor with an abnormally high threshold starting wind speed. The instrument was replaced in October 1999 during routine instrument maintenance, and the calm wind bias attributable to that instrument was no longer evident in the data after that date. However, an additional calm wind bias was noted beginning in August 2001 and extending until the time of discovery in late 2006. After analyzing the data and reviewing the monitoring and data logging instrumentation, it was found that, in August 2001, a new datalogger was installed as part of a system re-instrumentation program. The new datalogger was determined to have been programmed to record wind speeds using a “vector average” technique, with an instrument sampling rate of approximately once every second. Previous dataloggers used at the site (from 1976 to 2001) were programmed to record wind speeds using a “scalar average” technique, with a typical instrument sampling rate of once every 10 seconds. The vector averaging technique results in significantly higher observations of calm winds than are observed using scalar averaging methods. Although neither vector averaging nor scalar averaging techniques are considered to be technically incorrect, additional instrumentation was installed on the tower in November 2006. This instrumentation will facilitate a comparison of the vector and scalar average wind speeds. The program of dual measurements is currently underway. However, to avoid any bias in the analyses obtained using the on-site meteorological data, particularly in the short- and long-term diffusion estimates described in Subsections 2.3.4 and 2.3.5, the most recent contiguous 5-year POR using scalar averaging (i.e., March 1, 1994 through February 28, 1999) was selected for use. These data exceed NRC’s requirements (as described in Regulatory Guide 1.23, Revision 1) for a minimum 24-month POR. While a portion of the data period exceeds the 10-year guidance, more than 24 months of the data are compliant. PEC, therefore, considers the use of the 1994 to 1999 data to support of the COLA is justified.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

2.3.2.1.1 Wind Summaries

Detailed wind records are available from the HNP meteorological monitoring system for the POR from 1976 to 2006. For the purposes of this subsection, wind summaries for the POR from March 1, 1994, to February 28, 1999, were used, as described in Subsection 2.3.2. Monthly, annual, and 5-year average joint frequency distribution of wind speed and direction by Pasquill Stability Category

were constructed from wind speed and direction measurements made at the 12-m (39-ft.) and 61-m (200-ft.) levels of the on-site meteorological tower. It is noted that the measurement levels on the HNP meteorological tower are slightly different than the 10-m (33-ft.) and 60-m (197-ft.) levels recommended in Regulatory Guide 1.23, Revision 1. However, PEC believes that the measurement levels for wind speed and direction and vertical temperature difference, at 12 and 61 m, are consistent with the guidance of "approximately" 10 and 60 meters in Regulatory Guide 1.23, Revision 1 and, therefore, their continued use is justified. It is also noted that the six wind speed categories presented in the joint frequency distributions differ from the 11 wind speed categories recommended in Regulatory Guide 1.23, Revision 1.

The lower-level (12-m [39-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 1994 to 1999 period. Lower-level (12-m; 39-ft.) wind speed and wind direction data were also summarized for the "All Stability" category for each year from 1994 through 1999, as shown in Tables 2.3.2-209, 2.3.2-210, 2.3.2-211, 2.3.2-212, and 2.3.2-213. The percent occurrence of wind speed and wind direction has been summarized for the "All Stability" category for the period 1994 to 1999, as shown in Table 2.3.2-214. Additionally, the lower-level (12-m; 39-ft.) wind direction and wind speed are summarized monthly for the period March 1994 to February 1999 for the "All Stability" category in Tables 2.3.2-215, 2.3.2-216, 2.3.2-217, 2.3.2-218, 2.3.2-219, 2.3.2-220, 2.3.2-221, 2.3.2-222, 2.3.2-223, 2.3.2-224, 2.3.2-225, and 2.3.2-226. For this same period, graphical illustrations of the wind roses of wind speed and direction for the lower-level tower measurements (12-m [39-ft.]) are shown in Figure 2.3.2-201 (all stabilities, all 5 years) and in 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, all 5 years, by month).

The upper-level (61-m [200-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 2.3.2-227, 2.3.2-228, 2.3.2-229, 2.3.2-230, 2.3.2-231, 2.3.2-232, 2.3.2-233, and 2.3.2-234 for the 1994 to 1999 period. Upper-level (61-m; 200-ft.) wind speed and wind direction data were also summarized for the "All Stability" category for each year from 1994 through 1999, as shown in Tables 2.3.2-235, 2.3.2-236, 2.3.2-237, 2.3.2-238, and 2.3.2-239. The percent occurrence of wind speed and wind direction is summarized for the "All Stability" category for the period 1994 to 1999, as shown in Table 2.3.2-240. Additionally, the upper-level (61-m; 200-ft.) wind direction and wind speed are summarized monthly for the March 1994 to February 1999 period for the "All Stability" category in Tables 2.3.2-241, 2.3.2-242, 2.3.2-243, 2.3.2-244, 2.3.2-245, 2.3.2-246, 2.3.2-247, 2.3.2-248, 2.3.2-249, 2.3.2-250, 2.3.2-251, and 2.3.2-252.

The wind summaries prepared from the March 1, 1994, to February 28, 1999, wind data set were compared to the wind summaries provided in the HNP FSAR. A comparison of the composite wind data for the period 1994 – 1999 indicates that they are generally consistent with the observations of wind speed and direction that were provided in the HNP FSAR report for the period January 14, 1976, to December 31, 1978.

Graphical wind roses of wind speed and direction from the nearby Raleigh-Durham Airport are also provided for comparison with the on-site wind measurements described above. Figure 2.3.2-214 illustrates the wind rose for the 5-year period from March 1, 1984 through February 28, 1989, and Figure 2.3.2-215 illustrates the wind rose for the 5-year period from January 1, 2001 through December 31, 2005. It is noted that the Raleigh-Durham wind roses for the two 5-year periods are somewhat different, with an apparent difference in predominant wind direction on the order of 45 degrees. While there is no definitive explanation for the differences, it is possible that they could be attributable to local development, vegetation growth, or other factors. Differences between the Raleigh-Durham wind roses and the wind roses for the HNP site are more than likely attributable to differences in topography and vegetation between the Raleigh-Durham airport and the HNP site area.

2.3.2.1.2 Ambient Temperature

Ambient temperature data from the HNP site have been available from the on-site meteorological monitoring station since 1976. Temperature is measured at both the 12-m (39-ft.) and 61-m (200-ft.) levels, and differential temperature (used in determining wind stability classification) is measured between the 12-m (39-ft.) and 61-m (200-ft.) levels of the tower. For the 1994 to 1999 POR, the absolute maximum temperature recorded by the system was 36.4°C (97.5°F), and the absolute minimum temperature was -16.5°C (2.3°F). Information reported in the HNP FSAR, from the 1976 to 1978 POR, included composite monthly summaries of on-site ambient temperature measured at the 12-m (39-ft.) level, as well as similar information from the 1994 to 1999 data period. This information is presented in Table 2.3.2-253 (mean monthly and annual mean temperature). Table 2.3.2-254 contains mean monthly and annual maximum and minimum temperatures for the 1976 to 1978 data period. The diurnal temperature range for the site during this period is approximately 20 degrees in the winter and summer seasons and approximately 25 degrees in the fall or spring seasons, as reported in the HNP FSAR. The 8 years of on-site ambient temperature data reported in Table 2.3.2-253, are generally representative of the HAR site and consistently within the bounds of the long-term regional observations from Charlotte, Greensboro, and Raleigh-Durham when compared to long-term periods of record at those locations. Mean maximum and minimum daily temperatures for the Charlotte, Greensboro, and Raleigh-Durham stations are summarized in Table 2.3.2-255 (References 2.3-203, 2.3-204, and 2.3-205).

2.3.2.1.3 Not Used.

2.3.2.1.4 Atmospheric Moisture

2.3.2.1.4.1 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 3-year period from 1976 to 1978 were compared with regional observations from the Charlotte, Greensboro, and Raleigh-Durham stations. Table 2.3.2-256 compares mean dew-point measurements reported for the Charlotte, Greensboro, and Raleigh-Durham observing stations with measurements from the HNP on-site meteorological monitoring station for the 3-year period. Table 2.3.2-257 provides a summary of monthly and annual mean dew-point measurements from the on-site meteorological data for the period 1976 to 1978.

2.3.2.1.4.2 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, on average. Table 2.3.2-258 summarizes relative humidity observations from the Charlotte, Greensboro, and Raleigh-Durham meteorological observing stations.

2.3.2.1.5 Precipitation

The average yearly precipitation observed at the HNP meteorological monitoring station during the period from 1976 to 1978 was 89.92 cm (35.4 in.). Table 2.3.2-259 compares monthly and annual precipitation measurements at the Charlotte, Greensboro, and Raleigh-Durham meteorological observation stations with the monthly and annual average measurements from the HNP on-site meteorological monitoring station. On-site precipitation totals (monthly and annual) are summarized in Table 2.3.2-260. As shown in Table 2.3.2-259 and Table 2.3.2-260, the region displays some variance in total monthly and annual precipitation between stations from month-to-month and year-to-year, but there does not appear to be a well-defined “wet” or “dry” season.

The 8 years of on-site precipitation data reported in Table 2.3.2-260 are consistently within the bounds of the long-term regional observations from Charlotte, Greensboro, Raleigh-Durham and Wilmington when compared with long-term periods of record at those locations.

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 2.3-228).

Table 2.3.2-261 summarizes the occurrence of fog at the Charlotte, Greensboro, and Raleigh-Durham meteorological observation stations. Heavy fog (i.e., visibility less than or equal to 0.4 km [0.25 mi.]) has been observed at Charlotte, Greensboro, and Raleigh-Durham an average of 25.2, 32.4, and 32.5 days per year, respectively (References 2.3-203, 2.3-204, and 2.3-205). The greatest number of fog days typically occurs in the fall and winter, with approximately 3 days per month in November through February. However, fog can be a very localized phenomenon, and the information provided in Table 2.3.2-261 is used as a regional estimate for fog occurrence. The most common type of fog occurring near the HAR site is believed to be ground fog that results from nighttime radiational cooling. 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.

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 HAR 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. This is in accordance with NRC's Regulatory Guide 1.23, Revision 1. Joint frequency distributions of wind speed, wind direction, and atmospheric stability measured at the site for the period March 1, 1994, to February 28, 1999 are provided in a series of 52 tables, beginning with Table 2.3.2-201 and ending with Table 2.3.2-252.

Table 2.3.2-262 shows the frequency of occurrence of Pasquill stability categories for the periods 1976 to 1978 and March 1, 1994 to February 28, 1999. Based on the information presented, temporal variations within the individual stability categories are seen to be relatively small. Almost 50 percent of all hours fall into either neutral (D) or slightly stable (E) stability categories. Nearly 20 percent of all hours fall into the extremely stable (G) stability category. Extremely unstable (A), moderately unstable (B), and slightly unstable (C) categories combined occurred during only approximately 16 percent of the total hours. An assessment of the distribution of stability categories for the period from 1994 to 1999 would be expected to yield a distribution similar to that reported for the 1976 to 1978 data period. 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 HAR facility has the potential to influence the local micrometeorology in the immediate vicinity of the HAR site. These effects could occur as a result of minor changes to the topography resulting from the construction of additional buildings and supporting infrastructure, as well as the use of two additional natural draft cooling towers for system heat rejection to the atmosphere. Changes to the local topography are not expected to have a significant effect on diffusion characteristics in the area except in the immediate vicinity of the buildings. The use of natural draft cooling towers for system heat rejection will result in visible moisture plumes from the cooling towers during certain atmospheric conditions. The combination of visible plumes from each of the natural draft cooling towers (i.e., the HNP cooling tower and the proposed HAR 2 and HAR 3 cooling towers) will need to be considered; there is the potential for the mixed plume to be larger than a single visible plume. The amount of condensation of evaporated water vapor, and thus the formation of a visible plume, will be greatest during winter months when ambient air temperatures are cool and the air is moist.

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. Based on operational experience at the HNP, these types of conditions are observed only on a very limited basis at on-site locations (for example, on HAR or HNP plant property). During very cold, dry, and stable atmospheric conditions, the potential for fogging and icing conditions at off-site locations exists, but that potential is expected to be very small.

2.3.2.2.1 Topographical Description

The HAR 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. The plant lies within a shallow basin, as depicted in Figures 2.3.2-216, 2.3.2-217, 2.3.2-218, and 2.3.2-219, which show cross-sectional plots of elevation versus distance from the HNP plant center for each of 16 directional sectors. The general elevations within 16 km (10 mi.) of the site gradually increase from the plant grade of 79.2 m (260 ft.) msl to around 121.9 m (400 ft.) msl in all but the north, north-northwest, north-northeast, west-southwest, and southwest directions.

Figure 2.3.2-220 shows the topographic features within an 8-km (5-mi.) radius of the plant. The figure shows the topography as it will be influenced by the plant. The proposed increase in elevation of the Main Reservoir will result in a

significant increase in the surface area of the reservoir as discussed in Section 2.4.

The increase in surface area of the reservoir, and the corresponding increase in heat rejection to the reservoir attributable to operation of HAR 2 and HAR 3, could result in a slight increase in heat and moisture to the immediate vicinity of the plant. Under some conditions, the surface of the reservoir could be warmer than the surrounding ground and air. This increase in surface temperature could cause the layer of air over Harris Reservoir to achieve a neutral or unstable lapse rate in the vertical, especially when thermally stable conditions prevail over land. Under these conditions, a release from a ground-level source would undergo some additional vertical diffusion compared to what would be computed using a stability category obtained from the meteorological tower. However, because of the relatively small size of Harris Reservoir and its orientation with respect to HAR 2 and HAR 3, no adjustments are proposed to account for any additional surface heating effects. In general, the increase in surface heating effects is expected to be minimal.

There may also be periods when the average temperature of the water in the reservoir is colder than the surrounding land and air. It is expected that such periods could occur during or after synoptic changes in weather patterns or air masses that result in the flow of warm air into the region when the reservoir water is still cool. These occurrences are most likely to occur in the spring when synoptic scale air masses from the Gulf of Mexico or the Atlantic move into the area. Under these circumstances, there could be some short-term effects on local dispersion due to the presence of a stable layer of air over the reservoir. The most likely effects on dispersion would be a decrease in thermally-induced vertical mixing in the near-surface stable layer over the reservoir. However, these effects are not expected to persist for any significant distance from the reservoir for two reasons: 1) the reservoir consists of a number of smaller reaches that extend in different directions, rather than one large mass of water, and 2) the reservoir is surrounded by vegetative and forested areas with surface roughness features that will generate mechanical turbulence and mixing, which in turn will break up the localized layer of stable air.

In the immediate vicinity of the site, the size of Harris Reservoir will be increased significantly as a result of a proposed increase in the surface elevation of the reservoir from approximately 67.1 m (220 ft.) msl to 73.2 m (240 ft.) msl. This represents a discontinuity in the ground surface over which diffusing gases travel. Harris Reservoir presents a smoother surface than does the land over which the air will travel. For wind directions to or from the north-northeast, there is an increased upwind/downwind fetch where the wind will travel over the smoother water surface than would occur in other directions. Under certain conditions, this could reduce the surface or mechanically induced turbulence and, in turn, the resulting diffusion of any effluents released from the facility. At the same time, however, the reduced frictional effects could allow for an increased wind speed, possibly mitigating any reduced diffusion caused by

turbulence. Because of the limited potential for the reservoir to influence diffusion, no adjustments to the diffusion calculations are proposed.

Figure 2.3.2-221 shows topographic features within an 80-km (50-mi.) radius of the HAR site. In general, the terrain slopes upward northwest of the site, averaging about 3 m (10 ft.) per mile, and reaches an elevation of approximately 244 m (800 ft.) at 80 km (50 mi.) from the plant. The terrain through the north and west sectors is gently rolling, ranging from about 30.5 m (100 ft.) msl to 152.4 m (500 ft.) msl, as described in the HNP FSAR.

2.3.2.2.2 Fogging and Icing Effects Attributable to Cooling Tower Operation

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

Ground fogging could occur if ground elevations in the plant vicinity were comparable to plume heights. However, the release elevation of the cooling tower plumes will be approximately 262 m (860 ft.) msl, and the highest ground elevations in the vicinity are approximately 131 m (430 ft.) msl (8 km [5 mi.] southeast of the site) and 122 m (400 ft.) msl (10 km [6 mi.] west of the site). Plumes will easily clear these areas without considering the rise of the plume above the release elevation. As a result, ground fogging attributable to cooling tower operation is not expected to occur.

Extended visible plumes from the cooling towers will likely occur during periods of high humidity when restricted visibility occurs naturally. Naturally occurring fog was reported an average of 845 hours per year during the 3-year period from 1955 to 1957. Visibilities were less than 0.8 km (0.5 mi.) for 124 hours per year during this period and less than 0.6 km (0.4 mi.) for 90 hours per year. As shown previously in Table 2.3.1-202, observations of heavy fog (less than 0.4-km [0.25-mi.] visibility) have been reported an average of 25 to 32 days per year at the four meteorological observation stations located within 192 km (120 mi.) of the site (i.e., Charlotte, Greensboro, Raleigh-Durham, and Wilmington). The operation of the additional cooling towers is not expected to result in a significant increase in ground-level fog at these locations (References 2.3-203, 2.3-204, 2.3-205, and 2.3-206).

Ice formation is not expected to occur on structures in the vicinity of the plant, either on-site or off-site. The proposed cooling towers for HAR 2 and HAR 3 will be 183-m (600-ft.) high, and the cooling tower plumes will normally rise at least 305 m (1000 ft.) above the tower in the most stable case. The tallest plant structure at the HAR site (the containment building) will be less than 250 ft. high (refer to DCD Figure 3.7.2-12) and there are no known tall structures in the site vicinity. In general, the cooling tower plumes are not expected to intersect any structures on or in the vicinity of the site. The only exception is during high winds. Cooling tower plumes tend to be short because of turbulent diffusion when winds are strong. Occasionally, the wake effect of a cooling tower can cause the plume

to curl below the lip. Flow around a cylindrical natural draft tower is designed to quickly remove the downwash, and the plume will either ascend or evaporate.

There are no large safety-related plant structures or other nearby structures that are expected to be affected by icing from the cooling tower plumes. During times of naturally occurring snowfall, it is conceivable that snow conditions could conceivably be more intense under the plume and cause greater accumulation on the surrounding area and roadways. However, this should not create any greater hazard, since normal precautions taken by travelers in such circumstances would be adequate. Such an effect is expected to be very local, if it occurs.

Based on the operational experience at HNP, there have been only very limited observations of icing or fogging attributable to cooling tower operation on HNP property. There have been no reported occurrences of fogging or icing attributable to cooling tower operation at any off-site locations, including public roads.

2.3.2.2.3 Assessment of Heat Dissipation Effects on the Atmosphere

The information used to evaluate the potential impact of the proposed cooling towers on local meteorological conditions is based on information available prior to the issuance of the HNP operating license. The natural draft cooling towers that will be used to dissipate waste heat from HAR 2 and HAR 3 to the atmosphere are expected to be similar in design to the tower currently in operation at HNP. The new cooling towers are not expected to have a significant influence on local meteorological conditions. This is a result primarily of the proposed height of the towers (approximately 183 m [600 ft.] above plant grade). After leaving the cooling towers, the plumes will typically rise another 305 to 914 m (1000 to 3000 ft.), depending on wind speed and atmospheric temperature conditions. At these elevations, the additional water and heat added to the atmosphere by the cooling tower plumes should not significantly affect conditions at ground level.

Under full power, it is expected that the cooling towers will evaporate between 30,280 and 45,420 liters per minute (l/min.) (8000 and 12,000 gallons per minute [gpm]) of water per unit, 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 conducive to longer plumes. Most of the time, the visible plume will extend only a short distance from the tower and then disappear by evaporation. A study of cooling tower plumes at Keystone Power Plant reported that plume lengths were less than 1524 m (5000 ft.) over 97 percent of the time (as described in the HNP FSAR). On very humid days, when longer plumes are expected, there may be a naturally occurring overcast. On such occasions, it is difficult to distinguish the cooling tower plume from the overcast cloud layer.

Long, persistent, visible cooling tower plumes occur during stable conditions where vertical mixing is limited. Under these conditions, plumes tend to flatten or spread out horizontal due to extremely limited vertical mixing.

An extensive analysis of cooling tower plume behavior was presented in the FSAR that was developed for HNP. An analytical cooling tower plume model was used to predict plume lengths and plume orientation with respect to the HNP for all hours with visibilities greater than 0.8 km (0.5 mi.) using 3 years of on-site data (January 14, 1976, to December 31, 1978). The percent occurrence of visible plumes was calculated in 250-m (820-ft.) plume length intervals. Plume characteristics were categorized by season and annual average. The results of the analysis, which were documented in the HNP FSAR, indicated that 99.6 percent of visible plumes would be less than 2.5 km (1.6 mi.) in length. The maximum predicted plume length was 3.5 km (2.1 mi.) and occurred on average only once in 3 years. Plumes 3 km (1.9 mi.) in length were predicted to occur only about 1 hour per year, and 2 km (1.2 mi.) plume lengths were predicted to occur only about 10 hours per year.

The nearest major airport to the plant is the Raleigh-Durham Airport, located 29 km (18 mi.) northeast of the plant. The operation of the cooling towers for HAR 2 and HAR 3 is not expected to result in an air traffic safety hazard at any location.

Table 2.3.2-263 shows the predicted seasonal frequencies of 1-, 2-, and 3-km (0.62-, 1.25-, and 4.8-mi.) visible plumes from the HNP cooling tower as a function of wind direction. The greatest frequency of visible plumes occurs during the winter and fall months. The longest visible plumes are expected during the winter because condensation is enhanced and plume lengths increase with increasing ambient moisture content and decreasing temperature. The greatest frequency of predicted visible plumes is associated with north-to-northeast and south-to-southwest winds, which indicates the importance of colder temperatures (winds with northerly components) and greater moisture (winds with southerly components) in producing plumes.

Figure 2.3.2-222 shows the predicted annual frequency of visible plumes associated with the operation of the HNP cooling tower. The results are similar to those given in Table 2.3.2-263, with the greatest frequency of visible plumes expected to the north of the plant and the longest plumes expected to the southwest of the plant.

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 at elevations well below the top of the cooling towers. Any such releases would be at or near ambient temperature, and no significant plume rise would occur. Because the cooling tower plumes would be at a much higher elevation, the potential for the mixing of the plumes is expected to be minimal and well downwind of where any water droplets in the cooling tower plume would still be present.

A very small fraction of the water circulating through the cooling towers will be carried into the plumes as small water droplets. These water droplets, referred to as “cooling tower drift” (typically defined as kilograms [kg] of water per second leaving the tower top divided by the kg of water per second circulating through the tower heat exchange section) would average about 0.002 percent.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Design and operating bases, such as tornado parameters, ice glaze thickness, and winter probable maximum precipitation, are statistics that, by definition and necessity, are based on long-term regional records. Although data collected through the HNP on-site meteorological monitoring system can be considered representative of long-term site meteorology, long-term regional data are considered most appropriate for use as conservative estimates of climatological extremes. Therefore, the design and operating basis conditions were based on regional meteorological data, as previously described in Subsection 2.3.1.

2.3.3 ON-SITE METEOROLOGICAL MEASUREMENT PROGRAMS

HAR COL 2.3-3 The HNP on-site meteorological measurement program began in March 1973 with the installation of a 61.4-m (201.4-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 HNP, HAR 2, and HAR 3. The area surrounding the tower is generally considered to be “grassy” 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. There are also a number of utility poles located within a few hundred feet of the tower that are used for training Progress Energy employees. The presence of these poles is not expected to have an influence on any tower measurements. The location of the HNP meteorological tower is ideally situated for use in support of the HAR COL Application. Therefore, the monitoring results obtained from the tower will be used to characterize the on-site meteorological conditions for the HAR. Topographical cross-sections of the region were provided in Figures 2.3.2-201, 2.3.2-202, 2.3.2-203, and 2.3.2-204, which show the topographical changes by direction from the center of the site out to a distance of 80 km (50 mi.).

Five years of continuous and consecutive meteorological data from the on-site tower for the period of March 1, 1994 through February 28, 1999, are submitted with this application in electronic format consistent with the requirements of 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 Subsection 2.3.4 and Subsection 2.3.5. As discussed in Subsection 2.3.2, these data are considered the most recent meteorological data representative of the site.

The planned operational meteorological monitoring program will be a continuation of the existing program. Since there is currently an operating meteorological program for the existing HNP that meets the guidance provided in Regulatory Guide 1.23, Revision 1, the existing meteorological program was used as the pre-operational monitoring program for HAR 2 and HAR 3 and the program is planned to be continued as the operational program for HAR 2 and HAR 3, as well as HNP. Given that the existing program is planned to be continued during operation, both programs are described jointly.

2.3.3.1 Instrumentation

The meteorological tower was first installed and began operation on the HNP site in March 1973 in support of the development, construction, and operation of the HNP. The on-site tower is located approximately 1.6 km (1 mi.) northeast of the HNP reactor. The largest structure in the area is the HNP natural draft cooling tower, which is located approximately 1280 m (4200 ft.) southwest of the meteorological tower. The cooling towers for HAR 2 and HAR 3 are expected to be located approximately 1067 m (3500 ft.) and 1097 m (3600 ft.) to the west-southwest of the meteorological tower. The locations of the existing and proposed cooling towers with respect to the meteorological tower are shown in Figure 2.3.3-201. NRC's Regulatory Guide 1.23, Revision 1 indicates that meteorological sensors should be located "at least 10 times the height of any nearby obstruction if the height of the obstruction exceeds one-half the height of the height of the wind measurement." This differs from the guidance previously provided in NUREG-1555, which stated that sensors should be located at least five times the height of nearby obstructions. Given that the existing and proposed cooling tower structures are located a considerable distance from the meteorological tower, they are not considered to be "nearby" in relation to the tower. Additionally, guidance provided by the USEPA (Reference 2.3-229) regarding building wake and downwash effects for simple and complex structures, including natural draft cooling towers, states that the downwind influence of structures should be calculated based of the lesser of the height or the width of structures and that the extent of influence is five times that dimension. The existing HNP cooling tower is 159 m (523 ft.) high, and its diameter at the top of the structure is 70 m (230 ft.). Using the diameter of the top of the tower as the controlling dimension, the zone of influence of the cooling tower would be considerably less than the distance to the meteorological tower. The meteorological tower is therefore considered to be sufficiently far from the existing and proposed towers to avoid any airflow modifications attributable to the presence of the existing and proposed cooling towers. The base of the meteorological tower is at an elevation of approximately 79.2 m (260 ft.) msl. An environmentally controlled shelter located approximately 12 m (40 ft.) to the northwest of the tower houses the system datalogger and remote access

equipment. The information monitored on the tower is routinely accessed and downloaded and is archived remotely. The information is also available to the HNP Control Room operators, as described in the HNP FSAR.

2.3.3.1.1 Wind Systems

Lower-level wind speeds are recorded by sensors mounted on 3.7-m (12-ft.) retractable booms oriented perpendicular to the prevailing wind flow, which is from the southwest to the northeast, 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, as described in the HNP FSAR.

2.3.3.1.2 Temperature Systems

Ambient temperature and delta-T are monitored at both the lower and upper 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, as described in the HNP FSAR.

Dew point temperatures have historically been measured at the lower level of the tower, including during the POR used to characterize the site, as described in Subsection 2.3.2. Dew point measurements made during this POR were made consistent with the requirements of ANSI/ANS 2.5-1984 (Reference 2.3-215). However, the current meteorological monitoring system measures relative humidity at the lower level of the tower consistent with the accuracy requirements of NRC Regulatory Guide 1.23, Revision 1 as described in Table 2.3.3-202. Regulatory Guide 1.23, Revision 1 also indicates that "…..atmospheric moisture measurements may be made at the highest measurement level on the meteorological tower." The existing natural cooling tower height at HNP is 158 m (520 ft.), which is far in excess of the meteorological tower height of 61 m (200 ft.). Given this difference, along with the expectation that the influence of cooling tower plumes will not be observed within 200 ft. of ground level at the location of the meteorological tower, atmospheric moisture measurements have not been performed at the upper level of the meteorological 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, as described in the HNP FSAR.

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 sensor calibration or replacement is therefore not necessary. Deviation between the two ambient/differential temperature channels provides an early warning of a problem with one of these channels.
- The guy wires and the tower anchors are inspected prior to instrument maintenance and calibration events on a semiannual basis..

2.3.3.1.5 Data Reduction

Data from the HNP datalogger system are retrieved via a remote connection through a dial-up telephone link. If the primary telephone line is inoperable, a second dedicated telephone line is also available for data retrieval. 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 the nearby Raleigh-Durham observation station. 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, as described in the HNP FSAR:

- 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. In general, the accuracy of the meteorological monitoring system during the 5-year POR of on-site data described in Subsection 2.3.2 was consistent with the requirements of NRC Regulatory Guide 1.23, Revision 0, with the exception of the Dew Point Temperature measurements, which met the requirements of ANSI/ANS 2.5-1984 (Reference 2.3-215). The current monitoring system is compliant with the requirements of NRC Regulatory Guide 1.23, Revision 1.

2.3.4 SHORT-TERM DIFFUSION ESTIMATES

2.3.4.1 Objective

HAR COL 2.3-4 Conservative estimates of the local atmospheric dilution factors (Chi/Q) for HAR 2 and HAR 3 were made using an atmospheric dispersion model and on-site meteorological data for the 5-year period March 1, 1994, through February 28, 1999. This data was originally prepared in accordance with NRC Regulatory Guide 1.23, Revision 0 that designates six wind speed categories (plus calms), and these data were formatted for use in NRC's PAVAN dispersion model. The PAVAN modeling results discussed in this section are based on these six wind speed categories (plus calms) as input to the model. This is an exception to NRC Regulatory Guide 1.23, Revision 1, which provides new guidance for the use of eleven wind speed categories (plus calms). The PAVAN analyses for Subsection 2.3.4 were performed prior to the issuance of Regulatory Guide 1.23, Revision 1. The information presented in this section was initially prepared using Regulatory Guide 1.23, Revision 0.

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 HAR 2 and HAR 3 exclusion area boundary (EAB) and minimum calculated low population zone (LPZ) distances of 1245 m (4085 ft.) and

4830 m (3 mi.), respectively. The HAR EAB, which was previously discussed in Section 2.1 and illustrated on Figure 2.1.1-203, is defined as two overlapping circles centered on the reactor building of each unit. The radius of each circle is 1245 m (4085 ft.). The overall shape of the HAR EAB is defined by the outermost boundary of each unit's circle. The HAR site is located within a much larger tract of land that includes the HNP EAB, Harris Reservoir, and some surrounding lands. The minimum distance in any direction from each reactor to an EAB is approximately 1245 m (4085 ft.). The measurements to the EAB used in this analysis represent the distances from the centerpoint of HAR 2 and HAR 3 to the outermost boundary of the EABs at the centerpoint of each sector as shown on Figure 2.1.1-203. The predicted HAR 2 and HAR 3 Chi/Q values are compared in Table 2.3.4-201 with the acceptance criteria established in Subsection 2.3.4 of Westinghouse's AP1000 DCD and listed in DCD Table 2-1 (values reproduced in the table). The maximum predicted Chi/Q values were determined in accordance with NRC Regulatory Guide 1.145 for the 0.5-percent maximum sector Chi/Q and the 5-percent direction independent value. In addition, 50- percent direction independent values were determined for use in the environmental report evaluations.

Input to the PAVAN model consisted of the following information:

Meteorological Data:	Joint frequency distribution of wind speed, wind direction, atmospheric stability for 16 standard azimuthal sectors. Period of record March 1994 – February 1999 (Table 2.3.4-202).
Wind Sensor Height:	Lower – 12 m (39 ft.).
Delta-T Heights:	12 – 61 m (39 – 200 ft.).
Number of Wind Speed Categories:	7.
Minimum Building Cross-Section:	2730 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 HAR 2 and HAR 3 with respect to the main reservoir and 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 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's, 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 HAR COL Application evaluations and analyses and are shown in Table 2.3.4-204.

The 50-percent EAB and LPZ Chi/Q values were determined from the PAVAN output and by logarithmic interpolation. The conservative reported 0- to 2-hour 50-percent values at the EAB and LPZ without building wake are 6.16E-05 sec/m^3 and 1.38E-05 sec/m^3, respectively. The remaining values for the longer time periods for the LPZ are determined using the 0- to 2-hour 50-percent LPZ value and the LPZ average annual value of 1.78E-06 sec/m^3 from the PAVAN output by logarithmic interpolation at the intermediate time periods of 8 hours, 16 hours, 72 hours, and 624 hours. The values are shown on Table 2.3.4-205.

2.3.4.4 Control Room Diffusion Estimates

Conservative estimates of the site-specific Chi/Q for HAR 2 and HAR 3 control room were made using an atmospheric dispersion model and on-site meteorological data. The meteorological data consist of hourly data, covering the period from March 1, 1994, through February 28, 1999. Each record of the hourly data contains a location identifier, Julian day, hour, low-level direction, low-level speed, stability class, upper level direction, and upper level speed.

NRC's ARCON96 computer code was used to calculate short-term accident Chi/Q values for the HAR 2 and HAR 3 control room. The predicted HAR 2 and HAR 3 Chi/Q values are compared in Table 2.3.4-206 with the acceptance criteria established in Subsection 2.3.4 of Westinghouse's AP1000 DCD and listed in Table 2-1 of the AP1000 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 AP1000 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. Figure 2.1.1-203 shows the plant layout on the site, including the true north direction. 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

HAR COL 2.3-5 Estimates of long-term Chi/Q and relative deposition (D/Q) were estimated 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:

- Exclusion area boundary (as described in Section 2.1.2 and Section 2.3.4.2).
- Low population zone (variable distance based on 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 HAR site.

All distances are measured from a location defined by the mid-point of the two proposed units. Subsection 2.3.5.2 provides additional information on the calculations and results of long-term Chi/Q estimates for the HAR site.

2.3.5.2 Calculations

The calculations of Chi/Q and D/Q at the locations and distances listed above were made using NRC's XOQDOQ computer program using 5 years of hourly, on-site meteorological data.

Assumptions used in the analysis are summarized below:

- Meteorological Data Source – HNP on-site meteorological tower.
- Period of Record – March 1, 1994 to February 28, 1999.

- Wind Reference Level – 12 m (39 ft.).
- Stability Calculation – Delta-Temperature (12- and 61-m [39- and 200-ft.] tower levels).
- Release Type – Ground level.
- Release Height – 10 m (33 ft.).
- Building Wake Effects – Included (see 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 HNP, 8 km (5 mi.) was assumed as the location of the receptor. Also, some locations represent historical activities (milk cows for the N and NNE sectors) but were retained to be conservative.

Based on the location of HAR 2 and HAR 3 with respect to surrounding topography and the main and auxiliary reservoirs, the atmospheric diffusion parameter, sigma z, is not expected to be significantly influenced by the topographical conditions. Therefore, no modifications were made to this atmospheric dispersion parameter.

The results of the long-term Chi/Q and relative deposition (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 that account 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 6.00E-06 sec/m^3 for any given sector (i.e., SSW sector; refer to Table 2.3.5-201).

The DCD site boundary average annual X/Q is 2.0 X 10^{-5} sec/m^3. This bounds the HAR average annual EAB X/Q value. Table 2.0-201 provides a comparison of the HAR site characteristics with the DCD design parameters.

2.3.6 COMBINED LICENSE INFORMATION

2.3.6.1 Regional Climatology

This COL Item is addressed in Subsection 2.3.1.

HAR COL 2.3-1

2.3.6.2 Local Meteorology

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

2.3.6.3 On-Site Meteorological Measurements Program

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

2.3.6.4 Short-Term Diffusion Estimates

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

2.3.6.5 Long-Term Diffusion Estimates

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

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HAR COL 2.3-1

Table 2.3.1-201
Regional Meteorological Observation Station Locations

Station	Latitude			Longitude			Distance from HAR Site	Direction from HAR Site
	deg	min	sec	deg	min	sec	km (mi.)	(Compass)
Charlotte, NC	35	12	52	80	56	37	188 (117)	WSW
Greensboro, NC	36	5	51	79	56	37	111 (69)	WNW
Raleigh-Durham, NC	35	52	14	78	47	11	30 (19)	NNE
Wilmington, NC	34	16	6	77	54	22	179 (111)	SSE

Source: Reference 2.3-203, Reference 2.3-204, Reference 2.3-205, and Reference 2.3-206

HAR COL 2.3-1

Table 2.3.1-202 (Sheet 1 of 2)
Climatological Data from Charlotte, Greensboro, Raleigh-Durham, and Wilmington, North Carolina

	Station							
Parameter	**Charlotte/ Douglas**	**POR (yrs)**	**Greensboro/ Piedmont**	**POR (yrs)**	**Raleigh-Durham**	**POR (yrs)**	**Wilmington**	**POR (yrs)**
Location								
Distance from HAR Site (mi.)	117		69		19		111	
Direction from HAR Site	West Southwest		West Northwest		North Northeast		South Southeast	
Elevation Above Mean Sea Level (ft.)	721		904		427		30	
Temperature								
Average Annual Observed (°F)	60.5	58	58.1	58	59.5	58	63.6	58
Maximum Observed (°F)	104 (Sept. 1954)	66	103 (Aug. 1988)	77	105 (Aug. 1988)	61	104 (Jun. 1952)	54
Minimum Observed (°F)	-5 (Jan. 1985)	66	-8 (Jan. 1985)	77	-9 (Jan. 1985)	61	0 (Dec. 1989)	54
Normal Degree days/year (heating)	3162	30	3848	30	3465	30	2429	30
Normal Degree days/year (cooling)	1681	30	1332	30	1521	30	2017	30
Relative Humidity (%)								
Annual average at 7 A.M.	82	30	83	30	85	30	85	30
Annual average at 1 P.M.	53	30	55	30	54	30	57	30
Wind								
Annual average speed (mph)	7.4	56	7.5	52	7.4	51	8.3	39
Prevailing direction	South	33	Southwest	36	Southwest	36	Southwest	29
Fastest mile/Peak gust[(a)]								
Speed (mph)[(a)]	87 (Sept. 1989)	7	97 (July. 1996)[(b)]	6	62 (Feb. 1984)	7	78 (July 1986)	7
Direction[(a)]	East	7	Northwest	6	Southeast	7	Southwest	7

HAR COL 2.3-1

Table 2.3.1-202 (Sheet 2 of 2)
Climatological Data from Charlotte, Greensboro, Raleigh-Durham, and Wilmington, North Carolina

	Station							
Parameter	**Charlotte/ Douglas**	**POR (yrs)**	**Greensboro/ Piedmont**	**POR (yrs)**	**Raleigh-Durham**	**POR (yrs)**	**Wilmington**	**POR (yrs)**
Precipitation (in.)								
Annual average	43.51	30	43.14	30	43.05	30	57.07	30
Monthly maximum	14.72 (Oct. 1990)	66	13.26 (Sept. 1947)	77	21.79 (Sept. 1999)	61	23.41 (Sept. 1999)	54
Monthly minimum	Trace Amount (Oct. 1953)	66	Trace Amount (Jun. 1990)	77	0.23 (Sept. 1985)	61	0.16 (Apr. 1995)	54
24-hour maximum	5.46 (Oct. 1990)	66	7.49 (Sept. 1947)	77	5.78 (Oct. 2002)	61	14.84 (Sept. 1999)	54
Maximum annual	67.10 (1936)	115	62.32 (2003)	75	64.22 (1936)	116	72.06 (1999)	75
Snowfall (in.)								
Annual average	5.2	30	8.9	30	7.1	30	2.1	30
Monthly maximum	19.3 (Mar. 1960)	66	22.9 (Jan. 1966)	77	25.8 (Jan. 2000)	61	15.3 (Dec. 1989)	54
Maximum 24-hour	12.1 (Jan. 1988)	66	14.3 (Dec. 1930)	77	17.9 (Jan. 2000)	61	11.7 (Feb. 1973)	54
Mean Annual (number of days)								
Precipitation ≥ 0.01 in.	113.2	30	113.7	30	113.1	30	118.1	30
Snow, sleet, hail ≥ 1.0 in.	1.4	30	2.5	30	1.8	30	0.5	30
Heavy fog (visibility 0.25 mi or less)	25.2	67	32.4	78	32.5	56	25.2	54
Maximum temperature ≥ 90°F	40.3	30	29.3	30	39.3	30	46.3	30
Minimum temperature ≤ 32°F	57.9	30	79.1	30	72.7	30	39.3	30

a) Reference 2.3-202
b) Reference 2.3-231

Sources: Reference 2.3-203, Reference 2.3-204, Reference 2.3-205, Reference 2.3-206, and Reference 2.3-232

HAR COL 2.3-1

Table 2.3.1-203
Summary of Reported Tornado Occurrences in North Carolina

Tornado Intensity (Fujita Tornado Scale)	Number of Reported Occurrences January 1, 1950 – July 31, 2006
F0	372
F1	419
F2	172
F3	45
F4	27
F5	0

Notes:
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

HAR COL 2.3-1

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

County	F0	F1	F2	F3	F4	F5	No. of Reported Occurrences (1950 – 2006)
Wake	12	9	6	0	1	0	28
Chatham	3	3	2	0	0	0	8
Lee	1	1	1	0	0	0	3
Harnett	7	10	3	1	0	0	21
Johnston	1	5	0	1	0	0	7
Orange	4	1	1	1	0	0	7
Durham	2	1	3	0	0	0	6
Alamance	0	3	0	0	0	0	3

Notes: These statistics are based on the reporting periods between January 1, 1950, and July 31, 2006.

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

HAR COL 2.3-1

Table 2.3.1-205 (Sheet 1 of 2)
Reported Tornado Occurrences in North Carolina, 1950 to 2006

Year	F0	F1	F2	F3	F4	F5	Total
1950	1	6	1	1	0	0	**9**
1951	0	3	1	0	0	0	**4**
1952	0	1	5	1	0	0	**7**
1953	0	1	1	3	0	0	**5**
1954	0	4	5	0	0	0	**9**
1955	0	4	2	0	0	0	**6**
1956	1	7	2	0	0	0	**10**
1957	0	4	0	4	5	0	**13**
1958	2	2	0	0	0	0	**4**
1959	2	9	0	0	0	0	**11**
1960	0	8	2	0	0	0	**10**
1961	1	6	1	0	0	0	**8**
1962	1	1	1	0	0	0	**3**
1963	0	4	7	0	0	0	**11**
1964	2	7	4	2	0	0	**15**
1965	0	3	4	4	0	0	**11**
1966	0	4	4	0	0	0	**8**
1967	1	4	3	0	0	0	**8**
1968	0	3	3	0	0	0	**6**
1969	0	2	5	3	0	0	**10**
1970	0	2	0	0	0	0	**2**
1971	0	1	2	2	0	0	**5**
1972	2	0	1	1	0	0	**4**
1973	13	23	4	1	0	0	**41**
1974	3	13	6	0	1	0	**23**
1975	5	27	6	0	0	0	**38**
1976	1	16	6	0	0	0	**23**
1977	7	22	1	1	0	0	**31**
1978	2	7	5	0	0	0	**14**
1979	4	5	3	0	0	0	**12**
1980	7	6	1	0	0	0	**14**
1981	0	3	9	0	0	0	**12**
1982	2	6	5	0	0	0	**13**
1983	3	6	7	0	0	0	**16**

HAR COL 2.3-1

Table 2.3.1-205 (Sheet 2 of 2)
Reported Tornado Occurrences in North Carolina, 1950 to 2006

Year	F0	F1	F2	F3	F4	F5	Total
1984	3	8	7	7	10	0	**35**
1985	6	1	0	1	0	0	**8**
1986	8	4	5	0	0	0	**17**
1987	2	1	1	0	0	0	**4**
1988	5	10	4	0	5	0	**24**
1989	1	13	6	1	5	0	**26**
1990	6	7	4	1	0	0	**18**
1991	12	14	3	0	0	0	**29**
1992	15	15	1	10	0	0	**41**
1993	4	7	0	0	0	0	**11**
1994	9	3	1	0	0	0	**13**
1995	13	8	3	0	0	0	**24**
1996	28	19	9	0	0	0	**56**
1997	8	3	0	0	0	0	**11**
1998	30	27	6	2	1	0	**66**
1999	21	8	9	0	0	0	**38**
2000	23	1	0	0	0	0	**24**
2001	11	2	0	0	0	0	**13**
2002	6	4	1	0	0	0	**11**
2003	27	9	0	0	0	0	**36**
2004	50	18	2	0	0	0	**70**
2005	12	8	3	0	0	0	**23**
2006[a]	12	9	0	0	0	0	**21**
Total	372	419	172	45	27	0	**1035**
Average	7	7	3	1	0	0	**18**

a) Data reported for the year 2006 represent the period from January 1, 2006 to July 31, 2006.

Notes:
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

HAR COL 2.3-1

Table 2.3.1-206
Summary of Wet and Dry Bulb Temperature Observations

	Charlotte/Douglas		Greensboro		Raleigh-Durham	
	Wet Bulb (°F)	**Dry Bulb (°F)**	**Wet Bulb (°F)**	**Dry Bulb (°F)**	**Wet Bulb (°F)**	**Dry Bulb (°F)**
Highest Running Average Wet Bulb (with Coincident Dry Bulb)						
30 Day Average	73	81	74	80	74	79
5 Day Average	75	84	76	83	77	85
1 Day Average	77	85	78	85	79	87
Maximum Ambient Dry Bulb (with Coincident Wet Bulb)						
0% Exceedance	75	103	73	103	72	104
1% Exceedance	74	91	74	90	75	91
Minimum Ambient Dry Bulb (with Coincident Wet Bulb)						
100% Exceedance	-7	-5	-10	-8	-9	-7
99% Exceedance	20	23	17	19	18	20
Maximum Ambient Wet Bulb (with Coincident Dry Bulb)						
0% Exceedance	81	89	81	89	83	92
1% Exceedance	76	87	76	86	77	86

Source: Reference 2.3-220

HAR COL 2.3-1

Table 2.3.1-207
Seasonal Frequencies of Inversions below 152 m (500 ft.) in Greensboro, North Carolina

	Percent Frequency of Inversions Based Below 500 ft.				
Season	**0300 GMT**	**1500 GMT**	**0000 GMT**	**1200 GMT**	**All Times**
Winter	73	15	58	72	43
Spring	70	3	13	66	32
Summer	78	1	11	66	33
Fall	74	4	52	74	40

Notes: GMT – Greenwich Mean Time

Source: Reference 2.3-221

HAR COL 2.3-1

Table 2.3.1-208
Mean Monthly Mixing Depths at Greensboro, North Carolina

Month	Depth (m)
January	390
February	650
March	1130
April	1180
May	1530
June	1790
July	1490
August	1420
September	1370
October	1020
November	840
December	580

Sources: Reference 2.3-222 and Reference 2.3-223

HAR COL 2.3-1

Table 2.3.1-209 (Sheet 1 of 2)
Ambient Dry and Wet Bulb Temperature Observations for Charlotte/Douglas, Greensboro, and Raleigh-Durham, North Carolina

Maximum and Minimum Dry Bulb Temperatures (with Coincident Wet Bulb Temperatures) (°F)						
	Charlotte/Douglas		Greensboro		Raleigh-Durham	
	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb	Dry Bulb	Coincident Wet Bulb
Maximum Temperatures						
0% Occurrence	103	75	103	73	104	72
0.4% Occurrence	94	75	93	75	93	76
1.0% Occurrence	91	74	90	74	91	75
2.0% Occurrence	89	73	88	73	89	75
"Maximum Safety" (a)	(e)	(e)	(e)	(e)	106.6	73.6
"Maximum Normal" (b)	94	75	93	75	93	76
Minimum Temperatures						
97.5% Occurrence	28	25	24	21	26	23
99.0% Occurrence	23	20	19	17	20	18
99.6% Occurrence	19	17	14	12	16	14
100% Occurrence	-5	-7	-8	-10	-7	-9
"Minimum Safety" (c)	-8	NA	-11	NA	-10	NA
"Minimum Normal" (d)	19	17	14	12	16	14

HAR COL 2.3-1

Table 2.3.1-209 (Sheet 2 of 2)
Ambient Dry and Wet Bulb Temperature Observations for Charlotte/Douglas, Greensboro, and Raleigh-Durham, North Carolina

Maximum Wet Bulb Temperatures (with Coincident Dry Bulb Temperatures) (°F)						
	Charlotte/Douglas		Greensboro		Raleigh-Durham	
	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb	Wet Bulb	Coincident Dry Bulb
0% Occurrence	81	89	81	89	83	92
0.4% Occurrence	77	88	77	87	78	88
1.0% Occurrence	76	87	76	86	77	86
2.0% Occurrence	75	85	75	85	76	85
"Maximum Safety" (a)	(e)	(e)	(e)	(e)	83.5	NA
"Maximum Normal" (b)	76	NA	76	NA	77	NA

a) "Maximum Safety" temperatures are 100-year estimates based on available POR and regression analyses.
b) "Maximum Normal" temperatures are based the 0.4 percent annual occurrence temperatures from a 24-year POR from Reference 2.3-217.
c) "Minimum Safety" temperatures are 100-year estimates based on a 30-year POR from Reference 2.3-227.
d) "Minimum Normal" temperatures are based on the 99.6 percent annual occurrence temperatures from a 24-year POR from Reference 2.3-217.
e) Maximum Safety values not calculated for these stations.

°F = degrees Fahrenheit; NA = Not Applicable per AP1000 DCD
Sources: References 2.3-217, 2.3-220, and 2.3-227

HAR COL 2.3-2

Table 2.3.2-201
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category A

Wind Level: Lower Level
Stability Category: A
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	0	0	0	0	0	1	0	0	0	2	1	1	0	0	0	1	6
4-7	8	14	9	6	7	6	10	14	37	19	26	56	8	3	19	15	257
8-12	12	23	6	6	8	1	2	3	21	40	37	102	13	12	28	30	344
13-18	0	0	0	0	0	0	0	0	0	0	2	5	0	1	1	0	9
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	20	37	15	12	15	8	12	17	58	61	66	164	21	16	48	46	616

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 0

HAR COL 2.3-2

Table 2.3.2-202
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category B

Wind Level: Lower Level
Stability Category: B
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	5	2	2	2	0	3	0	2	1	5	5	2	0	1	5	2	37
4-7	50	46	37	41	23	19	21	26	76	51	42	130	18	17	72	65	734
8-12	19	38	26	6	4	1	0	3	15	32	49	88	23	29	75	46	454
13-18	0	0	0	0	0	0	0	0	0	2	1	4	2	2	2	0	13
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	74	86	65	49	27	23	21	31	92	90	97	224	43	49	154	113	1238

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 0

HAR COL 2.3-2

Table 2.3.2-203
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category C

Wind Level: Lower Level
Stability Category: C
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	5	4	11	4	8	4	6	9	11	18	11	9	7	7	4	7	125
4-7	122	101	78	96	51	55	55	70	143	109	96	188	62	69	115	117	1527
8-12	29	44	48	11	7	8	3	6	30	42	54	94	29	61	93	54	613
13-18	0	0	0	0	0	0	0	0	0	1	3	7	4	5	4	0	24
19-24	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	156	149	137	111	66	67	64	85	184	170	164	299	102	142	216	178	2290

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 0

HAR COL 2.3-2

Table 2.3.2-204
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category D

Wind Level: Lower Level
Stability Category: D
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	255	260	214	204	153	138	156	157	219	224	206	183	131	147	152	208	3007
4-7	982	876	671	567	321	258	287	376	536	490	578	756	380	374	535	614	8601
8-12	171	168	102	44	7	14	24	44	86	111	159	231	79	195	249	172	1856
13-18	1	1	1	0	0	0	1	5	1	0	12	26	8	16	13	2	87
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1409	1305	988	815	481	410	468	582	842	825	955	1196	598	732	949	996	13,551

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 56

HAR COL 2.3-2

Table 2.3.2-205
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category E

Wind Level: Lower Level
Stability Category: E
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	564	537	407	330	244	208	239	283	437	591	454	364	253	226	229	344	5710
4-7	572	388	255	192	125	142	229	405	575	638	500	449	196	201	230	381	5478
8-12	59	65	33	16	11	13	26	50	129	68	62	81	25	68	72	72	850
13-18	21	4	0	2	4	0	2	5	5	2	2	9	0	3	1	4	64
19-24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1218	994	695	540	384	363	496	743	1146	1299	1018	903	474	498	532	801	12,104

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 61

HAR COL 2.3-2

Table 2.3.2-206
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category F

Wind Level: Lower Level
Stability Category: F
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	513	458	369	266	189	173	131	165	195	255	227	224	215	220	188	270	4058
4-7	78	18	7	11	14	10	19	23	38	27	36	75	23	24	24	58	485
8-12	0	1	0	0	0	0	0	0	0	0	0	0	0	1	1	1	4
13-18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	591	477	376	277	203	183	150	188	233	282	263	299	238	245	213	329	4547

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 127

HAR COL 2.3-2

Table 2.3.2-207
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, Category G

Wind Level: Lower Level
Stability Category: G
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	1442	1777	1139	500	305	180	123	91	100	142	172	232	232	372	406	702	7915
4-7	23	2	2	1	1	5	5	3	0	3	0	6	2	4	1	7	65
8-12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13-18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1465	1779	1141	501	306	185	128	94	100	145	172	238	234	376	407	709	7980

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 742

HAR COL 2.3-2

Table 2.3.2-208
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	2784	3038	2142	1306	899	707	655	707	963	1237	1076	1015	838	973	984	1534	20,858
4-7	1835	1445	1059	914	542	495	626	917	1405	1337	1278	1660	689	692	996	1257	17,147
8-12	290	339	215	83	37	37	55	106	281	293	361	596	169	366	518	375	4121
13-18	22	5	1	2	4	0	3	10	6	5	20	51	14	27	21	6	197
19-24	2	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	3
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	4933	4827	3417	2305	1482	1239	1339	1740	2655	2872	2735	3323	1710	2058	2519	3172	42,326

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 986

HAR COL 2.3-2

Table 2.3.2-209
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1995
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/95

	Wind Direction (Blowing From)[b]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	471	490	343	227	152	148	141	163	246	251	247	218	180	173	197	243	3890
4-7	361	276	189	164	95	96	102	165	374	367	307	323	103	115	158	232	3427
8-12	113	115	71	22	4	11	20	15	62	79	107	177	37	52	97	105	1087
13-18	1	2	0	1	0	0	3	2	0	1	3	3	4	5	4	1	30
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	946	883	603	414	251	255	266	345	682	698	664	721	324	345	456	581	8434

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 302

HAR COL 2.3-2

Table 2.3.2-210
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1995 to February 29, 1996
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/95 – 02/29/96

	Wind Direction (Blowing From)[b]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	598	734	532	314	200	147	127	132	153	206	165	151	130	149	166	302	4206
4-7	395	307	251	236	135	119	153	189	256	258	202	246	107	150	179	268	3451
8-12	34	43	48	14	9	3	12	26	70	22	51	85	25	63	92	67	664
13-18	0	0	0	0	0	0	0	6	5	1	3	8	4	4	0	0	31
19-24	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1027	1084	831	564	344	269	292	353	484	487	421	491	266	366	437	637	8353

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 162

HAR COL 2.3-2

Table 2.3.2-211
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1996 to February 28, 1997
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/96 – 02/28/97

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	585	631	455	291	186	165	174	156	198	245	255	197	201	198	193	337	4467
4-7	313	248	195	159	98	110	148	244	271	258	285	361	106	127	187	238	3348
8-12	27	61	38	12	5	6	10	19	62	73	80	128	25	103	133	32	814
13-18	8	1	0	0	0	0	0	0	0	2	11	23	0	3	0	3	51
19-24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	935	941	688	462	289	281	332	419	531	578	631	709	332	431	513	610	8682

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 65

HAR COL 2.3-2

Table 2.3.2-212
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1997 to February 28, 1998
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/97 – 02/28/98

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	540	667	438	231	151	125	112	125	141	244	191	197	163	187	208	298	4018
4-7	401	315	213	184	115	97	107	154	239	234	233	364	199	157	227	245	3484
8-12	57	59	28	21	17	14	7	25	40	54	41	100	39	85	116	85	788
13-18	1	2	1	1	4	0	0	0	0	0	1	3	6	10	17	0	46
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	999	1043	680	437	287	236	226	304	420	532	466	664	407	439	568	628	8336

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 362

HAR COL 2.3-2

Table 2.3.2-213
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1998 to February 28, 1999
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/98 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	590	516	374	243	210	122	101	131	225	291	218	252	164	266	220	354	4277
4-7	365	299	211	171	99	73	116	165	265	220	251	366	174	143	245	274	3437
8-12	59	61	30	14	2	3	6	21	47	65	82	106	43	63	80	86	768
13-18	12	0	0	0	0	0	0	2	1	1	2	14	0	5	0	2	39
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1026	876	615	428	311	198	223	319	538	577	553	738	381	477	545	716	8521

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 95

HAR COL 2.3-2

Table 2.3.2-214
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Percentage of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/99

Speed (mph)	Wind Direction (Blowing From)(b)																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	6.43	7.01	4.95	3.02	2.08	1.63	1.51	1.63	2.22	2.86	2.48	2.34	1.93	2.25	6.43	7.01	48.16
4-7	4.24	3.34	2.45	2.11	1.25	1.14	1.45	2.12	3.24	3.09	2.95	3.83	1.59	1.60	4.24	3.34	39.59
8-12	0.67	0.78	0.50	0.19	0.09	0.09	0.13	0.24	0.65	0.68	0.83	1.38	0.39	0.85	0.67	0.78	9.51
13-18	0.05	0.01	0.00	0.00	0.01	0.00	0.01	0.02	0.01	0.01	0.05	0.12	0.03	0.06	0.05	0.01	0.45
19-24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01
>24	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total	11.39	11.14	7.89	5.32	3.42	2.86	3.09	4.02	6.13	6.63	6.31	7.67	3.95	4.75	5.82	7.32	97.72

a) Data represents the percentage of total observations that a condition occurred (excluding calm winds).

b) E = East, N = North, S = South, W = West

Notes: Missing Hours: 512

Number of Calm Hours: 986 (2.28%)

Total Observations: 43,312

HAR COL 2.3-2

Table 2.3.2-215
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: January (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: January (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	219	221	144	125	56	45	48	46	60	95	82	85	90	106	116	172	1710
4-7	133	92	84	98	38	46	75	77	94	93	108	145	60	81	127	119	1470
8-12	16	6	15	12	0	3	24	23	27	18	29	75	17	50	33	32	380
13-18	0	0	0	0	0	0	3	8	4	1	5	6	3	4	0	0	34
19-24	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	368	319	243	235	94	94	150	154	185	207	224	312	170	241	276	323	3595

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 99

HAR COL 2.3-2

Table 2.3.2-216
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: February (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: February (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	198	180	131	76	73	56	37	25	67	89	85	88	59	82	87	129	1462
4-7	145	103	63	63	25	40	39	60	79	114	94	116	55	57	67	118	1238
8-12	44	24	14	14	7	4	3	4	16	23	40	63	20	48	87	52	463
13-18	1	2	1	1	4	0	0	0	0	1	2	1	2	9	9	0	33
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	388	309	209	154	109	100	79	89	162	227	221	268	136	196	250	299	3196

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 47

HAR COL 2.3-2

Table 2.3.2-217
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: March (All Years)[(b)]

	Wind Direction (Blowing From)[(c)]																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	183	211	174	94	51	48	53	56	68	91	72	34	41	68	53	84	1381
4-7	123	134	113	97	56	54	60	88	112	117	115	130	51	58	101	150	1559
8-12	26	35	27	12	6	11	10	16	60	64	49	68	25	73	114	86	682
13-18	0	0	0	0	0	0	0	0	0	0	3	33	3	5	9	1	54
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	332	380	314	203	113	113	123	160	240	272	239	265	120	204	277	321	3676

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 31

HAR COL 2.3-2

Table 2.3.2-218
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: April (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: April (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	131	193	109	68	52	39	37	49	84	104	80	66	67	74	71	115	1339
4-7	133	86	78	58	43	37	42	74	224	212	122	163	58	68	93	111	1602
8-12	27	20	21	5	1	2	3	13	70	96	62	89	25	48	63	52	597
13-18	0	0	0	0	0	0	0	0	0	2	1	7	1	5	3	0	19
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	291	299	208	131	96	78	82	136	378	414	265	325	151	195	230	278	3557

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 43

HAR COL 2.3-2

Table 2.3.2-219
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: May (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: May (All Years)(b)

	Wind Direction (Blowing From)(c)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	203	213	186	100	67	54	52	67	76	116	80	74	52	73	73	113	1599
4-7	182	141	67	82	45	40	56	94	128	165	130	171	64	74	83	99	1621
8-12	18	32	14	7	11	2	0	11	10	33	58	62	16	44	53	20	391
13-18	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	2
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	403	386	267	189	123	96	108	172	214	314	270	307	132	191	209	232	3613

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 54

HAR COL 2.3-2

Table 2.3.2-220
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: June (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: June (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	184	202	191	135	80	75	76	73	91	112	86	92	70	62	70	88	1687
4-7	94	113	83	78	71	44	92	116	136	141	122	156	56	66	80	76	1524
8-12	8	4	9	4	6	2	1	3	14	11	29	60	11	16	13	17	208
13-18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	286	319	283	217	157	121	169	192	241	264	237	308	137	144	163	181	3419

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 81

HAR COL 2.3-2

Table 2.3.2-221
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: July (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: July (All Years)(b)

Speed (mph)	Wind Direction (Blowing From)(c)																
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	156	206	174	144	125	99	94	98	119	145	118	113	67	76	48	98	1880
4-7	80	65	90	84	49	50	72	129	239	157	145	194	42	63	58	65	1582
8-12	2	11	1	3	1	7	2	7	23	9	18	62	7	6	14	3	176
13-18	3	0	0	0	0	0	0	0	0	0	0	0	1	1	0	3	8
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	241	282	265	231	175	156	168	234	381	311	281	369	117	146	120	169	3646

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 67

HAR COL 2.3-2

Table 2.3.2-222
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: August (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: August (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	268	314	231	166	92	71	74	61	105	125	109	83	51	53	65	97	1965
4-7	117	151	119	101	87	76	64	69	118	83	102	100	48	37	60	59	1391
8-12	11	32	17	6	5	5	3	4	13	10	13	21	1	3	7	13	164
13-18	12	0	0	0	0	0	0	0	0	0	0	2	0	0	0	2	16
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	408	497	367	273	184	152	141	134	236	218	224	206	100	93	132	171	3536

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 184

HAR COL 2.3-2

Table 2.3.2-223
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: September (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: September (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	294	293	219	116	61	65	47	69	110	92	104	77	52	57	59	122	1837
4-7	206	165	123	84	44	41	52	78	84	66	92	137	57	24	67	59	1379
8-12	12	40	33	4	0	0	5	6	8	3	12	16	8	7	20	14	188
13-18	2	1	0	0	0	0	0	0	0	0	6	0	1	0	0	0	10
19-24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	516	499	375	204	105	106	104	153	202	161	214	230	118	88	146	195	3416

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 172

HAR COL 2.3-2

Table 2.3.2-224
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: October (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: October (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	366	425	256	113	74	43	51	69	63	71	71	84	58	84	78	157	2063
4-7	210	160	117	75	41	21	37	59	80	53	73	119	36	43	65	88	1277
8-12	34	52	34	10	0	0	2	10	9	5	9	11	10	15	22	20	243
13-18	4	2	0	1	0	0	0	2	0	0	0	0	0	0	0	0	9
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	614	639	407	199	115	64	90	140	152	129	153	214	104	142	165	265	3592

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 103

HAR COL 2.3-2

Table 2.3.2-225
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: November (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: November (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	308	322	185	79	91	74	44	53	62	99	100	113	85	109	114	155	1993
4-7	185	114	42	32	18	21	26	33	52	90	103	132	79	42	99	172	1240
8-12	45	34	18	1	0	1	2	5	26	16	28	27	16	29	34	24	306
13-18	0	0	0	0	0	0	0	0	2	1	1	2	1	2	0	0	9
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	538	470	245	112	109	96	72	91	142	206	232	274	181	182	247	351	3548

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 43

HAR COL 2.3-2

Table 2.3.2-226
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: December (All Years)
Lower Wind Level, All Categories

Wind Level: Lower Level
Stability Category: ALL
Period of Record: December (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	274	258	142	90	77	38	42	41	58	98	89	106	146	129	150	204	1942
4-7	227	121	80	62	25	25	11	40	59	46	72	97	83	79	96	141	1264
8-12	47	49	12	5	0	0	0	4	5	5	14	42	13	27	58	42	323
13-18	0	0	0	0	0	0	0	0	0	0	0	0	2	1	0	0	3
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	548	428	234	157	102	63	53	85	122	149	175	245	244	236	304	387	3532

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 62

HAR COL 2.3-2

Table 2.3.2-227
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category A

Wind Level: Upper Level
Stability Category: A
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	0	0	0	0	1	0	0	0	0	0	2	1	0	0	1	0	5
4-7	2	7	4	2	1	2	4	7	10	6	5	8	4	2	4	5	73
8-12	9	18	5	9	11	5	5	6	29	31	32	74	5	5	18	21	283
13-18	11	15	6	2	0	0	0	0	5	38	38	63	6	13	16	17	230
19-24	0	0	0	0	0	0	0	0	0	4	2	13	0	1	3	2	25
>24	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1
TOTAL	22	40	15	13	13	7	9	13	44	79	80	159	15	21	42	45	617

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 0

HAR COL 2.3-2

Table 2.3.2-228
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category B

Wind Level: Upper Level
Stability Category: B
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	2	5	1	1	1	0	1	0	0	3	2	3	3	1	3	2	28
4-7	29	20	24	23	10	10	15	13	28	18	23	44	3	7	30	24	321
8-12	26	55	38	20	15	5	6	7	47	49	37	123	16	33	66	58	601
13-18	11	17	16	0	2	0	0	0	4	22	46	48	13	12	39	23	253
19-24	1	0	0	0	0	0	0	0	0	2	10	8	4	3	3	4	35
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	69	97	79	44	28	15	22	20	79	94	118	226	39	56	141	111	1238

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 0

HAR COL 2.3-2

Table 2.3.2-229
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category C

Wind Level: Upper Level
Stability Category: C
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	4	4	7	5	3	3	4	5	5	11	18	9	2	4	7	5	96
4-7	60	42	44	70	32	34	30	38	77	54	61	68	26	34	60	47	777
8-12	74	90	59	55	23	13	14	20	59	85	71	162	38	47	106	84	1000
13-18	15	22	25	6	2	3	1	1	14	45	42	53	19	39	45	29	361
19-24	1	1	0	1	0	0	0	1	0	6	12	14	6	4	5	2	53
>24	0	0	0	0	0	0	0	0	0	0	1	1	0	1	0	0	3
TOTAL	154	159	135	137	60	53	49	65	155	201	205	307	91	129	223	167	2290

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 1

HAR COL 2.3-2

Table 2.3.2-230
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category D

Wind Level: Upper Level
Stability Category: D
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	88	100	81	86	81	80	66	75	74	120	130	106	85	82	105	81	1440
4-7	414	510	393	389	255	207	236	212	376	418	355	491	238	230	342	389	5455
8-12	489	697	388	311	140	86	94	146	244	295	337	452	217	216	315	285	4712
13-18	164	156	119	54	7	5	16	25	60	158	174	178	65	138	154	154	1627
19-24	11	19	3	1	0	0	0	8	13	25	45	44	16	26	15	19	245
>24	2	0	1	0	0	0	0	0	0	0	9	15	0	2	2	0	31
TOTAL	1168	1482	985	841	483	378	412	466	767	1016	1050	1286	621	694	933	928	13,510

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 46

HAR COL 2.3-2

Table 2.3.2-231
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category E

Wind Level: Upper Level
Stability Category: E
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	45	46	27	51	47	43	59	60	51	64	74	59	51	61	57	35	830
4-7	174	233	224	191	127	140	154	178	316	426	362	275	186	138	144	163	3431
8-12	490	589	346	285	164	144	182	272	497	814	759	404	208	203	241	308	5906
13-18	138	88	65	31	19	16	22	67	186	298	251	173	53	77	94	143	1721
19-24	22	33	14	2	5	0	3	8	25	22	16	20	3	13	7	8	201
>24	11	1	0	0	1	0	0	0	2	1	0	2	0	0	0	2	20
TOTAL	880	990	676	560	363	343	420	585	1077	1625	1462	933	501	492	543	659	12,109

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 27

HAR COL 2.3-2

Table 2.3.2-232
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category F

Wind Level: Upper Level
Stability Category: F
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	11	20	20	17	18	25	23	30	39	30	34	30	22	24	29	24	396
4-7	61	56	94	84	99	86	79	98	167	178	151	120	103	62	102	60	1600
8-12	202	244	190	139	97	66	73	93	147	218	269	200	114	107	102	112	2373
13-18	61	28	13	9	6	2	0	3	14	21	34	31	12	20	10	31	295
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	335	348	317	249	220	179	175	224	367	447	488	381	251	213	243	227	4664

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 10

HAR COL 2.3-2

Table 2.3.2-233
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, Category G

Wind Level: Upper Level
Stability Category: G
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	63	65	46	54	55	61	64	81	88	132	142	94	83	96	106	81	1311
4-7	170	178	177	164	171	194	197	204	268	324	473	410	228	165	203	195	3721
8-12	234	250	221	232	245	187	171	154	164	201	395	363	177	136	122	208	3460
13-18	46	23	15	18	10	4	6	5	3	5	7	21	7	8	6	17	201
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	513	516	459	468	481	446	438	444	523	662	1017	888	495	405	437	501	8693

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 33

HAR COL 2.3-2

Table 2.3.2-234
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	213	240	182	214	206	212	217	251	257	360	402	302	246	268	308	228	4106
4-7	910	1046	960	923	695	673	715	750	1242	1424	1430	1416	788	638	885	883	15,378
8-12	1524	1943	1247	1051	695	506	545	698	1187	1693	1900	1778	775	747	970	1076	18,335
13-18	446	349	259	120	46	30	45	101	286	587	592	567	175	307	364	414	4688
19-24	35	53	17	4	5	0	3	17	38	59	85	99	29	47	33	35	559
>24	13	1	1	0	1	0	0	0	2	1	11	18	0	3	2	2	55
TOTAL	3141	3632	2666	2312	1648	1421	1525	1817	3012	4124	4420	4180	2013	2010	2562	2638	43,121

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 117

HAR COL 2.3-2

Table 2.3.2-235
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1995
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/95

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	35	48	30	32	38	38	30	40	52	87	95	60	48	49	61	46	789
4-7	132	153	147	169	136	98	107	133	254	319	318	312	133	117	137	146	2811
8-12	306	386	273	193	109	89	87	113	298	459	473	376	143	119	155	185	3764
13-18	139	113	92	37	14	9	15	21	64	138	125	134	43	53	75	119	1191
19-24	14	34	3	2	0	0	3	8	5	16	16	13	7	13	7	4	145
>24	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1
TOTAL	626	734	545	433	297	234	242	315	673	1019	1027	896	374	351	435	500	8701

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 36

HAR COL 2.3-2

Table 2.3.2-236
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1995 to February 29, 1996
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/95 – 02/29/96

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	37	42	30	46	47	45	43	43	56	49	65	64	37	45	54	44	747
4-7	196	253	234	204	172	169	161	160	282	319	231	234	136	103	180	162	3196
8-12	305	420	303	233	190	118	132	161	224	284	296	267	144	168	196	190	3631
13-18	68	51	45	20	5	6	8	22	67	74	95	91	32	62	67	76	789
19-24	1	0	0	0	0	0	0	3	23	4	10	18	9	9	1	5	83
>24	0	0	0	0	0	0	0	0	2	1	2	1	0	1	0	0	7
TOTAL	607	766	612	503	414	338	344	389	654	731	699	675	358	388	498	477	8453

a) Data represent the number of hours a condition occurred.
b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 4

HAR COL 2.3-2

Table 2.3.2-237
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1996 to February 28, 1997
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/96 – 02/28/97

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	58	65	49	62	49	54	47	75	50	63	90	74	61	62	56	62	977
4-7	215	221	211	196	141	147	171	165	274	234	269	279	153	120	160	195	3151
8-12	276	306	195	172	110	90	113	167	265	331	371	335	166	156	212	226	3491
13-18	53	62	37	24	9	3	11	27	65	144	129	128	35	88	97	60	972
19-24	1	2	1	0	0	0	0	0	2	14	27	36	0	9	5	6	103
>24	4	0	0	0	0	0	0	0	0	0	9	9	0	0	0	2	24
TOTAL	607	656	493	454	309	294	342	434	656	786	895	861	415	435	530	551	8718

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 29

HAR COL 2.3-2

Table 2.3.2-238
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1997 to February 28, 1998
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/97 – 02/28/98

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	46	47	42	38	33	33	56	45	42	71	89	50	57	52	61	43	805
4-7	206	215	174	152	118	133	152	149	203	254	300	309	191	155	163	175	3049
8-12	353	418	230	218	144	122	108	141	200	293	363	392	163	167	236	239	3787
13-18	105	71	41	26	13	11	6	16	43	99	97	107	34	67	89	80	905
19-24	7	11	8	2	5	0	0	1	3	5	6	12	11	9	20	15	115
>24	1	0	1	0	1	0	0	0	0	0	0	0	0	1	2	0	6
TOTAL	718	762	496	436	314	299	322	352	491	722	855	870	456	451	571	552	8667

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 31

HAR COL 2.3-2

Table 2.3.2-239
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March 1, 1998 to February 28, 1999
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/98 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	37	38	31	36	39	42	41	48	57	90	63	54	43	60	76	33	788
4-7	161	204	194	202	128	126	124	143	229	298	312	282	175	143	245	205	3171
8-12	284	413	246	235	142	87	105	116	200	326	397	408	159	137	171	236	3662
13-18	81	52	44	13	5	1	5	15	47	132	146	107	31	37	36	79	831
19-24	12	6	5	0	0	0	0	5	5	20	26	20	2	7	0	5	113
>24	8	1	0	0	0	0	0	0	0	0	0	7	0	1	0	0	17
TOTAL	583	714	520	486	314	256	275	327	538	866	944	878	410	385	528	558	8582

a) Data represent the number of hours a condition occurred.

b) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 17

HAR COL 2.3-2

Table 2.3.2-240
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Percentage of Occurrence[a])
Period of Record: March 1, 1994 to February 28, 1999
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: 03/01/94 – 02/28/99

	Wind Direction (Blowing From)(b)																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	0.49	0.56	0.42	0.49	0.48	0.49	0.50	0.58	0.59	0.83	0.93	0.70	0.57	0.62	0.71	0.53	9.50
4-7	2.10	2.42	2.22	2.13	1.61	1.56	1.65	1.73	2.87	3.29	3.31	3.27	1.82	1.48	2.05	2.04	35.57
8-12	3.52	4.49	2.88	2.43	1.61	1.17	1.26	1.61	2.75	3.92	4.39	4.11	1.79	1.73	2.24	2.49	42.40
13-18	1.03	0.81	0.60	0.28	0.11	0.07	0.10	0.23	0.66	1.36	1.37	1.31	0.40	0.71	0.84	0.96	10.84
19-24	0.08	0.12	0.04	0.01	0.01	0.00	0.01	0.04	0.09	0.14	0.20	0.23	0.07	0.11	0.08	0.08	1.29
>24	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.04	0.00	0.01	0.00	0.00	0.13
Total	7.26	8.40	6.17	5.35	3.81	3.29	3.53	4.20	6.97	9.54	10.22	9.67	4.66	4.65	5.93	6.10	99.73

a) Data represent the percentage of total observations that a condition occurred (excluding calm winds).

b) E = East, N = North, S = South, W = West

Notes: Missing Hours: 586

Number of Calm Hours: 117 (0.27%)

Total Observations: 43,238

HAR COL 2.3-2

Table 2.3.2-241
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: January (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: January (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	17	20	11	17	16	14	12	26	23	31	21	19	12	12	17	16	284
4-7	93	60	54	61	53	41	52	66	57	105	88	80	45	63	106	82	1106
8-12	147	108	86	109	40	53	81	74	77	128	114	167	104	106	126	127	1647
13-18	28	6	21	13	2	2	12	25	41	54	63	81	19	43	37	26	473
19-24	2	0	1	0	0	0	3	15	16	9	12	14	4	6	1	7	90
>24	1	0	0	0	0	0	0	0	1	1	2	1	0	1	0	0	7
TOTAL	288	194	173	200	111	110	160	206	215	328	300	362	184	231	287	258	3607

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 43

HAR COL 2.3-2

Table 2.3.2-242
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: February (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: February (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	13	12	16	11	9	18	13	23	22	21	26	19	16	12	21	29	281
4-7	76	76	61	60	21	42	47	32	50	83	84	85	44	39	78	76	954
8-12	120	118	79	68	37	43	64	63	71	136	126	131	66	75	95	107	1399
13-18	54	19	11	15	8	4	5	4	13	48	71	73	19	45	66	68	523
19-24	3	11	7	1	5	0	0	1	1	8	9	11	5	15	6	0	83
>24	0	0	0	0	1	0	0	0	0	0	1	0	0	1	0	0	3
TOTAL	266	236	174	155	81	107	129	123	157	296	317	319	150	187	266	280	3243

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 36

HAR COL 2.3-2

Table 2.3.2-243
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: March (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: March (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	11	13	7	12	3	15	12	4	8	12	10	8	8	8	17	9	157
4-7	54	101	79	89	48	51	58	39	70	103	82	65	33	38	61	64	1035
8-12	108	152	109	110	69	51	57	60	94	157	198	129	66	93	129	117	1699
13-18	52	40	30	15	2	9	9	17	53	103	54	54	23	71	66	80	678
19-24	0	0	2	0	0	0	0	0	8	18	14	35	8	6	19	7	117
>24	1	0	0	0	0	0	0	0	0	0	3	13	0	1	2	0	20
TOTAL	226	306	227	226	122	126	136	120	233	393	361	304	138	217	294	277	3706

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 1

HAR COL 2.3-2

Table 2.3.2-244
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: April (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: April (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	13	6	7	6	4	6	10	5	9	14	18	14	11	16	20	12	171
4-7	58	62	52	48	37	40	31	33	64	114	75	85	71	43	70	55	938
8-12	112	130	66	55	44	36	22	66	144	304	227	188	81	79	93	84	1731
13-18	39	22	23	6	4	2	4	12	61	167	92	94	28	38	44	52	688
19-24	2	0	0	0	0	0	0	0	1	7	22	12	4	8	3	8	67
>24	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	3
TOTAL	224	220	148	115	89	84	67	116	279	606	434	396	195	184	230	211	3598

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 2

HAR COL 2.3-2

Table 2.3.2-245
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: May (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: May (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	11	12	17	16	15	9	12	13	12	18	24	19	12	22	26	11	249
4-7	69	70	71	79	54	48	61	67	101	115	121	125	68	56	76	70	1251
8-12	157	179	85	84	66	29	42	45	123	186	192	213	57	49	73	75	1655
13-18	56	46	12	11	12	3	1	7	22	67	90	48	16	32	47	25	495
19-24	1	0	0	0	0	0	0	0	0	0	5	3	0	1	2	0	12
>24	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1
TOTAL	294	307	186	190	147	89	116	132	258	386	432	408	153	160	224	181	3663

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 5

HAR COL 2.3-2

Table 2.3.2-246
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: June (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: June (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	15	31	19	23	28	18	23	26	24	28	28	32	16	25	25	18	379
4-7	53	74	62	79	70	60	83	70	170	156	123	121	94	59	73	67	1414
8-12	67	114	93	93	77	49	53	110	131	120	158	145	58	38	56	48	1410
13-18	17	11	9	11	5	1	2	5	16	24	46	46	7	6	2	10	218
19-24	0	0	0	0	0	0	0	0	0	1	8	1	0	0	0	0	10
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	152	230	183	206	180	128	161	211	341	329	363	345	175	128	156	143	3431

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 3

HAR COL 2.3-2

Table 2.3.2-247
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: July (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: July (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	19	24	24	34	27	30	22	29	28	50	41	36	31	29	29	24	477
4-7	70	86	81	86	83	80	86	125	222	191	191	156	76	67	62	72	1734
8-12	62	62	76	64	51	46	40	63	198	187	180	160	44	31	24	28	1316
13-18	3	11	2	3	1	5	1	5	16	26	28	42	6	5	11	4	169
19-24	0	1	1	0	0	0	0	0	0	0	1	0	2	0	2	1	8
>24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	4
TOTAL	156	184	184	187	162	161	149	222	464	454	441	394	159	132	128	131	3708

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 3

HAR COL 2.3-2

Table 2.3.2-248
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: August (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: August (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	23	21	25	21	30	30	24	37	35	55	67	45	35	37	24	19	528
4-7	86	117	122	137	88	88	100	99	168	149	174	127	71	45	53	54	1678
8-12	103	166	137	134	108	74	52	56	81	101	121	92	25	14	32	53	1349
13-18	8	15	21	6	2	2	3	5	12	12	17	11	0	2	4	9	129
19-24	4	6	2	0	0	0	0	0	0	0	1	5	0	0	0	3	21
>24	7	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	8
TOTAL	231	326	307	298	228	194	179	197	296	317	380	280	131	98	113	138	3713

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 7

HAR COL 2.3-2

Table 2.3.2-249
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: September (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: September (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	27	27	21	24	22	16	28	28	28	52	46	29	29	21	26	10	434
4-7	105	128	129	69	74	70	60	75	124	129	138	161	78	43	60	60	1503
8-12	121	218	177	124	60	40	55	48	62	80	147	125	40	29	47	57	1430
13-18	19	24	41	5	1	0	4	3	6	11	20	21	5	6	10	21	197
19-24	4	0	3	0	0	0	0	0	0	0	4	2	1	1	0	1	16
≥24	0	0	0	0	0	0	0	0	0	0	5	0	0	0	0	0	5
TOTAL	276	397	371	222	157	126	147	154	220	272	360	338	153	100	143	149	3585

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 4

HAR COL 2.3-2

Table 2.3.2-250
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: October (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: October (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL
1-3	18	19	10	19	15	19	24	24	24	30	42	23	24	24	35	33	383
4-7	89	100	120	90	87	54	36	61	105	122	168	167	55	51	67	75	1447
8-12	177	265	150	79	61	34	22	59	96	73	115	125	44	47	66	92	1505
13-18	55	44	50	30	4	0	1	6	13	12	15	20	6	10	17	31	314
19-24	3	15	1	3	0	0	0	1	6	0	1	4	0	0	0	5	39
>24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2
TOTAL	344	443	331	221	167	107	83	151	244	237	341	339	129	132	185	236	3690

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 5

HAR COL 2.3-2

Table 2.3.2-251
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: November (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: November (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	16	17	6	17	14	16	20	15	25	26	46	30	26	33	26	14	347
4-7	70	85	48	61	32	54	67	47	64	111	119	123	73	64	86	87	1191
8-12	188	202	100	66	39	19	24	27	61	126	177	155	81	73	114	149	1601
13-18	60	50	26	1	2	2	3	3	22	38	52	36	33	22	13	34	397
19-24	5	8	0	0	0	0	0	0	6	12	7	5	2	7	0	0	52
>24	0	0	0	0	0	0	0	0	1	0	0	1	0	0	0	0	2
TOTAL	339	362	180	145	87	91	114	92	179	313	401	350	215	199	239	284	3590

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 1

HAR COL 2.3-2

Table 2.3.2-252
Joint Frequency Distribution of Wind Speed, Wind Direction, and Atmospheric Stability (Hours of Occurrence[a])
Period of Record: December (All Years)
Upper Wind Level, All Categories

Wind Level: Upper Level
Stability Category: ALL
Period of Record: December (All Years)[b]

	Wind Direction (Blowing From)[c]																
Speed (mph)	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**TOTAL**
1-3	30	38	19	14	23	21	17	21	19	23	33	28	26	29	42	33	416
4-7	87	87	81	64	48	45	34	36	47	46	67	121	80	70	93	121	1127
8-12	162	229	89	65	43	32	33	27	49	95	145	148	109	113	115	139	1593
13-18	55	61	13	4	3	0	0	9	11	25	44	41	13	27	47	54	407
19-24	11	12	0	0	0	0	0	0	0	4	1	7	3	3	0	3	44
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	345	427	202	147	117	98	84	93	126	193	290	345	231	242	297	350	3587

a) Data represent the number of hours a condition occurred.

b) Represents the Period of Record from March 1, 1994 to December 31, 1999

c) E = East, N = North, S = South, W = West

Notes: Number of Calm Hours: 7

HAR COL 2.3-2

Table 2.3.2-253
Mean Monthly and Annual Mean Temperatures (°F)
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: January 14, 1976 to December 31, 1978 and March 1, 1994 to February 28, 1999

Month	1976[a]	1977[a]	1978[a]	1994[b]	1995[b]	1996[b]	1997[b]	1998[b]	1999[b]	Average
January	40.1	29.3	34.3	NA	42.1	38.8	42.1	45.5	47.1	39.9
February	51.2	41.7	33.8	NA	41.0	44.8	48.0	48.0	46.2	44.3
March	55.7	55.5	48.0	52.3	52.2	46.9	56.1	52.5	NA	52.4
April	59.0	63.0	59.4	63.5	61.3	60.3	56.3	60.4	NA	60.4
May	66.5	68.4	65.2	64.2	68.0	69.1	65.7	69.6	NA	67.1
June	73.1	73.7	73.7	76.5	72.3	76.1	71.1	77.7	NA	74.3
July	77.9	81.3	76.2	78.4	78.8	78.1	78.8	79.3	NA	78.6
August	75.5	77.6	77.3	74.7	79.0	74.5	75.4	77.7	NA	76.5
September	69.9	72.4	72.1	68.0	69.1	70.3	70.9	73.8	NA	70.8
October	55.2	56.6	57.2	57.7	61.9	60.4	59.5	59.9	NA	58.6
November	43.5	52.7	54.0	52.9	46.8	45.9	48.6	51.8	NA	49.5
December	39.6	41.1	43.8	46.4	39.2	46.2	42.3	50.4	NA	43.6
Annual	58.9	59.4	57.9	NA	42.1	38.8	42.1	45.5	NA	49.0

a) Period of Record: January 14, 1976 to December 31, 1978 (Source: HNP FSAR)

b) Period of Record: March 1, 1994 to February 28, 1999

HAR COL 2.3-2

Table 2.3.2-254
Mean Monthly and Annual Maximum and Minimum Temperatures (°F)
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: January 14, 1976 to December 31, 1978

	1976		**1977**		**1978**		**Average**	
Month	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**
January	52.0	27.8	38.6	19.6	44.7	25.3	45.1	24.2
February	65.0	36.7	54.5	28.2	43.8	24.4	54.3	29.8
March	68.5	42.3	67.0	43.6	58.5	37.5	64.7	41.1
April	74.0	43.2	76.0	49.1	71.6	47.0	73.9	46.4
May	78.8	54.5	80.8	56.8	76.0	53.6	78.5	55.0
June	83.6	63.9	84.7	59.7	84.5	63.4	84.3	62.3
July	89.6	67.1	93.0	69.6	86.8	66.3	89.8	67.7
August	86.3	64.4	91.0	68.5	87.8	69.0	88.4	67.3
September	82.2	57.9	83.4	62.9	83.2	62.9	82.9	61.2
October	67.6	43.2	67.7	45.8	71.0	44.2	68.8	44.4
November	56.5	30.8	62.7	42.9	63.6	44.8	60.9	39.5
December	50.4	28.1	50.7	31.1	55.6	31.6	52.2	30.3
Annual	71.2	46.7	70.8	48.2	68.9	47.5	70.3	47.5

Source: HNP FSAR

HAR COL 2.3-2

Table 2.3.2-255
Summary of Mean Daily Temperatures (°F)

	Charlotte[a]		Greensboro[b]		Raleigh-Durham[c]	
Month	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**
January	51.0	31.3	48.1	28.2	50.5	29.9
February	54.9	33.5	51.6	30.5	53.9	31.8
March	62.6	39.9	60.0	37.2	61.7	38.3
April	72.3	48.7	70.3	45.9	72.0	46.7
May	79.5	57.4	77.6	55.1	79.0	55.4
June	85.9	65.5	84.2	63.3	85.7	63.7
July	89.0	69.4	87.5	67.6	88.8	68.1
August	87.8	68.3	85.8	66.5	87.3	67.0
September	81.7	62.0	79.9	59.5	81.4	60.6
October	72.2	50.3	70.1	47.2	71.9	48.4
November	62.3	40.2	60.1	37.8	62.6	39.0
December	53.1	33.2	50.4	30.4	53.0	32.0
Annual	71.0	50.0	68.8	47.4	70.7	48.4
Period of Record (yrs)	58	58	58	58	58	58

Sources:

a) Reference 2.3-203

b) Reference 2.3-204

c) Reference 2.3-205

HAR COL 2.3-2

Table 2.3.2-256
Comparison of Mean Dew-Point Temperatures (°F)

Month	Charlotte[a]	Greensboro[b]	Raleigh-Durham[c]	HNP (On-Site)[d]
January	28.7	26.1	28.4	22.0
February	31.7	28.9	31.2	26.3
March	36.5	34.4	36.9	38.6
April	44.7	43.2	45.3	44.1
May	55.6	54.2	53.3	55.9
June	63.8	62.8	64.6	64.4
July	67.9	67.5	66.1	67.6
August	67.1	66.5	64.8	68.8
September	60.9	60.3	59.0	63.4
October	50.5	48.7	50.9	47.3
November	40.7	38.5	40.6	39.7
December	31.4	29.2	29.5	30.5
Annual	48.3	46.7	47.6	47.4
Period of Record	21 years	20 years	21 years	3 years[d]

Sources:

a) Reference 2.3-203

b) Reference 2.3-204

c) Reference 2.3-205

d) January 14, 1976 – December 31, 1978 (HNP FSAR)

HAR COL 2.3-2

Table 2.3.2-257
Mean Dew-Point Temperatures (°F)
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: January 14, 1976 to December 31, 1978

Month	1976	1977	1978	Average
January	25.6	18.8	21.7	22.0
February	32.4	25.6	21.0	26.3
March	40.5	41.2	34.2	38.6
April	39.2	50.0	43.2	44.1
May	53.6	58.5	55.7	55.9
June	65.1	64.5	63.7	64.4
July	67.2	69.0	66.6	67.6
August	65.5	72.2	68.6	68.8
September	60.7	65.5	64.0	63.4
October	46.8	48.8	46.4	47.3
November	32.2	41.3	45.5	39.7
December	30.0	30.9	30.7	30.5
Annual	46.6	48.9	46.8	47.4

Source: HNP FSAR

HAR COL 2.3-2

Table 2.3.2-258
Summary of Diurnal Relative Humidity (%)

	Charlotte[a]				Greensboro[b]				Raleigh-Durham[c]			
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**
January	72	78	56	61	74	78	56	64	74	80	56	65
February	68	76	51	54	70	77	53	57	72	79	53	60
March	68	77	49	52	70	77	50	54	71	80	50	56
April	68	76	46	49	70	77	47	52	74	81	46	54
May	78	81	52	57	81	82	54	61	84	86	54	66
June	80	83	54	61	84	84	56	65	87	87	56	67
July	82	85	56	64	86	87	58	67	88	89	57	70
August	84	88	57	66	87	90	59	69	89	92	59	73
September	84	88	58	68	88	90	60	72	89	92	59	77
October	80	86	53	66	84	88	54	72	86	90	53	77
November	76	83	53	63	78	83	54	66	80	85	53	69
December	73	79	56	62	75	79	56	65	76	81	56	67
Annual	76	82	53	60	79	83	55	64	81	85	54	67
Period of Record (years)	30	30	30	30	30	30	30	30	30	30	30	30

Sources:

a) Reference 2.3-203

b) Reference 2.3-204

c) Reference 2.3-205

HAR COL 2.3-2

Table 2.3.2-259
Summary of Average Monthly and Annual Precipitation Measurements (in.)

Month	Charlotte[(a)]	Greensboro[(b)]	Raleigh-Durham[(c)]	HNP (On-Site)[(d)]
January	4.00	3.54	4.02	3.79
February	3.55	3.10	3.47	1.49
March	4.39	3.85	4.03	4.91
April	2.95	3.43	2.80	2.32
May	3.66	3.95	3.79	2.73
June	3.42	3.53	3.42	2.86
July	3.79	4.44	4.29	2.74
August	3.72	3.71	3.78	3.00
September	3.83	4.30	4.26	3.92
October	3.66	3.27	3.18	2.11
November	3.36	2.96	2.97	2.35
December	3.18	3.06	3.04	3.20
Annual	43.51	43.14	43.05	35.41
Period of Record	30 years	30 years	30 years	3 years

Sources:

a) Reference 2.3-203

b) Reference 2.3-204

c) Reference 2.3-205

d) January 14, 1976 – December 31, 1978 (HNP FSAR)

HAR COL 2.3-2

Table 2.3.2-260
Monthly and Annual Precipitation (in.)
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: January 14, 1976 to December 31, 1978 and March 1, 1994 to February 28, 1999

Month	1976[a]	1977[a]	1978[a]	1994[b]	1995[c]	1996[c]	1997[c]	1998[c]	1999[d]	Average
January	1.29	2.65	7.42	NA	4.09	2.82	2.06	3.22	2.56	3.26
February	1.15	1.57	1.74	NA	5.38	2.23	2.11	3.24	1.06	2.31
March	4.69	6.18	3.85	3.83	2.30	3.14	2.29	6.33	NA	4.08
April	0.43	2.17	4.36	0.58	0.83	3.48	4.51	3.10	NA	2.43
May	2.72	1.87	3.59	3.86	4.60	2.67	1.91	6.26	NA	3.43
June	2.74	0.77	5.08	3.22	5.80	3.11	2.87	1.35	NA	3.12
July	1.66	1.92	4.63	5.56	2.08	5.80	5.54	2.99	NA	3.77
August	1.76	3.78	3.47	3.75	3.02	2.31	0.47	0.79	NA	2.42
September	2.87	6.16	2.72	2.35	2.14	7.09	2.69	2.19	NA	3.53
October	1.26	4.17	0.91	4.90	10.07	3.70	2.25	1.57	NA	3.60
November	1.14	2.35	3.57	1.37	3.35	2.42	1.96	0.95	NA	2.14
December	3.66	3.08	2.85	1.11	1.09	1.98	1.83	0.60	NA	2.03
Annual	25.37	36.67	44.19	30.54	44.74	40.76	30.50	32.61	3.62	36.12

Sources:

a) Period of Record: January 14, 1976 to December 31, 1978 (HNP FSAR)

b) Period of Record: March 1, 1994 to December 31, 1994

c) Period of Record: January 1 to December 31 of indicated year

d) Period of Record: January 1, 1999 to February 28, 1999

HAR COL 2.3-2

Table 2.3.2-261
Average Number of Days of Fog Occurrence

Month	Charlotte[(a)]	Greensboro[(b)]	Raleigh-Durham[(c)]
January	3.7	4.7	3.5
February	2.8	3.3	2.7
March	2.4	2.8	2.1
April	1.2	1.6	1.4
May	1.0	1.9	2.3
June	1.0	1.4	1.9
July	1.1	1.8	2.6
August	1.4	2.3	3.1
September	2.0	3.0	3.3
October	1.8	2.6	3.3
November	3.0	3.2	3.1
December	3.8	3.8	3.2
Total	25.2	32.4	32.5
Period of Record	67 years	78 years	56 years

Sources:

a) Reference 2.3-203

b) Reference 2.3-204

c) Reference 2.3-205

HAR COL 2.3-2

Table 2.3.2-262
Frequency of Occurrence of Stability Class
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: January 14, 1976 to December 31, 1978 and March 1, 1994 to February 28, 1999

	Pasquill Stability Category (% Occurrence)						
Year	**A**	**B**	**C**	**D**	**E**	**F**	**G**
1976	8.5	4.8	5.2	24.3	23.7	12.6	21.0
1977	6.8	4.9	7.0	27.6	22.5	12.9	18.4
1978	4.5	3.2	4.1	29.0	26.6	12.7	19.9
1976-1978	6.5	4.3	5.4	27.0	24.3	12.7	19.8
3/1/94 – 2/28/99	1.5	2.9	5.4	32.0	28.6	10.7	18.9

Source: HNP FSAR

HAR COL 2.3-2

Table 2.3.2-263
Seasonal Frequency of Predicted Cooling Tower Plume Length (Hours per Year)
HNP Natural Draft Cooling Tower

		Wind Direction (Blowing From)																	
Season	**Distance from Site (km)**	**N**	**NNE**	**NE**	**ENE**	**E**	**ESE**	**SE**	**SSE**	**S**	**SSW**	**SW**	**WSW**	**W**	**WNW**	**NW**	**NNW**	**CALM**	**TOTAL[(a)]**
Winter	1	35	29	17	14	11	15	15	10	26	30	22	18	22	11	21	27	0	323
	2	4	4	3	2	1	2	2	1	3	8	4	8	8	4	5	4	0	63
	3	1	0	1	0	0	0	0	0	0	1	0	0	1	0	0	0	0	4
Spring	1	10	10	12	6	7	9	8	8	12	12	12	5	3	5	7	6	0	132
	2	2	1	1	0	0	0	1	1	1	1	2	0	0	0	1	0	0	10
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Summer	1	12	7	9	5	8	11	7	5	13	12	10	10	5	7	2	3	0	126
	2	0	0	1	0	1	2	1	0	1	2	2	1	1	1	0	0	0	13
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Fall	1	35	17	18	10	15	9	11	12	25	20	27	8	12	14	15	17	0	265
	2	5	2	1	1	2	1	1	3	3	2	5	1	4	3	2	2	0	38
	3	0	0	0	0	0	0	0	1	0	0	1	0	0	0	0	0	0	1
Annual	1	92	63	56	35	42	44	40	35	76	74	71	41	42	36	46	52	0	845
	2	11	7	7	3	4	6	6	5	8	13	12	11	13	8	8	7	0	129
	3	1	0	1	0	0	0	0	0	0	1	1	0	1	0	0	0	0	5

a) Observations of calm winds were assigned with wind direction reported during previous hour.

Source: HNP FSAR

HAR COL 2.3-3

Table 2.3.3-201
HNP/HAR Meteorological Monitoring Tower
Meteorological Sensor Elevations

Sensor	Approximate Elevation Above Tower Base (m)
Wind Speed and Direction	12 and 61
Dew Point	12
Solar Radiation	1.5
Ambient Temperature	12 and 61
Delta-Temperature[(a)]	12 and 61
Precipitation	1.5
Barometric Pressure	1.5

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

Source: HNP FSAR

HAR COL 2.3-3

Table 2.3.3-202
HNP/HAR Meteorological Monitoring Tower
Accuracy of Monitored Parameters

Monitored Parameter	Basis	Accuracy Criteria
Wind Direction 0 – 360 degrees (12 & 61m)	NRC Regulatory Guide 1.23, Revision 1	±5 degrees (°). Starting threshold <0.45 meter per second (m/s) (1 mph). Resolution to 1.0°.
Wind Speed 0 – 90 mph (12 & 61m)	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 -4°F - 104°F (12 & 61m)	NRC Regulatory Guide 1.23, Revision 1	±0.5°C (±0.9°F). Resolution to 0.1°C (0.1°F).
Differential Temperature (-108°F to +108°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 – 100%	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 millimeter (mm) (0.1 in.) of precipitation at a rate <50 millimeters per hour (mm/h) (<2 inches per hour [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] 26.06 – 32.58 “Hg	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 (min.). Resolution to ±1 min.

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

HAR COL 2.3-4

Table 2.3.4-201
Predicted HAR 2 and HAR 3 Chi/Q Values

Location and Averaging Period	AP1000 DCD Acceptance Criteria Chi/Q	HAR 2 and HAR 3 Maximum Predicted Chi/Q[(a)]
Exclusion Area Boundary		
0 – 2 hr	$\leq 5.1 \times 10^{-4}$ sec/m^3	4.7×10^{-4} sec/m^3
Low Population Zone		
0 – 8 hr	$\leq 2.2 \times 10^{-4}$ sec/m^3	9.1×10^{-5} sec/m^3
8 – 24 hr	$\leq 1.6 \times 10^{-4}$ sec/m^3	6.3×10^{-5} sec/m^3
24 – 96 hr	$\leq 1.0 \times 10^{-4}$ sec/m^3	2.8×10^{-5} sec/m^3
96 – 720 hr	$\leq 8.0 \times 10^{-5}$ sec/m^3	9.0×10^{-6} sec/m^3

a) Maximum predicted Chi/Q values occurred in the SSW sector for all averaging periods.

HAR COL 2.3-4

Table 2.3.4-202 (Sheet 1 of 4)
Meteorological Input Data for PAVAN Model
Joint Frequency Distribution by Hours
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: March 1994 to February 1999 (Lower Elevation)

Wind Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class A																
1-3	0	0	0	0	0	1	0	0	0	2	1	1	0	0	0	1
4-7	8	14	9	6	7	6	10	14	37	19	26	56	8	3	19	15
8-12	12	23	6	6	8	1	2	3	21	40	37	102	13	12	28	30
13-18	0	0	0	0	0	0	0	0	0	0	2	5	0	1	1	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Class B																
1-3	5	2	2	2	0	3	0	2	1	5	5	2	0	1	5	2
4-7	50	46	37	41	23	19	21	26	76	51	42	130	18	17	72	65
8-12	19	38	26	6	4	1	0	3	15	32	49	88	23	29	75	46
13-18	0	0	0	0	0	0	0	0	0	2	1	4	2	2	2	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

HAR COL 2.3-4

Table 2.3.4-202 (Sheet 2 of 4)
Meteorological Input Data for PAVAN Model
Joint Frequency Distribution by Hours
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: March 1994 to February 1999 (Lower Elevation)

Wind Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class C																
1-3	5	4	11	4	8	4	6	9	11	18	11	9	7	7	4	7
4-7	122	101	78	96	51	55	55	70	143	109	96	188	62	69	115	117
8-12	29	44	48	11	7	8	3	6	30	42	54	94	29	61	93	54
13-18	0	0	0	0	0	0	0	0	0	1	3	7	4	5	4	0
19-24	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Class D																
1-3	255	260	214	204	153	138	156	157	219	224	206	183	131	147	152	208
4-7	982	876	671	567	321	258	287	376	536	490	578	756	380	374	535	614
8-12	171	168	102	44	7	14	24	44	86	111	159	231	79	195	249	172
13-18	1	1	1	0	0	0	1	5	1	0	12	26	8	16	13	2
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

HAR COL 2.3-4

Table 2.3.4-202 (Sheet 3 of 4)
Meteorological Input Data for PAVAN Model
Joint Frequency Distribution by Hours
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: March 1994 to February 1999 (Lower Elevation)

Wind Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class E																
1-3	564	537	407	330	244	208	239	283	437	591	454	364	253	226	229	344
4-7	572	388	255	192	125	142	229	405	575	638	500	449	196	201	230	381
8-12	59	65	33	16	11	13	26	50	129	68	62	81	25	68	72	72
13-18	21	4	0	2	4	0	2	5	5	2	2	9	0	3	1	4
19-24	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Class F																
1-3	513	458	369	266	189	173	131	165	195	255	227	224	215	220	188	270
4-7	78	18	7	11	14	10	19	23	38	27	36	75	23	24	24	58
8-12	0	1	0	0	0	0	0	0	0	0	0	0	0	1	1	1
13-18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

HAR COL 2.3-4

Table 2.3.4-202 (Sheet 4 of 4)
Meteorological Input Data for PAVAN Model
Joint Frequency Distribution by Hours
Shearon Harris Nuclear Power Plant Meteorological Monitoring Station
Period of Record: March 1994 to February 1999 (Lower Elevation)

Wind Speed (mph)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
Class G																
1-3	1442	1777	1139	500	305	180	123	91	100	142	172	232	232	372	406	702
4-7	23	2	2	1	1	5	5	3	0	3	0	6	2	4	1	7
8-12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13-18	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19-24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
>24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

HAR COL 2.3-4

Table 2.3.4-203
0- to 2-hour 5th Percentile Exclusion Area Boundary Chi/Q Values[a]
for HAR 2 and HAR 3

Downwind Sector	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	1600	5250	4.48E-04	4.48E-04
SSW	1600	5250	4.70E-04	4.70E-04
SW	1600	5250	4.32E-04	4.32E-04
WSW	1600	5250	3.37E-04	3.37E-04
W	1245	4085	3.10E-04	3.10E-04
WNW	1245	4085	2.20E-04	2.20E-04
NW	1245	4085	1.56E-04	1.56E-04
NNW	1245	4085	1.48E-04	1.48E-04
N	1245	4085	1.68E-04	1.68E-04
NNE	1245	4085	1.90E-04	1.90E-04
NE	1245	4085	2.12E-04	2.12E-04
ENE	1245	4085	2.48E-04	2.48E-04
E	1245	4085	2.58E-04	2.58E-04
ESE	1600	5250	3.15E-04	3.15E-04
SE	1600	5250	3.14E-04	3.14E-04
SSE	1600	5250	3.97E-04	3.97E-04
MAX Chi/Q			4.70E-04	4.70E-04

a) Predictions based on PAVAN model as described in Subsection 2.3.4.2 of this FSAR.

HAR COL 2.3-4

Table 2.3.4-204
0- to 30-day 5th Percentile Low Population Zone Chi/Q Values[(a)] for HAR 2 and HAR 3

Downwind Sector	Distance (m)	Distance (mi.)	0-8 hr. Chi/Q With Wake sec/m^3	8-24 hr. Chi/Q With Wake sec/m^3	1-4 d Chi/Q With Wake sec/m^3	4-30 d Chi/Q With Wake sec/m^3	0-8 hr. Chi/Q Without Wake sec/m^3	8-24 hr. Chi/Q Without Wake sec/m^3	1-4 d Chi/Q Without Wake sec/m^3	4-30 d Chi/Q Without Wake sec/m^3
S	4830	3	6.61E-05	4.48E-05	1.93E-05	5.75E-06	6.83E-05	4.70E-05	2.10E-05	6.57E-06
SSW	4830	3	7.21E-05	4.90E-05	2.11E-05	6.33E-06	7.46E-05	5.16E-05	2.31E-05	7.30E-06
SW	4830	3	5.81E-05	3.88E-05	1.61E-05	4.58E-06	6.01E-05	4.08E-05	1.76E-05	5.25E-06
WSW	4830	3	3.88E-05	2.54E-05	1.01E-05	2.70E-06	4.00E-05	2.65E-05	1.09E-05	3.05E-06
W	4830	3	2.98E-05	1.92E-05	7.38E-06	1.87E-06	3.06E-05	2.00E-05	7.93E-06	2.10E-06
WNW	4830	3	2.24E-05	1.44E-05	5.51E-06	1.39E-06	2.29E-05	1.49E-05	5.88E-06	1.54E-06
NW	4830	3	1.66E-05	1.09E-05	4.34E-06	1.16E-06	1.70E-05	1.12E-05	4.58E-06	1.26E-06
NNW	4830	3	1.60E-05	1.06E-05	4.36E-06	1.21E-06	1.63E-05	1.09E-05	4.56E-06	1.31E-06
N	4830	3	1.79E-05	1.21E-05	5.17E-06	1.52E-06	1.82E-05	1.24E-05	5.39E-06	1.63E-06
NNE	4830	3	2.21E-05	1.49E-05	6.36E-06	1.87E-06	2.24E-05	1.53E-05	6.65E-06	2.01E-06
NE	4830	3	2.36E-05	1.57E-05	6.52E-06	1.84E-06	2.41E-05	1.62E-05	6.85E-06	1.99E-06
ENE	4830	3	2.76E-05	1.82E-05	7.41E-06	2.04E-06	2.82E-05	1.88E-05	7.83E-06	2.23E-06
E	4830	3	2.65E-05	1.71E-05	6.66E-06	1.72E-06	2.71E-05	1.78E-05	7.11E-06	1.91E-06
ESE	4830	3	3.32E-05	2.15E-05	8.38E-06	2.17E-06	3.41E-05	2.24E-05	9.02E-06	2.44E-06
SE	4830	3	3.46E-05	2.25E-05	8.78E-06	2.28E-06	3.56E-05	2.34E-05	9.45E-06	2.57E-06
SSE	4830	3	4.60E-05	3.04E-05	1.23E-05	3.38E-06	4.75E-05	3.18E-05	1.33E-05	3.83E-06
MAX Chi/Q							7.46E-05	5.16E-05	2.31E-05	7.30E-06

a) Predictions based on PAVAN model as described in Subsection 2.3.4.2 of this FSAR.

HAR COL 2.3-4

Table 2.3.4-205
0- to 2-hour 50th Percentile EAB Chi/Q Values for HAR 2 and HAR 3

Time Period	Chi/Q (sec/m^3)	Source
0 - 2 hr.	6.16 E -05	PAVAN Model

0- to 30-day 50th Percentile LPZ Chi/Q Values for HAR 2 and HAR 3

Time Period	Chi/Q (sec/m^3)	Source
0 - 2 hr.	1.38E-05	PAVAN Model
0 - 8 hr.	9.70E-06	Interpolation
8 - 24 hr.	8.15E-06	Interpolation
1 - 4 days	5.70E-06	Interpolation
4 - 30 days	3.30E-06	Interpolation
Annual Average	1.78E-06	PAVAN Model

HAR 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 HAR Units 2 & 3

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

	Plant Vent or PCS Air Diffuser[b]	Plant Vent	PCS Air Diffuser	Ground Level Containment Release Points[c,h]	Ground Level Containment Release Points	PORV and Safety Valve Releases[d]	PORV and Safety Valve Releases	Condenser Air Removal Stack[g]	Condenser Air Removal Stack	Steam Line Break Releases	Steam Vent	Fuel Handling Area[e]	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	**HAR**	**HAR**	**DCD**	**HAR 2 and HAR 3**	**DCD**	**HAR**	**DCD**	**HAR 2 and HAR 3**	**DCD**	**HAR**	**DCD**	**HAR**	**HAR**
0 - 2 hours	**3.0E-03**	2.0E-03	1.6E-03	**6.0E-03**	4.2E-03	**2.0E-02**	1.2E-02	**6.0E-03**	1.5E-03	**2.4E-02**	1.4E-02	**6.0E-03**	1.4E-03	1.1E-03
2 - 8 hours	**2.5E-03**	1.4E-03	1.1E-03	**3.6E-03**	3.2E-03	**1.8E-02**	1.0E-02	**4.0E-03**	1.2E-03	**2.0E-02**	1.1E-02	**4.0E-03**	1.0E-03	8.3E-04
8 - 24 hours	**1.0E-03**	6.1E-04	4.9E-04	**1.4E-03**	1.2E-03	**7.0E-03**	4.2E-03	**2.0E-03**	5.1E-04	**7.5E-03**	4.6E-03	**2.0E-03**	4.5E-04	3.7E-04
1 - 4 days	**8.0E-04**	4.5E-04	3.4E-04	**1.8E-03**	1.2E-03	**5.0E-03**	3.1E-03	**1.5E-03**	3.4E-04	**5.5E-03**	3.4E-03	**1.5E-03**	3.2E-04	2.6E-04
4 - 30 days	**6.0E-04**	3.5E-04	3.0E-04	**1.5E-03**	1.0E-03	**4.5E-03**	2.6E-03	**1.0E-03**	2.6E-04	**5.0E-03**	2.8E-03	**1.0E-03**	2.5E-04	2.1E-04

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

	Plant Vent or PCS Air Diffuser[b]	Plant Vent	PCS Air Diffuser	Ground Level Containment Release Points[c]	Ground Level Containment Release Points	PORV and Safety Valve Releases[d]	PORV and Safety Valve Releases	Condenser Air Removal Stack[g]	Condenser Air Removal Stack	Steam Line Break Releases	Steam Vent	Fuel Handling Area[e]	Fuel Handling Area Blowout Panel	Radwaste Building Truck Staging Area Door
Release Time	**DCD**	**HAR**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**DCD**	**HAR**	**HAR**
0 - 2 hours	**1.0E-03**	4.2E-04	4.3E-04	**1.0E-03**	3.6E-04	**4.0E-03**	8.6E-04	**2.0E-02**	3.4E-03	**4.0E-03**	8.4E-04	**6.0E-03**	3.4E-04	3.3E-04
2 - 8 hours	**7.5E-04**	3.2E-04	3.1E-04	**7.5E-04**	2.9E-04	**3.2E-03**	6.2E-04	**1.8E-02**	2.6E-03	**3.2E-03**	5.9E-04	**4.0E-03**	2.5E-04	2.4E-04
8 - 24 hours	**3.5E-04**	1.4E-04	1.4E-04	**3.5E-04**	1.4E-04	**1.2E-03**	2.6E-04	**7.0E-03**	1.2E-03	**1.2E-03**	2.5E-04	**2.0E-03**	1.1E-04	1.1E-04
1 - 4 days	**2.8E-04**	1.1E-04	1.0E-04	**2.8E-04**	1.0E-04	**1.0E-03**	2.0E-04	**5.0E-03**	7.1E-04	**1.0E-03**	1.9E-04	**1.5E-03**	8.2E-05	8.0E-05
4 - 30 days	**2.5E-04**	8.3E-05	8.4E-05	**2.5E-04**	8.9E-05	**8.0E-04**	1.6E-04	**4.5E-03**	6.0E-04	**8.0E-04**	1.5E-04	**1.0E-03**	6.7E-05	6.5E-05

HAR 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 HAR Units 2 & 3

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

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

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

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

h) The LOCA dose analysis models the ground level containment release point HVAC intake atmospheric dispersion factors. Other analyses model more conservative values.

HAR 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	261	262
PCS Air Diffuser	291	273
Containment Shell (Diffuse Area Source)	284	267
Fuel Building Blowout Panel	244	254
Radwaste Building Truck Staging Area Door	235	249
Steam Vent	336	277
PORV/Safety Valves	346	279
Condenser Air Removal Stack	75	310

HAR COL 2.3-5

Table 2.3.5-201 (Sheet 1 of 3)
Long-Term Chi/Q (in sec/m³) Calculations for Routine Releases for HAR 2 and HAR 3

	Exclusion Area Boundary		Low Population Zone[a]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q
N	1245	2.30E-06	4959	3.10E-07	2973	6.50E-07	7472	1.80E-07	2974	6.50E-07	2974	6.50E-07
NNE	1245	2.80E-06	4925	3.90E-07	4297	4.70E-07	7630	2.10E-07	2668	9.30E-07	2668	9.30E-07
NE	1245	2.60E-06	4876	3.70E-07	7851	1.90E-07	7851	1.90E-07	7851	1.90E-07	7851	1.90E-07
ENE	1245	2.80E-06	4832	4.10E-07	8100	2.00E-07	8100	2.00E-07	8100	2.00E-07	8100	2.00E-07
E	1245	2.10E-06	4887	3.10E-07	8338	1.50E-07	8338	1.50E-07	2994	6.10E-07	2994	6.10E-07
ESE	1600	1.80E-06	4934	4.00E-07	8530	1.90E-07	8530	1.90E-07	7887	2.10E-07	7887	2.10E-07
SE	1600	1.90E-06	4964	4.20E-07	8651	2.00E-07	8651	2.00E-07	7202	2.50E-07	4790	4.40E-07
SSE	1600	2.90E-06	4973	6.60E-07	8686	3.20E-07	8686	3.20E-07	7398	3.90E-07	7398	3.90E-07
S	1600	5.50E-06	4959	1.30E-06	8631	6.20E-07	8631	6.20E-07	8631	6.20E-07	8631	6.20E-07
SSW	1600	6.00E-06	4925	1.40E-06	8492	7.10E-07	8492	7.10E-07	6565	9.80E-07	8492	7.10E-07
SW	1600	4.10E-06	4876	9.60E-07	8287	4.90E-07	8287	4.90E-07	4922	9.50E-07	4922	9.50E-07
WSW	1600	2.30E-06	4832	5.30E-07	8044	2.70E-07	8044	2.70E-07	7242	3.10E-07	7242	3.10E-07
W	1245	2.20E-06	4887	3.40E-07	7798	1.80E-07	7798	1.80E-07	4754	3.50E-07	4754	3.50E-07
WNW	1245	1.70E-06	4934	2.50E-07	7588	1.40E-07	7588	1.40E-07	7588	1.40E-07	7588	1.40E-07
NW	1245	1.50E-06	4964	2.10E-07	7449	1.20E-07	7449	1.20E-07	7449	1.20E-07	7449	1.20E-07
NNW	1245	1.70E-06	4973	2.30E-07	7408	1.30E-07	7408	1.30E-07	2580	5.90E-07	2580	5.90E-07

HAR COL 2.3-5

Table 2.3.5-201 (Sheet 2 of 3)
Long-Term Chi/Q (in sec/m³) Calculations for Routine Releases for HAR 2 and HAR 3

	Nearest Residence		Downwind Distance (mi.)									
Downwind Sector	Distance (m)	Chi/Q	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2974	6.50E-07	4.47E-06	2.41E-06	1.57E-06	8.74E-07	5.79E-07	4.21E-07	3.25E-07	2.62E-07	2.17E-07	1.84E-07
NNE	2668	9.30E-07	5.45E-06	2.94E-06	1.91E-06	1.07E-06	7.11E-07	5.19E-07	4.02E-07	3.24E-07	2.69E-07	2.29E-07
NE	3534	5.80E-07	5.09E-06	2.72E-06	1.76E-06	9.93E-07	6.62E-07	4.85E-07	3.76E-07	3.04E-07	2.53E-07	2.15E-07
ENE	8100	2.00E-07	5.52E-06	2.90E-06	1.88E-06	1.06E-06	7.11E-07	5.21E-07	4.06E-07	3.28E-07	2.74E-07	2.33E-07
E	2680	7.10E-07	4.21E-06	2.20E-06	1.42E-06	8.18E-07	5.53E-07	4.09E-07	3.20E-07	2.60E-07	2.18E-07	1.86E-07
ESE	4676	4.30E-07	5.35E-06	2.74E-06	1.77E-06	1.03E-06	6.99E-07	5.20E-07	4.08E-07	3.33E-07	2.80E-07	2.40E-07
SE	4790	4.40E-07	5.73E-06	2.92E-06	1.88E-06	1.09E-06	7.40E-07	5.50E-07	4.31E-07	3.52E-07	2.95E-07	2.53E-07
SSE	7398	3.90E-07	8.96E-06	4.55E-06	2.92E-06	1.71E-06	1.17E-06	8.69E-07	6.84E-07	5.60E-07	4.70E-07	4.04E-07
S	8631	6.20E-07	1.68E-05	8.44E-06	5.43E-06	3.20E-06	2.20E-06	1.64E-06	1.30E-06	1.06E-06	8.95E-07	7.70E-07
SSW	6565	9.80E-07	1.86E-05	9.28E-06	5.95E-06	3.53E-06	2.43E-06	1.83E-06	1.45E-06	1.19E-06	1.00E-06	8.63E-07
SW	4922	9.50E-07	1.26E-05	6.32E-06	4.06E-06	2.40E-06	1.65E-06	1.24E-06	9.76E-07	8.01E-07	6.75E-07	5.81E-07
WSW	7242	3.10E-07	6.93E-06	3.54E-06	2.28E-06	1.33E-06	9.05E-07	6.73E-07	5.29E-07	4.32E-07	3.63E-07	3.11E-07
W	4594	3.70E-07	4.50E-06	2.32E-06	1.50E-06	8.67E-07	5.90E-07	4.38E-07	3.44E-07	2.81E-07	2.35E-07	2.02E-07
WNW	3577	3.80E-07	3.33E-06	1.74E-06	1.13E-06	6.47E-07	4.37E-07	3.23E-07	2.53E-07	2.06E-07	1.72E-07	1.47E-07
NW	3268	3.80E-07	3.00E-06	1.59E-06	1.03E-06	5.82E-07	3.90E-07	2.86E-07	2.23E-07	1.80E-07	1.50E-07	1.28E-07
NNW	1936	8.80E-07	3.29E-06	1.77E-06	1.15E-06	6.46E-07	4.30E-07	3.14E-07	2.43E-07	1.96E-07	1.63E-07	1.39E-07

HAR COL 2.3-5

Table 2.3.5-201 (Sheet 3 of 3)
Long-Term Chi/Q (in sec/m^3) Calculations for Routine Releases for HAR 2 and HAR 3

Downwind Sector	Downwind Distance (mi.)										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	1.59E-07	9.11E-08	6.16E-08	3.56E-08	2.43E-08	1.81E-08	1.42E-08	1.16E-08	9.77E-09	8.38E-09	7.31E-09
NNE	1.98E-07	1.14E-07	7.71E-08	4.49E-08	3.07E-08	2.29E-08	1.80E-08	1.48E-08	1.24E-08	1.07E-08	9.31E-09
NE	1.86E-07	1.08E-07	7.31E-08	4.27E-08	2.93E-08	2.19E-08	1.73E-08	1.42E-08	1.20E-08	1.03E-08	8.99E-09
ENE	2.02E-07	1.17E-07	8.01E-08	4.71E-08	3.24E-08	2.43E-08	1.93E-08	1.59E-08	1.34E-08	1.15E-08	1.01E-08
E	1.62E-07	9.50E-08	6.53E-08	3.88E-08	2.69E-08	2.03E-08	1.61E-08	1.33E-08	1.13E-08	9.72E-09	8.53E-09
ESE	2.09E-07	1.24E-07	8.55E-08	5.11E-08	3.56E-08	2.70E-08	2.15E-08	1.78E-08	1.51E-08	1.31E-08	1.15E-08
SE	2.20E-07	1.30E-07	9.01E-08	5.39E-08	3.76E-08	2.85E-08	2.27E-08	1.88E-08	1.59E-08	1.38E-08	1.21E-08
SSE	3.52E-07	2.10E-07	1.45E-07	8.73E-08	6.10E-08	4.63E-08	3.70E-08	3.06E-08	2.60E-08	2.26E-08	1.99E-08
S	6.73E-07	4.02E-07	2.80E-07	1.69E-07	1.18E-07	9.00E-08	7.20E-08	5.97E-08	5.08E-08	4.41E-08	3.88E-08
SSW	7.55E-07	4.53E-07	3.17E-07	1.92E-07	1.35E-07	1.03E-07	8.23E-08	6.84E-08	5.82E-08	5.05E-08	4.45E-08
SW	5.08E-07	3.04E-07	2.12E-07	1.28E-07	8.99E-08	6.84E-08	5.48E-08	4.55E-08	3.87E-08	3.36E-08	2.96E-08
WSW	2.71E-07	1.61E-07	1.11E-07	6.65E-08	4.64E-08	3.51E-08	2.80E-08	2.32E-08	1.97E-08	1.70E-08	1.50E-08
W	1.76E-07	1.04E-07	7.18E-08	4.29E-08	2.99E-08	2.26E-08	1.80E-08	1.49E-08	1.26E-08	1.09E-08	9.60E-09
WNW	1.28E-07	7.50E-08	5.15E-08	3.06E-08	2.12E-08	1.60E-08	1.27E-08	1.05E-08	8.86E-09	7.65E-09	6.71E-09
NW	1.11E-07	6.44E-08	4.40E-08	2.58E-08	1.78E-08	1.33E-08	1.06E-08	8.66E-09	7.31E-09	6.30E-09	5.51E-09
NNW	1.20E-07	6.90E-08	4.68E-08	2.72E-08	1.86E-08	1.39E-08	1.10E-08	8.97E-09	7.55E-09	6.49E-09	5.66E-09

Notes:

a) The reported distance of the Low Population Zone (LPZ) is measured from the centerpoint of HAR 2 and HAR 3 to the outermost boundary of the LPZ.

Wind Reference Level: 12 m

Stability Type: ΔT (61 – 12 m)

Release Type: Ground Level: 12 m

Building Height/Cross Section: 43.9 m/2730 m^2

HAR COL 2.3-5

Table 2.3.5-202 (Sheet 1 of 3)
Long-Term Average D/Q (in m^{-2}) Calculations for Routine Releases for HAR 2 and HAR 3

	Exclusion Area Boundary		Low Population Zone[a]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector	Distance (m)	D/Q	Distance (m)	D/Q	Distance (m)	D/Q	Distance (m)	D/Q	Distance (m)	D/Q	Distance (m)	D/Q
N	1245	5.90E-09	4959	5.40E-10	2973	1.30E-09	7472	2.60E-10	2974	1.30E-09	2974	1.30E-09
NNE	1245	6.40E-09	4925	5.90E-10	4297	7.50E-10	7630	2.70E-10	2668	1.70E-09	2668	1.70E-09
NE	1245	6.10E-09	4876	5.70E-10	7851	2.40E-10	7851	2.40E-10	7851	2.40E-10	7851	2.40E-10
ENE	1245	7.40E-09	4832	7.10E-10	8100	2.80E-10	8100	2.80E-10	8100	2.80E-10	8100	2.80E-10
E	1245	3.80E-09	4887	3.60E-10	8338	1.40E-10	8338	1.40E-10	2994	8.50E-10	2994	8.50E-10
ESE	1600	3.00E-09	4934	4.30E-10	8530	1.60E-10	8530	1.60E-10	7887	1.80E-10	7887	1.80E-10
SE	1600	3.66E-09	4964	5.20E-10	8651	1.90E-10	8651	1.90E-10	7202	2.70E-10	4790	5.50E-10
SSE	1600	4.63E-09	4973	6.50E-10	8686	2.40E-10	8686	2.40E-10	7398	3.20E-10	7398	3.20E-10
S	1600	7.26E-09	4959	1.00E-09	8631	3.80E-10	8631	3.80E-10	8631	3.80E-10	8631	3.80E-10
SSW	1600	7.15E-09	4925	1.00E-09	8492	3.90E-10	8492	3.90E-10	6565	6.10E-10	8492	3.90E-10
SW	1600	5.05E-09	4876	7.40E-10	8287	2.80E-10	8287	2.80E-10	4922	7.20E-10	4922	7.20E-10
WSW	1600	3.37E-09	4832	5.00E-10	8044	2.00E-10	8044	2.00E-10	7242	2.40E-10	7242	2.40E-10
W	1245	3.40E-09	4887	3.10E-10	7798	1.40E-10	7798	1.40E-10	4754	3.30E-10	4754	3.30E-10
WNW	1245	2.80E-09	4934	2.60E-10	7588	1.20E-10	7588	1.20E-10	7588	1.20E-10	7588	1.20E-10
NW	1245	3.00E-09	4964	2.70E-10	7449	1.30E-10	7449	1.30E-10	7449	1.30E-10	7449	1.30E-10
NNW	1245	3.90E-09	4973	3.50E-10	7408	1.70E-10	7408	1.70E-10	2580	1.10E-09	2580	1.10E-09

HAR COL 2.3-5

Table 2.3.5-202 (Sheet 2 of 3)
Long-Term Average D/Q (in m^{-2}) Calculations for Routine Releases for HAR 2 and HAR 3

	Nearest Residence		Downwind Distance (mi.)									
Downwind Sector	**Distance (m)**	**D/Q**	**0.5**	**0.75**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**
N	2974	1.30E-9	1.21E-08	6.22E-09	3.82E-09	1.90E-09	1.16E-09	7.81E-10	5.66E-10	4.30E-10	3.39E-10	2.74E-10
NNE	2668	1.70E-9	1.31E-08	6.74E-09	4.14E-09	2.06E-09	1.25E-09	8.46E-10	6.13E-10	4.66E-10	3.67E-10	2.97E-10
NE	3534	1.00E-9	1.25E-08	6.42E-09	3.94E-09	1.97E-09	1.19E-09	8.06E-10	5.84E-10	4.44E-10	3.50E-10	2.83E-10
ENE	8100	2.80E-10	1.52E-08	7.80E-09	4.79E-09	2.39E-09	1.45E-09	9.79E-10	7.09E-10	5.39E-10	4.25E-10	3.44E-10
E	2680	1.00E-9	7.88E-09	4.05E-09	2.49E-09	1.24E-09	7.51E-10	5.08E-10	3.68E-10	2.80E-10	2.21E-10	1.79E-10
ESE	4676	4.70E-10	9.52E-09	4.89E-09	3.00E-09	1.50E-09	9.07E-10	6.13E-10	4.44E-10	3.38E-10	2.66E-10	2.16E-10
SE	4790	5.50E-10	1.16E-08	5.96E-09	3.66E-09	1.83E-09	1.11E-09	7.48E-10	5.42E-10	4.12E-10	3.25E-10	2.63E-10
SSE	7398	3.20E-10	1.47E-08	7.55E-09	4.64E-09	2.31E-09	1.40E-09	9.48E-10	6.87E-10	5.22E-10	4.12E-10	3.33E-10
S	8631	3.80E-10	2.30E-08	1.18E-08	7.26E-09	3.62E-09	2.20E-09	1.49E-09	1.08E-09	8.18E-10	6.44E-10	5.22E-10
SSW	6565	6.10E-10	2.27E-08	1.17E-08	7.15E-09	3.57E-09	2.16E-09	1.46E-09	1.06E-09	8.06E-10	6.35E-10	5.14E-10
SW	4922	7.20E-10	1.60E-08	8.23E-09	5.05E-09	2.52E-09	1.53E-09	1.03E-09	7.48E-10	5.69E-10	4.48E-10	3.63E-10
WSW	7242	2.40E-10	1.07E-08	5.50E-09	3.37E-09	1.68E-09	1.02E-09	6.90E-10	5.00E-10	3.80E-10	2.99E-10	2.42E-10
W	4594	3.50E-10	6.88E-09	3.53E-09	2.17E-09	1.08E-09	6.56E-10	4.43E-10	3.21E-10	2.44E-10	1.93E-10	1.56E-10
WNW	3577	4.50E-10	5.72E-09	2.94E-09	1.80E-09	9.00E-10	5.46E-10	3.69E-10	2.67E-10	2.03E-10	1.60E-10	1.30E-10
NW	3268	5.70E-10	6.15E-09	3.16E-09	1.94E-09	9.66E-10	5.86E-10	3.96E-10	2.87E-10	2.18E-10	1.72E-10	1.39E-10
NNW	1936	1.80E-09	7.96E-09	4.08E-09	2.51E-09	1.25E-09	7.58E-10	5.13E-10	3.72E-10	2.83E-10	2.23E-10	1.80E-10

HAR COL 2.3-5

Table 2.3.5-202 (Sheet 3 of 3)
Long-Term Average D/Q (in m^{-2}) Calculations for Routine Releases for HAR 2 and HAR 3

Downwind Sector	Downward Distance (mi.)										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	2.27E-10	1.11E-10	6.98E-11	3.53E-11	2.14E-11	1.43E-11	1.03E-11	7.70E-12	5.99E-12	4.78E-12	3.90E-12
NNE	2.46E-10	1.21E-10	7.56E-11	3.82E-11	2.31E-11	1.55E-11	1.11E-11	8.35E-12	6.49E-12	5.18E-12	4.23E-12
NE	2.34E-10	1.15E-10	7.21E-11	3.64E-11	2.20E-11	1.48E-11	1.06E-11	7.95E-12	6.18E-12	4.94E-12	4.03E-12
ENE	2.85E-10	1.39E-10	8.75E-11	4.42E-11	2.68E-11	1.79E-11	1.29E-11	9.66E-12	7.51E-12	6.00E-12	4.89E-12
E	1.48E-10	7.24E-11	4.54E-11	2.30E-11	1.39E-11	9.31E-12	6.67E-12	5.01E-12	3.90E-12	3.11E-12	2.54E-12
ESE	1.78E-10	8.74E-11	5.48E-11	2.77E-11	1.68E-11	1.12E-11	8.06E-12	6.05E-12	4.70E-12	3.76E-12	3.07E-12
SE	2.18E-10	1.07E-10	6.69E-11	3.38E-11	2.05E-11	1.37E-11	9.83E-12	7.38E-12	5.74E-12	4.59E-12	3.74E-12
SSE	2.76E-10	1.35E-10	8.47E-11	4.28E-11	2.59E-11	1.74E-11	1.25E-11	9.35E-12	7.27E-12	5.81E-12	4.74E-12
S	4.32E-10	2.12E-10	1.33E-10	6.71E-11	4.06E-11	2.72E-11	1.95E-11	1.46E-11	1.14E-11	9.10E-12	7.42E-12
SSW	4.25E-10	2.08E-10	1.31E-10	6.61E-11	4.00E-11	2.68E-11	1.92E-11	1.44E-11	1.12E-11	8.96E-12	7.31E-12
SW	3.00E-10	1.47E-10	9.23E-11	4.67E-11	2.82E-11	1.89E-11	1.36E-11	1.02E-11	7.92E-12	6.33E-12	5.16E-12
WSW	2.01E-10	9.83E-11	6.17E-11	3.12E-11	1.89E-11	1.27E-11	9.06E-12	6.81E-12	5.29E-12	4.23E-12	3.45E-12
W	1.29E-10	6.32E-11	3.96E-11	2.00E-11	1.21E-11	8.13E-12	5.83E-12	4.37E-12	3.40E-12	2.72E-12	2.22E-12
WNW	1.07E-10	5.26E-11	3.30E-11	1.67E-11	1.01E-11	6.76E-12	4.85E-12	3.64E-12	2.83E-12	2.26E-12	1.85E-12
NW	1.15E-10	5.65E-11	3.54E-11	1.79E-11	1.08E-11	7.27E-12	5.21E-12	3.91E-12	3.04E-12	2.43E-12	1.98E-12
NNW	1.49E-10	7.30E-11	4.58E-11	2.32E-11	1.40E-11	9.40E-12	6.74E-12	5.06E-12	3.93E-12	3.14E-12	2.56E-12

Notes:

a) The reported distance of the Low Population Zone (LPZ) is measured from the centerpoint of HAR 2 and HAR 3 to the outermost boundary of the LPZ.

Wind Reference Level: 12 m

Stability Type: ΔT (61 – 12 m)

Release Type: Ground Level: 12 m

Building Height/Cross Section: 43.9 m/2730 m^2

HAR COL 2.3-5

Table 2.3.5-203 (Sheet 1 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (2.26-Day Decay) for Routine Releases for HAR 2 and HAR 3

	Exclusion Area Boundary		Low Population Zone[a]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q
N	1245	2.30E-06	4959	3.10E-07	2973	6.40E-07	7472	1.70E-07	2974	6.40E-07	2974	6.40E-07
NNE	1245	2.80E-06	4925	3.80E-07	4297	4.60E-07	7630	2.00E-07	2668	9.10E-07	2668	9.10E-07
NE	1245	2.60E-06	4876	3.60E-07	7851	1.80E-07	7851	1.80E-07	7851	1.80E-07	7851	1.80E-07
ENE	1245	2.70E-06	4832	3.90E-07	8100	1.90E-07	8100	1.90E-07	8100	1.90E-07	8100	1.90E-07
E	1245	2.10E-06	4887	3.00E-07	8338	1.50E-07	8338	1.50E-07	2994	6.00E-07	2994	6.00E-07
ESE	1600	1.80E-06	4934	3.80E-07	8530	1.80E-07	8530	1.80E-07	7887	2.00E-07	7887	2.00E-07
SE	1600	1.90E-06	4964	4.00E-07	8651	1.90E-07	8651	1.90E-07	7202	2.40E-07	4790	4.20E-07
SSE	1600	2.90E-06	4973	6.30E-07	8686	3.00E-07	8686	3.00E-07	7398	3.70E-07	7398	3.70E-07
S	1600	5.40E-06	4959	1.20E-06	8631	5.70E-07	8631	5.70E-07	8631	5.70E-07	8631	5.70E-07
SSW	1600	5.90E-06	4925	1.40E-06	8492	6.60E-07	8492	6.60E-07	6565	9.20E-07	8492	6.60E-07
SW	1600	4.00E-06	4876	9.30E-07	8287	4.60E-07	8287	4.60E-07	4922	9.10E-07	4922	9.10E-07
WSW	1600	2.30E-06	4832	5.10E-07	8044	2.50E-07	8044	2.50E-07	7242	2.90E-07	7242	2.90E-07
W	1245	2.20E-06	4887	3.30E-07	7798	1.70E-07	7798	1.70E-07	4754	3.40E-07	4754	3.40E-07
WNW	1245	1.60E-06	4934	2.40E-07	7588	1.30E-07	7588	1.30E-07	7588	1.30E-07	7588	1.30E-07
NW	1245	1.50E-06	4964	2.10E-07	7449	1.20E-07	7449	1.20E-07	7449	1.20E-07	7449	1.20E-07
NNW	1245	1.70E-06	4973	2.30E-07	7408	1.30E-07	7408	1.30E-07	2580	5.80E-07	2580	5.80E-07

HAR COL 2.3-5

Table 2.3.5-203 (Sheet 2 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (2.26-Day Decay) for Routine Releases for HAR 2 and HAR 3

	Nearest Residence		Downwind Distance (mi.)									
Downwind Sector	Distance (m)	Chi/Q	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2974	6.40E-07	4.45E-06	2.40E-06	1.56E-06	8.63E-07	5.69E-07	4.12E-07	3.17E-07	2.54E-07	2.10E-07	1.77E-07
NNE	2668	9.10E-07	5.42E-06	2.92E-06	1.89E-06	1.06E-06	6.99E-07	5.08E-07	3.91E-07	3.14E-07	2.60E-07	2.20E-07
NE	3534	5.70E-07	5.07E-06	2.70E-06	1.75E-06	9.79E-07	6.50E-07	4.73E-07	3.65E-07	2.94E-07	2.43E-07	2.06E-07
ENE	8100	1.90E-07	5.50E-06	2.88E-06	1.86E-06	1.05E-06	6.97E-07	5.09E-07	3.94E-07	3.17E-07	2.63E-07	2.23E-07
E	2680	7.00E-07	4.18E-06	2.18E-06	1.41E-06	8.04E-07	5.41E-07	3.97E-07	3.09E-07	2.50E-07	2.08E-07	1.77E-07
ESE	4676	4.10E-07	5.32E-06	2.72E-06	1.75E-06	1.01E-06	6.83E-07	5.04E-07	3.94E-07	3.19E-07	2.66E-07	2.27E-07
SE	4790	4.20E-07	5.70E-06	2.90E-06	1.86E-06	1.07E-06	7.23E-07	5.33E-07	4.16E-07	3.37E-07	2.81E-07	2.40E-07
SSE	7398	3.70E-07	8.90E-06	4.50E-06	2.89E-06	1.68E-06	1.14E-06	8.43E-07	6.59E-07	5.36E-07	4.47E-07	3.82E-07
S	8631	5.70E-07	1.66E-05	8.36E-06	5.36E-06	3.13E-06	2.14E-06	1.59E-06	1.25E-06	1.02E-06	8.49E-07	7.26E-07
SSW	6565	9.20E-07	1.85E-05	9.19E-06	5.87E-06	3.46E-06	2.37E-06	1.77E-06	1.39E-06	1.13E-06	9.48E-07	8.11E-07
SW	4922	9.10E-07	1.25E-05	6.26E-06	4.01E-06	2.35E-06	1.61E-06	1.19E-06	9.38E-07	7.64E-07	6.39E-07	5.46E-07
WSW	7242	2.90E-07	6.89E-06	3.51E-06	2.26E-06	1.30E-06	8.83E-07	6.53E-07	5.10E-07	4.14E-07	3.45E-07	2.94E-07
W	4594	3.50E-07	4.48E-06	2.30E-06	1.48E-06	8.52E-07	5.76E-07	4.25E-07	3.32E-07	2.69E-07	2.24E-07	1.91E-07
WNW	3577	3.70E-07	3.31E-06	1.73E-06	1.12E-06	6.37E-07	4.28E-07	3.14E-07	2.44E-07	1.98E-07	1.64E-07	1.40E-07
NW	3268	3.70E-07	2.98E-06	1.57E-06	1.02E-06	5.74E-07	3.82E-07	2.79E-07	2.16E-07	1.74E-07	1.44E-07	1.22E-07
NNW	1936	8.80E-07	3.28E-06	1.76E-06	1.14E-06	6.38E-07	4.22E-07	3.07E-07	2.37E-07	1.90E-07	1.57E-07	1.33E-07

HAR COL 2.3-5

Table 2.3.5-203 (Sheet 3 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (2.26-Day Decay) for Routine Releases for HAR 2 and HAR 3

Downwind Sector	Downwind Distance (mi.)										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	1.53E-07	8.54E-08	5.64E-08	3.13E-08	2.05E-08	1.47E-08	1.11E-08	8.74E-09	7.08E-09	5.87E-09	4.95E-09
NNE	1.89E-07	1.06E-07	7.04E-08	3.91E-08	2.56E-08	1.83E-08	1.39E-08	1.09E-08	8.85E-09	7.32E-09	6.17E-09
NE	1.77E-07	1.00E-07	6.63E-08	3.70E-08	2.42E-08	1.74E-08	1.31E-08	1.03E-08	8.37E-09	6.93E-09	5.84E-09
ENE	1.92E-07	1.09E-07	7.24E-08	4.05E-08	2.66E-08	1.91E-08	1.45E-08	1.14E-08	9.27E-09	7.68E-09	6.48E-09
E	1.53E-07	8.72E-08	5.83E-08	3.28E-08	2.16E-08	1.55E-08	1.17E-08	9.22E-09	7.47E-09	6.18E-09	5.20E-09
ESE	1.97E-07	1.13E-07	7.57E-08	4.27E-08	2.82E-08	2.02E-08	1.54E-08	1.21E-08	9.79E-09	8.11E-09	6.83E-09
SE	2.07E-07	1.19E-07	7.98E-08	4.51E-08	2.97E-08	2.13E-08	1.62E-08	1.28E-08	1.03E-08	8.56E-09	7.21E-09
SSE	3.31E-07	1.91E-07	1.28E-07	7.26E-08	4.79E-08	3.45E-08	2.62E-08	2.06E-08	1.67E-08	1.38E-08	1.17E-08
S	6.30E-07	3.64E-07	2.46E-07	1.40E-07	9.22E-08	6.64E-08	5.04E-08	3.97E-08	3.22E-08	2.66E-08	2.24E-08
SSW	7.05E-07	4.09E-07	2.77E-07	1.57E-07	1.04E-07	7.48E-08	5.68E-08	4.47E-08	3.62E-08	3.00E-08	2.52E-08
SW	4.75E-07	2.75E-07	1.86E-07	1.05E-07	6.96E-08	5.01E-08	3.80E-08	2.99E-08	2.42E-08	2.01E-08	1.69E-08
WSW	2.55E-07	1.46E-07	9.82E-08	5.54E-08	3.65E-08	2.62E-08	1.99E-08	1.56E-08	1.27E-08	1.05E-08	8.81E-09
W	1.65E-07	9.47E-08	6.35E-08	3.58E-08	2.36E-08	1.69E-08	1.28E-08	1.01E-08	8.17E-09	6.76E-09	5.68E-09
WNW	1.21E-07	6.88E-08	4.60E-08	2.58E-08	1.70E-08	1.22E-08	9.22E-09	7.25E-09	5.87E-09	4.86E-09	4.09E-09
NW	1.06E-07	5.96E-08	3.97E-08	2.21E-08	1.45E-08	1.04E-08	7.87E-09	6.19E-09	5.01E-09	4.15E-09	3.49E-09
NNW	1.15E-07	6.44E-08	4.27E-08	2.37E-08	1.56E-08	1.11E-08	8.43E-09	6.63E-09	5.37E-09	4.45E-09	3.75E-09

Notes:

a) The reported distance of the Low Population Zone (LPZ) is measured from the centerpoint of HAR 2 and HAR 3 to the outermost boundary of the LPZ.

Wind Reference Level: 12 m

Stability Type: ΔT (61 – 12 m)

Release Type: Ground Level: 12 m

Building Height/Cross Section: 43.9 m/2730 m^2

HAR COL 2.3-5

Table 2.3.5-204 (Sheet 1 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (Depleted and 8-Day Decayed) for Routine Releases for HAR 2 and HAR 3

	Exclusion Area Boundary		Low Population Zone[a]		Nearest Milk Cow		Nearest Milk Goat		Nearest Garden		Nearest Meat Animal	
Downwind Sector	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q	Distance (m)	Chi/Q
N	1245	2.00E-06	4959	2.50E-07	2973	5.40E-07	7472	1.30E-07	2974	5.40E-07	2974	5.40E-07
NNE	1245	2.50E-06	4925	3.10E-07	4297	3.80E-07	7630	1.60E-07	2668	7.80E-07	2668	7.80E-07
NE	1245	2.30E-06	4876	2.90E-07	7851	1.40E-07	7851	1.40E-07	7851	1.40E-07	7851	1.40E-07
ENE	1245	2.50E-06	4832	3.20E-07	8100	1.50E-07	8100	1.50E-07	8100	1.50E-07	8100	1.50E-07
E	1245	1.90E-06	4887	2.50E-07	8338	1.10E-07	8338	1.10E-07	2994	5.10E-07	2994	5.10E-07
ESE	1600	1.60E-06	4934	3.10E-07	8530	1.40E-07	8530	1.40E-07	7887	1.60E-07	7887	1.60E-07
SE	1600	1.70E-06	4964	3.30E-07	8651	1.50E-07	8651	1.50E-07	7202	1.90E-07	4790	3.40E-07
SSE	1600	2.60E-06	4973	5.20E-07	8686	2.30E-07	8686	2.30E-07	7398	2.90E-07	7398	2.90E-07
S	1600	4.80E-06	4959	9.80E-07	8631	4.40E-07	8631	4.40E-07	8631	4.40E-07	8631	4.40E-07
SSW	1600	5.20E-06	4925	1.10E-06	8492	5.10E-07	8492	5.10E-07	6565	7.40E-07	8492	5.10E-07
SW	1600	3.60E-06	4876	7.60E-07	8287	3.60E-07	8287	3.60E-07	4922	7.50E-07	4922	7.50E-07
WSW	1600	2.00E-06	4832	4.20E-07	8044	2.00E-07	8044	2.00E-07	7242	2.30E-07	7242	2.30E-07
W	1245	2.00E-06	4887	2.70E-07	7798	1.30E-07	7798	1.30E-07	4754	2.80E-07	4754	2.80E-07
WNW	1245	1.50E-06	4934	1.90E-07	7588	1.00E-07	7588	1.00E-07	7588	1.00E-07	7588	1.00E-07
NW	1245	1.30E-06	4964	1.70E-07	7449	9.20E-08	7449	9.20E-08	7449	9.20E-08	7449	9.20E-08
NNW	1245	1.50E-06	4973	1.80E-07	7408	1.00E-07	7408	1.00E-07	2580	4.90E-07	2580	4.90E-07

HAR COL 2.3-5

Table 2.3.5-204 (Sheet 2 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (Depleted and 8-Day Decayed) for Routine Releases for HAR 2 and HAR 3

	Nearest Residence		Downwind Distance (mi.)									
Downwind Sector	Distance (m)	Chi/Q	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5
N	2974	5.40E-07	4.08E-06	2.15E-06	1.37E-06	7.39E-07	4.77E-07	3.39E-07	2.57E-07	2.03E-07	1.65E-07	1.38E-07
NNE	2668	7.80E-07	4.97E-06	2.61E-06	1.67E-06	9.05E-07	5.86E-07	4.18E-07	3.17E-07	2.51E-07	2.05E-07	1.71E-07
NE	3534	4.70E-07	4.64E-06	2.42E-06	1.54E-06	8.40E-07	5.45E-07	3.90E-07	2.96E-07	2.35E-07	1.92E-07	1.61E-07
ENE	8100	1.50E-07	5.04E-06	2.58E-06	1.64E-06	8.98E-07	5.85E-07	4.19E-07	3.19E-07	2.54E-07	2.08E-07	1.74E-07
E	2680	5.90E-07	3.83E-06	1.95E-06	1.24E-06	6.91E-07	4.55E-07	3.28E-07	2.52E-07	2.01E-07	1.65E-07	1.39E-07
ESE	4676	3.40E-07	4.88E-06	2.44E-06	1.54E-06	8.68E-07	5.75E-07	4.17E-07	3.21E-07	2.57E-07	2.12E-07	1.78E-07
SE	4790	3.40E-07	5.22E-06	2.60E-06	1.64E-06	9.20E-07	6.09E-07	4.41E-07	3.39E-07	2.71E-07	2.23E-07	1.88E-07
SSE	7398	2.90E-07	8.17E-06	4.04E-06	2.55E-06	1.44E-06	9.59E-07	6.98E-07	5.38E-07	4.31E-07	3.56E-07	3.00E-07
S	8631	4.40E-07	1.53E-05	7.50E-06	4.74E-06	2.70E-06	1.80E-06	1.32E-06	1.02E-06	8.18E-07	6.77E-07	5.72E-07
SSW	6565	7.40E-07	1.70E-05	8.25E-06	5.19E-06	2.98E-06	2.00E-06	1.47E-06	1.14E-06	9.14E-07	7.57E-07	6.41E-07
SW	4922	7.50E-07	1.15E-05	5.62E-06	3.54E-06	2.02E-06	1.36E-06	9.91E-07	7.67E-07	6.16E-07	5.10E-07	4.31E-07
WSW	7242	2.30E-07	6.32E-06	3.15E-06	1.99E-06	1.12E-06	7.44E-07	5.40E-07	4.16E-07	3.33E-07	2.74E-07	2.31E-07
W	4594	2.90E-07	4.11E-06	2.06E-06	1.31E-06	7.33E-07	4.85E-07	3.52E-07	2.70E-07	2.16E-07	1.78E-07	1.50E-07
WNW	3577	3.10E-07	3.04E-06	1.55E-06	9.84E-07	5.47E-07	3.60E-07	2.60E-07	1.99E-07	1.59E-07	1.30E-07	1.10E-07
NW	3268	3.10E-07	2.73E-06	1.41E-06	8.97E-07	4.92E-07	3.21E-07	2.30E-07	1.75E-07	1.39E-07	1.14E-07	9.55E-08
NNW	1936	7.60E-07	3.00E-06	1.58E-06	1.01E-06	5.47E-07	3.54E-07	2.53E-07	1.92E-07	1.52E-07	1.24E-07	1.04E-07

HAR COL 2.3-5

Table 2.3.5-204 (Sheet 3 of 3)
Long-Term Average Chi/Q (in sec/m^3) Calculations (Depleted and 8-Day Decayed) for Routine Releases for HAR 2 and HAR 3

Downwind Sector	Downwind Distance (mi.)										
	5.0	7.5	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
N	1.17E-07	6.30E-08	4.03E-08	2.13E-08	1.35E-08	9.38E-09	6.95E-09	5.36E-09	4.27E-09	3.49E-09	2.90E-09
NNE	1.46E-07	7.86E-08	5.05E-08	2.68E-08	1.70E-08	1.18E-08	8.77E-09	6.77E-09	5.40E-09	4.41E-09	3.67E-09
NE	1.37E-07	7.42E-08	4.77E-08	2.54E-08	1.62E-08	1.13E-08	8.37E-09	6.48E-09	5.17E-09	4.22E-09	3.51E-09
ENE	1.48E-07	8.09E-08	5.22E-08	2.80E-08	1.78E-08	1.25E-08	9.29E-09	7.20E-09	5.76E-09	4.71E-09	3.92E-09
E	1.19E-07	6.53E-08	4.24E-08	2.29E-08	1.47E-08	1.03E-08	7.69E-09	5.97E-09	4.77E-09	3.91E-09	3.26E-09
ESE	1.53E-07	8.48E-08	5.54E-08	3.01E-08	1.94E-08	1.37E-08	1.02E-08	7.93E-09	6.35E-09	5.21E-09	4.34E-09
SE	1.61E-07	8.95E-08	5.84E-08	3.18E-08	2.04E-08	1.44E-08	1.08E-08	8.36E-09	6.70E-09	5.49E-09	4.58E-09
SSE	2.58E-07	1.44E-07	9.41E-08	5.14E-08	3.31E-08	2.34E-08	1.75E-08	1.36E-08	1.09E-08	8.95E-09	7.47E-09
S	4.92E-07	2.75E-07	1.81E-07	9.91E-08	6.40E-08	4.53E-08	3.39E-08	2.64E-08	2.12E-08	1.74E-08	1.45E-08
SSW	5.52E-07	3.10E-07	2.04E-07	1.12E-07	7.27E-08	5.15E-08	3.86E-08	3.01E-08	2.42E-08	1.98E-08	1.66E-08
SW	3.71E-07	2.08E-07	1.37E-07	7.51E-08	4.86E-08	3.44E-08	2.57E-08	2.01E-08	1.61E-08	1.32E-08	1.10E-08
WSW	1.98E-07	1.10E-07	7.20E-08	3.92E-08	2.52E-08	1.78E-08	1.33E-08	1.03E-08	8.26E-09	6.77E-09	5.65E-09
W	1.29E-07	7.13E-08	4.65E-08	2.53E-08	1.62E-08	1.14E-08	8.53E-09	6.63E-09	5.31E-09	4.35E-09	3.63E-09
WNW	9.37E-08	5.15E-08	3.35E-08	1.81E-08	1.16E-08	8.12E-09	6.05E-09	4.70E-09	3.76E-09	3.08E-09	2.56E-09
NW	8.16E-08	4.44E-08	2.87E-08	1.53E-08	9.76E-09	6.83E-09	5.07E-09	3.93E-09	3.14E-09	2.56E-09	2.13E-09
NNW	8.82E-08	4.77E-08	3.06E-08	1.63E-08	1.03E-08	7.18E-09	5.32E-09	4.12E-09	3.28E-09	2.68E-09	2.23E-09

Notes:

a) The reported distance of the Low Population Zone (LPZ) is measured from the centerpoint of HAR 2 and HAR 3 to the outermost boundary of the LPZ.

Wind Reference Level: 12 m

Stability Type: ΔT (61 – 12 m)

Release Type: Ground Level: 12 m

Building Height/Cross Section: 43.9 m/2730 m^2

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

HAR SUP 2.4-1

The proposed Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR) site is located in southwest Wake County and southeast Chatham County, North Carolina. Progress Energy Carolinas, Inc. (PEC) owns the site. Major cities near the site include Cary, Raleigh, and Sanford, North Carolina. The closest major city is Cary, which is located 21 kilometers (km) (13 miles [mi.]) northeast of the site. Raleigh is located about 34.9 km (21.7 mi.) northeast of the site, and Sanford is located about 26.5 km (16.5 mi.) southwest of the site. One 900 megawatt electrical (MWe) Westinghouse Electric Company, LLC (Westinghouse) pressurized water reactor is currently in operation. This facility is referred to as the existing Shearon Harris Nuclear Power Plant Unit 1 (HNP). PEC has selected Westinghouse's AP1000 Reactor (AP1000) as the certified plant design for the HAR site. The proposed Westinghouse AP1000 units are referred to as the proposed Shearon Harris Nuclear Power Plant Unit 2 (HAR 2) and the proposed Shearon Harris Nuclear Power Plant Unit 3 (HAR 3).

Currently, the HNP obtains its water supply from the Main Reservoir (also known as Harris Reservoir). The Main Reservoir was originally designed to provide cooling water and to remove the design heat load from the Cooling Tower blowdown water for four reactor units. During construction activities for the units, a decision was made to reduce the number of units to one; therefore, only the HNP was completed. The Main Reservoir was completed before the decision; therefore, the current reservoir was designed for multiple units. However, the reservoir level was raised only to the level to support the one unit and the makeup water system from the river was never built. PEC intends to use the Main Reservoir for HAR 2 and HAR 3 along with HNP.

Harris Lake consists of two reservoirs: the Main Reservoir and the Auxiliary Reservoir (Figure 2.4.1-201). Carolina Power and Light Company (CP&L) constructed the Main Reservoir in 1980 by building an earthen dam across Buckhorn Creek about 365.8 meters (m) (1200 feet [ft.]) downstream of the confluence of White Oak and Buckhorn Creeks. CP&L constructed the Auxiliary Reservoir in 1980 by installing an earthen dam across Tom Jack Creek. The Auxiliary Reservoir is an emergency water source for the HNP. In contrast, the

Auxiliary Reservoir will not be an emergency water source for HAR. The Main Reservoir is also a water source for the HNP emergency service water system.

The HAR site is located immediately north of the HNP between the Thomas Creek and Tom Jack Creek branches of the reservoir system about 7.2 km (4.5 mi.) north of the Main Dam and about 11.3 km (7 mi.) north of the Cape Fear River. The nominal grade elevation for the HNP facility is 79.2 m (260 ft.) National Geodetic Vertical Datum of 1929 (NGVD29). The proposed nominal plant grade elevation for the HAR site is 79.6 m (261 ft.) NGVD29. The actual plant grade will be lower and will vary to accommodate site grading, drainage, and local site flooding requirements. The nominal plant grade floor elevation for the HAR site is 79.6 m (261 ft.) NGVD29.

The current normal pool water elevation for the Main Reservoir and the Auxiliary Reservoir is 67.1 m (220 ft.) NGVD29 and 76.8 m (252 ft.) NGVD29, respectively. The proposed normal pool water elevation for the Main Reservoir is 73.2 m (240 ft.) NGVD29; no reservoir level changes are proposed for the Auxiliary Reservoir.

HAR COL 2.4-3

HNP collects Cooling Tower makeup water at the Cooling Tower makeup water intake structure, located on the Thomas Creek branch of the Main Reservoir, east of the site (Figure 2.4.1-202). After cooling, the blowdown water is discharged into the Main Reservoir through a 121.9 cm (48 in.) diameter, 6096 m (20,000 ft.) long pipeline at a location about 1484.4 m (4870 ft.) north of the Main Dam. Under conditions of Main Dam failure, the HNP would use the independent Auxiliary Reservoir for emergency core cooling purposes. Emergency service water would be supplied through the emergency service water screening structure and the emergency service water intake structure in the Auxiliary Reservoir to the Cooling Tower and discharged back into the Auxiliary Reservoir through the emergency service water discharge structure. A separating dike located across the east arm of the Auxiliary Reservoir creates a flow boundary between the emergency service water intake and discharge structures to extend the emergency service water residence time within the reservoir. The decks of the emergency service water and Cooling Tower makeup water intake structure, the emergency service water screening structure, and the emergency service water discharge structure for the HNP are all at an elevation of 79.9 m (262 ft.) NGVD29.

HAR 2 and HAR 3 will collect Cooling Tower makeup water at the proposed HAR raw water pumphouse located on the Thomas Creek branch of the Main Reservoir east of the site and approximately 975.4 m (3200 ft.) north of the HNP Cooling Tower makeup water intake channel (Figure 2.4.1-203). After being used, the Cooling Tower blowdown water will be discharged into the Main Reservoir, through a new pipe installed parallel to the current blowdown discharge pipe for the HNP. Under conditions of Main Reservoir failure, HAR 2 and HAR 3 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. Use of the Auxiliary Reservoir will not be required for emergency cooling water for HAR 2 and HAR 3. The proposed elevation of the HAR raw water pumphouse structure deck is 79.9 m (262 ft.) NGVD29.

The water supply source to provide makeup water to the HAR service water system cooling tower will be the Main Reservoir via the raw water system. Detailed information regarding this system can be found in Section 9.2 of this FSAR and in Section 9.2 of the DCD.

Makeup water will be obtained from the Cape Fear River to maintain the proposed operating water level of the Main Reservoir at 73.2 m (240 ft.) NGVD29. The Harris Lake makeup water system has been designed to maintain the required reservoir level and to minimize radionuclide buildup in the reservoir. This system includes the intake structure and pipe in the Cape Fear River, the Harris Lake makeup water system pumphouse on the Cape Fear River, the Harris Lake makeup water system pipeline from the Cape Fear River to the Main Reservoir, and the Harris Lake makeup water system discharge structure on the Main Reservoir (Figure 2.4.1-203). The structures associated with the Harris Lake makeup water system are nonsafety-related. The maximum flow capacity from the Harris Lake makeup water system pumphouse to the Main Reservoir is 60,000 gpm or 3.79 m^3/s (133.68 cfs).

Water from the Cape Fear River, in addition to the Main Reservoir drainage area, will be required to fill and maintain the required pool level for normal operations. The rate at which water is withdrawn from the Cape Fear River for maintenance of water quality will be based on a set of operational rules designed to meet target flows, such as a minimum discharge of 0.57 m^3/s (20 cfs) from the Main Reservoir to Buckhorn Creek. A higher withdrawal rate will be used during high river flow periods to fill the lake and manage water quality. During periods of drought, the Main Reservoir will provide some or all of the required cooling water supply. To achieve a minimum discharge of 0.57 m^3/s (20 cfs) from the Main Reservoir to Buckhorn Creek during periods when water does not flow over the Main Dam spillway, water can be released from the Main Reservoir by use of three Howell-Bunger valves located in the central pier and side abutments of the spillway. For more information on the Howell-Bunger valves, see Section 2.4.8 of the HNP FSAR.

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 proposed location for HAR 2 is in the northern portion of the plant site at an existing grade elevation of 79.2 m (260 ft.) NGVD29. The proposed location for HAR 3, which will require cutting and filling of the existing grade, will affect the current drainage pattern. Currently, most of the surface water runoff from the plant site is drained by a storm sewer system. The peripheral areas of the plant site drain through open ditches and culverts to the Main Reservoir or to the sides of the emergency service water intake and discharge channels. If the drainage system becomes blocked, the plant site can be drained by overland flow on open

roads and ground surface directly to the Main Reservoir, the Auxiliary Reservoir, or the emergency service water intake and discharge channels.

Seismic Category I structures that should be considered from the hydrologic standpoint include the Main Dam and the Auxiliary Dam, their spillways, and 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 FSAR provides details related to these structures.

The Main Dam structure has a length of 472.4 m (1550 ft.), with a 2:1 (horizontal to vertical) slope on both the upstream and downstream faces. The top of the dam is at an elevation of 79.2 m (260 ft.) NGVD29 (about 12.2 m [40 ft.] above the current normal pool elevation and 6.1 m [20 ft.] above the proposed normal pool elevation), with a width of 7.6 m (25 ft.). The maximum height of the dam is 33.0 m (108 ft.) above the top of the bedrock. Riprap is provided on the upstream slope of the dam for protection against wind-wave erosion and the effects of reservoir draw-down. The downstream slope has a 1.2 m (4 ft.) thick layer of oversized rock for erosion protection.

The Main Dam currently includes a concrete spillway with an ogee-shaped crest on the west abutment to pass floods as the only flow component. The spillway is uncontrolled and has a crest net length of 15.2 m (50 ft.) with a pier at mid-length. The crest of the current spillway is at an elevation of 67.1 m (220 ft.) NGVD29. The proposed spillway design for the Main Reservoir includes raising the crest of the existing spillway from elevation 67.1 m (220 ft.) NGVD29 to elevation 73.2 m (240 ft.) NGVD29, and providing an emergency spillway west of the existing spillway with a crest length of 172.2 m (565 ft.).

The Auxiliary Dam structure has a length of 1189.6 m (3903 ft.), with a 2.5:1 (horizontal to vertical) slope on both the upstream and downstream faces. The top of the dam is at an elevation of 79.2 m (260 ft.) NGVD29 (about 2.4 m [8 ft.] above the normal pool elevation), with a width of 6.1 m (20 ft.). The maximum height of the dam is 21.9 m (72 ft.) above the top of bedrock. Riprap is provided on the upstream and downstream slopes of the dam for protection against wind-wave erosion and reservoir draw-down effects.

The Auxiliary Dam includes a concrete spillway with an ogee-shaped crest on the west abutment of the dam to pass floods. The crest of the spillway is at an elevation of 76.8 m (252 ft.) NGVD29 and has a crest length of 51.8 m (170 ft.).

The Auxiliary Reservoir separating dike is located about 518.2 m (1700 ft.) north of the Auxiliary Dam across the eastern arm of the Auxiliary Reservoir separating the emergency service water intake channel from the emergency service water discharge channel used by the HNP. The separating dike has a length of 365.8 m (1200 ft.), with a 2.5:1 (horizontal to vertical) slope on both the upstream and downstream faces. The top of the dike is at an elevation of 77.7 m (255 ft.) NGVD29 (about 0.9 m [3 ft.] above the normal pool elevation), with a width of

6.1 m (20 ft.). The maximum height of the dike is 16.8 m (55 ft.) above the top of bedrock. Riprap is provided on the upstream and downstream slopes of the dike for protection against wind-wave erosion and reservoir draw-down effects.

HAR COL 2.4-1

2.4.1.2 Hydrosphere

2.4.1.2.1 HAR Site

As stated in Subsection 2.4.1.1, the principal source of water for the HAR site and the HNP is the Main Reservoir which is part of Harris Lake, which consists of the Main Reservoir and the Auxiliary Reservoir (Figure 2.4.1-201). The Main Reservoir, situated on Buckhorn Creek, is impounded by an earthen dam located just below the confluence of White Oak Creek and Buckhorn Creek. The Auxiliary Reservoir, located on Tom Jack Creek, is formed by an earthen dam situated to the west of the plant site. There are two reservoir branches adjacent to the HAR site: Tom Jack Creek to the west and Thomas Creek to the east. Drainage area for Harris Lake at the Main Dam site is 182.1 square kilometers (km^2) (70.3 square miles [$mi.^2$]).

Currently, the Main Reservoir has the following characteristics:

- A maximum depth of about 18 m (59 ft.).
- A mean depth of 5.3 m (17.4 ft.).
- A normal pool elevation of 67.1 m (220 ft.) NGVD29.
- A storage capacity of 9.0 x 107 cubic meters (m^3) (73,000 acre-feet [ac-ft]) at the normal pool elevation.
- A surface area of about 14.4 km^2 (5.6 $mi.^2$ or 7.8 percent of the drainage area).
- A residence time of 28 months.

With the addition of HAR 2 and HAR 3, the normal pool elevation of the Main Reservoir will be raised to 73.2 m (240 ft.) NGVD29. The storage capacity and surface area will increase to 2.2 x 10^8 m^3 (177,563 ac-ft) and 30.8 km^2 (11.9 $mi.^2$ or approximately 16 percent of the drainage area), respectively. Figure 2.4.1-206 shows area and capacity curves for the Main Reservoir.

The current Auxiliary Reservoir normal pool elevation is 76.8 m (252 ft.) NGVD29 and has a surface area of about 1.3 km^2 (0.5 $mi.^2$ or 0.7 percent of the drainage area); no changes to the current water level will occur in the Auxiliary Reservoir. Figure 2.4.1-207 shows area and capacity curves for the Auxiliary Reservoir.

Two ponds and one stormwater retention pond are located within the boundary of the plant site. Originally, both ponds were drainage valleys or depressions that directed overland flow to Tom Jack Creek and Thomas Creek prior to construction of the HNP. The drainage areas filled with water after the Main Reservoir and the Auxiliary Reservoir were filled to capacity. PEC proposes to fill the two ponds located directly north of the HAR site with soil during construction activities. The stormwater retention pond, which will be retained, is located west of the HNP and has a width of about 16.8 m (55 ft.) and a length of about 45.7 m (150 ft.).

2.4.1.2.2 Buckhorn Creek

Buckhorn Creek, which has its headwaters near Holly Springs and Apex, North Carolina, flows southwest to its confluence with the Cape Fear River. The confluence is located about 22.0 km (13.7 mi.) northwest of the Town of Lillington, North Carolina. As shown on Figure 2.4.1-208, Buckhorn Creek has five tributaries above the Main Dam: Tom Jack Creek, Thomas Creek, Little White Oak Creek, White Oak Creek, and Cary Creek. These five creeks, together with the remainder of the Buckhorn Creek basin, drain a watershed area of approximately 205.9 km^2 (79.5 $mi.^2$). The entire drainage basin lies near the eastern edge of the Piedmont Plateau, with elevations between 45.7 and 137.2 m (150 and 450 ft.) mean sea level (msl).

A U.S. Geological Survey (USGS) gauging station (USGS 02102192 Buckhorn Creek NR Corinth, North Carolina) is located on Buckhorn Creek about 0.9 km (0.56 mi.) below the Main Dam spillway. Periods of record for data collected include June 9, 1972 through present day for stream stage and flow measurements; and December 15, 1972 through September 1, 1978, for suspended sediment concentrations. The drainage area at the station is 197.6 km^2 (76.3 $mi.^2$) (Reference 2.4-201).

Table 2.4.1-201 presents the mean monthly discharge (June 1972 through September 2004) for the Buckhorn Creek basin at the Buckhorn Creek gauging station (Reference 2.4-201). The average monthly discharge of Buckhorn Creek for this 33-year period is 1.7 m^3/s (59.8 cfs). March has the highest average monthly discharge of 4.1 m^3/s (143 cfs) and October has the lowest average monthly discharge of 0.5 m^3/s (18.6 cfs). A maximum average daily streamflow of 88.6 m^3/s (3130 cfs) was recorded on February 2, 1973 (Table 2.4.1-202).

Currently, no reservoirs, dams, or creek control structures are located upstream or downstream of Harris Lake that can affect the availability of the water supply to the reservoir system and HAR site structures. Furthermore, Buckhorn Creek (downstream of the Main Dam) is not a likely candidate for changes that would result in additional water demand because the flow is often low for long periods.

2.4.1.2.3 Cape Fear River

The Cape Fear River basin is the largest river basin located entirely in North Carolina. The basin has an oblong shape with a maximum width of about 96.6 km (60 mi.), maximum length of about 321.9 km (200 mi.), and about 9734.9 km (6049 mi.) of streams and rivers (Reference 2.4-202). The basin has a total area of 23,673 km^2 (9140 $mi.^2$), of which approximately 8099 km^2 (3127 $mi.^2$) are located above the confluence of the Deep and Haw Rivers (Reference 2.4-203). The Cape Fear River is formed by the confluence of the Deep and Haw Rivers. The Cape Fear River flows southeast for about 318.7 km (198 mi.) and empties into the Atlantic Ocean at Cape Fear, North Carolina, located 45.1 km (28 mi.) below Wilmington, North Carolina. Figure 2.4.1-209 shows details of the Cape Fear River drainage basin and its relation to Buckhorn Creek.

The lower Cape Fear River is an estuary with the tidal reach extending to Lock and Dam 1, about 62.8 km (39 mi.) above Wilmington, North Carolina. The river is navigable to Fayetteville, North Carolina:

- A channel width of generally 121.9 m (400 ft.) and a depth ranging from 9.1 to 10.7 m (30 to 35 ft.) occurs from the Atlantic Ocean to Wilmington, North Carolina.

- A 61.0-m (200-ft.) width and 7.6-m (25-ft.) depth occurs from Wilmington to Navassa, North Carolina.

- A depth of 2.4 m (8 ft.) with varying widths occurs for the remaining distance to Fayetteville, North Carolina.

The average width of the floodplain is about 3.5 km (2.2 mi.).

Table 2.4.1-203 presents the locations of flow monitoring stations in the Cape Fear River basin and the maximum flows at each station. Bank-full flood discharge for the Cape Fear River is an estimated 849.5 m^3/s (30,000 cfs) at Lillington, North Carolina, and 858.0 m^3/s (30,300 cfs) at Fayetteville, North Carolina. Distances downstream from the confluence of the Buckhorn Creek and Cape Fear River are approximately 22.0 km (13.7 mi.) and 80.8 km (50.2 mi.), respectively (Table 2.4.1-203). Figure 2.4.1-210 shows USGS monitoring stations located on the Haw, Deep, and Cape Fear Rivers.

Table 2.4.1-204 presents the mean monthly discharge (January 1924 through September 2004) for the Cape Fear River USGS gauging station 02102500 at Lillington, North Carolina (Reference 2.4-201). The average monthly discharge for the Cape Fear River at Lillington is 95.9 m^3/s (3387 cfs). March has the highest average monthly discharge of 182.4 m^3/s (6441 cfs) and August has the lowest average monthly discharge of 55.8 m^3/s (1970 cfs). A maximum average daily streamflow of 3964.4 m^3/s (140,000 cfs) was recorded on

September 19, 1945 (Table 2.4.1-205). The drainage area at the Cape Fear River at Lillington USGS station is 8971 km^2 (3464 $mi.^2$) (Reference 2.4-271).

2.4.1.2.4 Cape Fear River Basin – Tributaries

The Cape Fear River has two major tributaries above the Buckhorn Dam (which is located immediately upstream of the confluence of the Buckhorn Creek and Cape Fear River). These major tributaries are the Haw and Deep Rivers, both of which originate in Forsyth County, North Carolina. The Deep River has a total length of 186.7 km (116 mi.) and a drainage area of 3683 km^2 (1422 $mi.^2$). The Haw River is about 144.8 km (90 mi.) in length and drains about 4415 km^2 (1705 $mi.^2$). Both rivers originate at elevations of about 304.8 m (1000 ft.) msl and have numerous falls and rapids, with the Haw River having the steepest gradient. The water surface elevation of the junction of the two rivers is about 48.2 m (158 ft.) NGVD29.

Other major tributaries downstream of the withdrawal point include the Black River and Northeast Cape Fear River. The Black River has a drainage area of 4048 km^2 (1563 $mi.^2$) and joins the Cape Fear River at river mile 44. The Northeast Cape Fear River drains a basin of 4501 km^2 (1738 $mi.^2$) and enters the Cape Fear River at Wilmington, North Carolina.

There are numerous minor tributaries including Upper Little River, Little River, Rockfish Creek, and Buckhorn Creek.

2.4.1.2.5 Cape Fear River Basin – Dams, Reservoirs, and Locks

There are a number of regulating structures and reservoirs on the Cape Fear River. Figure 2.4.1-211 shows the locations of these structures and reservoirs. Lock and Dam 1, Lock and Dam 2, and the William O. Huske Dam are located at river miles 67, 99, and 123, respectively. Buckhorn Dam, which is located at river mile 192, has a spillway crest elevation of 48.21 m (158.18 ft.) msl.

Lockville Hydro Dam is situated on the lower reach of the Deep River. In addition, on August 7, 2001, the Piedmont Triad Regional Water Authority (PTRWA) began construction of the Randleman Dam, which is located on the upper reach of the Deep River (Reference 2.4-204). Upon completion, Randleman Lake will be located in Randolph and Guilford Counties, North Carolina, and have a surface area of 12.2 km^2 (4.7 $mi.^2$). Water from the dam will be used as a local water supply for the Piedmont Triad region (Reference 2.4-205).

The B. Everett Jordan Lake and associated dam are located on the lower reach of the Haw River about 4.2 river miles above the confluence of the Haw and Deep Rivers, and about 18.2 km (11.3 river miles) upstream of the confluence of the Buckhorn Creek and Cape Fear River. The lake has a normal pool elevation of 65.8 m (216 ft.) msl, a surface area of about 56.4 km^2 (21.8 $mi.^2$ or 1.3 percent of the drainage area), and a storage capacity of 2.7 x 10^8 m^3 (215,130 ac-ft) at

normal pool elevation (Reference 2.4-203). The drainage area for the B. Everett Jordan Lake at the dam site is 4377 km^2 (1690 $mi.^2$).

The B. Everett Jordan Dam is constructed of earth and rockfill (zoned) with an uncontrolled spillway and a multilevel intake structure. The dam has a length of 405.4 m (1330 ft.), a top-of-dam elevation of 81.2 m (266.5 ft.) msl, and a maximum height of 34.4 m (113 ft.) (Reference 2.4-203). The spillway has a crest net length of 243.8 m (800 ft.), a top-of-spillway elevation of 73.2 m (240 ft.) msl, and a capacity of 7334.1 m^3/s (259,000 cfs) at an elevation of 79.7 m (261.5 ft.) msl. Minimum downstream discharge rates fluctuate to maintain the water quality flow requirement of approximately 17.0 m^3/s (600 cfs) in the Cape Fear River at the Lillington streamflow gauge throughout the year, as specified in the approved 1992 Water Control Manual for the B. Everett Jordan Project (Reference 2.4-206).

2.4.1.2.6 Cape Fear River Basin – Additional Waterbodies

Figures 2.4.1-212 and 2.4.1-213 show other reservoirs, lakes, and ponds within a 16.1-km (10-mi.) radius and a 40.2-km (25-mi.) radius of the HAR site, respectively.

2.4.1.2.7 Surface Water Users

There are no known communities either upstream or downstream of the Main Reservoir that draw water from Buckhorn Creek for public water supply. The HAR site will use surface water from the Main Reservoir for domestic, process, and Cooling Tower makeup water. The closest public surface water user downstream of the HAR site is in Lillington, North Carolina, on the Cape Fear River, about 22.0 km (13.7 mi.) downstream from the confluence of the Cape Fear River and Buckhorn Creek. Public water supply locations for surface water and groundwater systems within 16.1 km and 40.2 km (10 mi. and 25 mi.) of the HAR site are shown on Figure 2.4.1-214 and Figure 2.4.1-215 and detailed in Tables 2.4.1-206/2.4.1-207 and 2.4.1-208/2.4.1-209, respectively. Information and details for public water supply locations were obtained from two separate sources — the North Carolina Department of Environment and Natural Resources (NCDENR) (Reference 2.4-207) and the North Carolina Center for Geographic Information and Analysis (Reference 2.4-208). Subsection 2.4.13.2 of this FSAR summarizes groundwater users.

Cities within a 16.1-km (10-mi.) radius of the HAR site are Apex, Cary, Holly Springs, and Fuquay-Varina. The public water source for the cities of Apex (estimated 2004 population: 27,509) and Cary (estimated 2004 population: 101,265) is the B. Everett Jordan Lake (Reference 2.4-209, Reference 2.4-210, Reference 2.4-211, Reference 2.4-212). Drinking water for Holly Springs, North Carolina, (estimated 2004 population: 13,740) is supplied by Harnett County (Cape Fear River; intake is upstream of the Buckhorn Creek Dam) and the City of Raleigh (Falls Lake) (Reference 2.4-213, Reference 2.4-214). The Town of Fuquay-Varina (estimated 2004 population: 11,110) purchases water from the

City of Raleigh, Harnett County, and Johnston County; water sources are not specified (Reference 2.4-215, Reference 2.4-216).

Figure 2.4.1-216 shows that Chatham, Harnett, Lee, and Wake counties are within a 16.1-km (10-mi.) radius of the HAR site. These counties have a combined population of 676,770 that uses groundwater and surface water from a public water source (Table 2.4.1-210). Public water supplies draw about 336.9 million liters (l) (89 million gallons [gal.]) of water per day from surface waters. There are no known private surface water withdrawals for domestic water supply. Additional surface water withdrawals within a 16.1-km (10-mi.) radius of the HAR site include the following (Reference 2.4-217):

- 3.7 million liters per day (l/day) (1 million gallons per day [gpd]) for livestock purposes.
- 64.4 million l/day (17 million gpd) for irrigation purposes.
- 7.6 million l/day (2 million gpd) for industrial purposes.
- 1,465 million l/day (387 million gpd) for thermoelectric power using once-through technology.

Counties within an 80.5-km (50-mi.) radius of the HAR site have a combined population of 2,148,750 that uses groundwater and surface water from public water sources (Table 2.4.1-210). Public water supplies draw about 1,207.5 million l/day (319 million gpd) from surface waters. There are no known private surface water withdrawals for domestic water supply. Approximate surface water withdrawals within an 80.5-km (50-mi.) radius of the HAR site include the following (Reference 2.4-217):

- 45.4 l/day (12 million gpd) of surface water for livestock purposes.
- 355.8 l/day (94 million gpd) of surface water for irrigation purposes.
- 26.5 l/day (7 million gpd) for industrial purposes.

2.4.2 FLOODS

2.4.2.1 Flood History

HAR COL 2.4-2

The yearly peak streamflow and average daily streamflow measurements recorded at the Buckhorn Creek gauging station (USGS# 02102192) from 1972 to 2005 (34 years) indicate the Auxiliary Reservoir and the Main Reservoir are significantly attenuating flood flows in Buckhorn Creek (Tables 2.4.2-203 and 2.4.1-202, and Figure 2.4.2-201). Prior to completion of the Auxiliary Dam and Main Dam Structures in late 1980, the average yearly peak streamflow was 80.5 m^3/s (2841 cfs), with a peak streamflow of 196.0 m^3/s (6920 cfs) occurring

on February 2, 1973; the maximum recorded average daily streamflow of 88.6 m^3/s (3130 cfs) also occurred on February 2, 1973. Following completion of Harris Lake, the average yearly peak streamflow was 29.4 m^3/s (1038 cfs), with a peak streamflow of 121.8 m^3/s (4300 cfs) occurring on September 6, 1996; the maximum recorded average daily streamflow of 54.9 m^3/s (1940 cfs) also occurred on September 6, 1996 (Reference 2.4-201).

Yearly peak streamflow and average daily streamflow measurements recorded at the Cape Fear River at Lillington gauging station (USGS# 02102500) from 1924 to 2005 (82 years) were also reviewed (Tables 2.4.2-204 and 2.4.1-205, and Figure 2.4.2-202) as follows (Reference 2.4-271):

- The average yearly peak streamflow based on available data was 1153.5 m^3/s (40,736 cfs), with a peak streamflow of 4247.5 m^3/s (150,000 cfs) occurring on September 19, 1945.
- The maximum recorded average daily streamflow of 3964.4 m^3/s (140,000 cfs) also occurred on September 19, 1945.

Starting in 1973, the discharge or streamflow of the Cape Fear River has been affected by regulation or diversion (Reference 2.4-271).

- Prior to 1973, the average yearly peak streamflow was 1362.6 m^3/s (48,118 cfs).
- After 1972, the average yearly peak streamflow has been 843.1 m^3/s (29,774 cfs).

Figures 2.4.2-203 and 2.4.2-204 show the flood frequency analysis curves created using the Log Pearson Type III Distribution statistical technique for the Buckhorn Creek and the Cape Fear River at Lillington gauging stations. Table 2.4.2-201 shows calculated recurrence intervals of 2.33, 10, 25, 50, and 100 years, and associated streamflows for each gauging station. The maximum recorded average daily discharge of 88.6 m^3/s (3130 cfs) at the Buckhorn Creek gauging station has a calculated recurrence interval of about 40.5 years. At the Cape Fear River at Lillington gauging station, the maximum recorded average daily discharge of 3964.4 m^3/s (140,000 cfs) has a calculated recurrence interval of about 678.5 years. The highest maximum average daily discharge after 1945 was 2024.7 m^3/s (71,500 cfs), which has a calculated recurrence interval of about 21.5 years. The unusually high yearly peak streamflow measurements (greater than 1982.2 m^3/s [70,000 cfs]) at the Cape Fear River at Lillington gauging station occurred prior to 1952. These elevated measurements are likely attributed to land usage and to fewer locks and dams built on the Cape Fear River to attenuate flood events.

2.4.2.2 Flood Design Considerations

Safety-related structures and facilities for the HAR 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 Subsection 2.4.3, Probable Maximum Flood on Streams and Rivers, and Subsection 2.4.5, Probable Maximum Surge and Seiche Flooding, 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.

None of the nonsafety-related structures, systems, and components are important based on flooding considerations. As a result, nonsafety-related structures, systems, and components are not important for mitigating flood events, and 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 HAR 2 and HAR 3.

The tops of the Main Dam and the Auxiliary Dam (79.2 m [260 ft.] and 79.2 m [260 ft.] NGVD29, respectively) are above the PMF water level in the reservoirs as indicated in Subsection 2.4.3, Probable Maximum Flood on Streams and Rivers. Subsection 2.4.1.1 discusses additional structural information pertaining to the dams and spillways.

The PMF in the Cape Fear River is not considered because of the large difference in elevation between the river bank (approximate elevation 48.8 m [160 ft.] NGVD29) and the top of the Main Dam (elevation 79.2 m [260 ft.] NGVD29), which is the nearest seismic Category I structure to the river. A layer of oversized rock protects the downstream face of the Main Dam from possible wind-wave action whenever the Buckhorn Creek backwater reaches the Main Dam. No specific design basis exists for downstream slope protection of the Main Dam. The 500-year flood water elevation for the Cape Fear River is estimated to be 55.5 m (182 ft.). The Cape Fear River 500-year flood backwater effect on Buckhorn Creek near the downstream face of the Main Dam is not expected to result in wave action on the dam. Therefore, because a small downstream fetch severely limits the size of wind-generated waves, the rockfill shell does not require special slope protection.

The oversized rock zone on the downstream face is primarily a construction-related feature. When the Main Dam was being constructed, oversized rocks were removed from each of the rockfill lifts to meet specifications. Where the oversized rocks were within practical limits (50.8 to 76.2 centimeters [cm] [20 to 30 inches (in.)]), they were placed near the downstream face to provide additional protection for the Main Dam.

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 Revision 19, 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 a 50-year precipitation event will consist of conveying water from roof gutters and/or scuppers, as well as runoff from the HAR 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 plant site is drained by overland flow on open roads and ground surface away from the safety-related structures and directly to the Main Reservoir, the Auxiliary Reservoir, or the Emergency Service Water Intake and Discharge Channels.

The proposed site grade elevation for the HAR site is 79.25 m (260 ft.) NGVD29. The plant site area is bounded by the Main Reservoir on the east and the Auxiliary Reservoir on the west. The proposed normal water elevation for the Main Reservoir will be 73.2 m (240.0 ft.) NGVD29, which will be 6.4 m (21 ft.) below the plant floor elevation of 79.55 m (261 ft.). The normal water level elevation in the Auxiliary Reservoir is 76.8 m (252 ft.) NGVD29, which is 2.7 m (9 ft.) below the plant floor elevation. The proposed floor elevation for facilities associated with HAR 2 and HAR 3 is 79.6 m (261 ft.) NGVD29.

Figure 2.4.2-205 presents the conceptual grading and drainage of the HAR site, which is subdivided into Zones A through D. Most of the runoff from a local PMP event on the site area will drain toward the Main Reservoir on the east and toward the Auxiliary Reservoir on the west and southwest.

Zone A consists of portions of the HAR 2 and HAR 3 area east of the power block. Zone A drains east towards the Main Reservoir.

Zone B, which is located to the west of HAR 2 and HAR 3, is graded and sloped to allow runoff to drain south and west toward the Emergency Service Water Discharge Channel for the HNP and the Auxiliary Reservoir. The drainage area north of Zone B has an existing grade elevation varying from 88.4 m (290 ft.) NGVD29 to 82.3 m (270 ft.) NGVD29, and drains west toward the Auxiliary Reservoir.

Zone C, which is located north of the HAR site, is graded at a higher elevation. The high point of the construction parking area within Zone C is at an elevation of 82.3 m (270 ft.) NGVD29. Zone C is graded and sloped to allow runoff from a local PMP event to drain east towards the Main Reservoir. Similarly, the office and warehouse area within Zone C, which is located immediately north of HAR 3, is graded and sloped to drain east towards the Main Reservoir.

Zone D, located north of Zone C, has a drainage area of 20 acres, which is used for construction parking and as the HAR 3 switchyard access road. A grade elevation along the north boundary of this zone is at or above the elevation of 88.4 m (290.0 ft.) along the existing landfill. The southern boundary of the zone is at 82.3 m (270 ft.), which also acts as the northern boundary for Zone C. The PMP runoff for this zone will flow eastward and eventually flow to the Main Reservoir through the low lying area between the existing landfill and HAR Unit 3 switchyard. The HAR 3 switchyard is graded to slope towards the north and eventually drains to the Main Reservoir.

The local intense PMP is defined by Hydrometeorological Report (HMR) No. 52 (Reference 2.4-218). 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 52. As indicated in HMR 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 Figures 36, 37, and 38.

PMP values for a 2.6-km^2 (1-mi.^{2}) area are shown in Table 2.4.2-202. Figures 2.4.2-206 and 2.4.2-207 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 2.4.2-202, 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.

$$D = \frac{4.579 + 25.517T}{1 + 0.566T - 0.003T^2}$$

where

D = depth (in.), and

T = duration (hr.).

The AP1000 design is based on a PMP of 49.3 cm/hr (19.4 in/hr) and 16.0 cm/5 min (6.3 in/5 min). The estimated 2.6-km^2 (1-mi.^{2}) PMP values for the HAR site are 48.0 cm/hr (18.9 in/hr) and 15.7 cm/5 min (6.2 in/5 min)

(Table 2.4.2-202). Therefore, the HAR site is within the plant design limits for PMP.

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 2.4-219):

$$Q = C\ I\ A$$

where

Q= peak runoff in cfs,

C = coefficient of runoff,

I = intensity of rainfall in in/hr, and

A = drainage area in ac.

The times of concentration for each drainage zone are estimated and the corresponding intensities are determined. The value of C is conservatively assumed to be 1.0.

Water levels corresponding to runoff associated with a local PMP event for drainage zones A and B identified in Figure 2.4.2-205 are estimated by performing a backwater analysis using HEC-RAS (Reference 2.4-275) computer program assuming the downstream boundary condition as critical flow depth.

The drainage area for Zone A is 84.3 ac., which includes a portion of offices and warehouses north of HAR 3 and south of the high point (HP) elevation of 80.6 m (264.5 ft.) NGVD29. PMP runoff from Zone A will flow to the east and will drain into the Main Reservoir. For the computation using HEC-RAS, Zone A was modeled with 15 cross-sections running north south and perpendicular to the flow direction. The cooling tower area and the building areas within the power block were modeled as no-flow portions of the respective cross sections. The north and south boundaries of Zone A were considered as no-flow boundaries. The east boundary, adjacent to the Main Reservoir, was modeled as the downstream boundary with critical flow depth. The western-most cross-section along the reactor centerline was modeled as the upstream boundary for this zone. The computed peak runoff (Q) at the downstream cross-section for this zone was 2884 cfs. Peak discharges for other cross sections were computed by prorating the downstream peak discharge in the ratio of upstream drainage area at the cross-section to the maximum drainage area.

The drainage area for Zone B is 54.6 ac. PMP runoff from Zone B will flow to the southwest over the 365.76 m (1200 ft.) long railroad track and will drain into the auxiliary reservoir through the emergency service water discharge channel for the existing HNP. For the computation using HEC-RAS, Zone B was modeled using 11 cross sections running north-south near the reactor building to

northwest-southeast near the downstream boundary. Cross sections were modeled perpendicular to the flow direction. All boundaries except upstream and downstream boundaries were modeled as no-flow boundaries. The downstream boundary of 365.76 m (1200 ft.) of railroad track adjacent to the discharge channel was modeled as the boundary with critical flow depth. The upstream cross-section along the reactor centerline was modeled as the upstream boundary for this zone. Areas upstream of each cross-section were measured and identified as the contributing drainage area for the respective cross section. The computed peak runoff discharge (Q) at the downstream cross section for this zone was 1988 cfs. Peak discharges for other cross-sections were computed by prorating the downstream peak discharge in the ratio of upstream drainage area at the cross-section to the maximum drainage area.

The maximum elevation of the calculated water level in Zone A is 79.4 m (260.62 ft.) NGVD29 and in Zone B is 79.5 m (260.76 ft.) NGVD29, which are below the proposed plant floor elevation of 79.6 m (261 ft.) NGVD29 for HAR 2 and HAR 3. Therefore, flooding caused by a local PMP event within Zones A and B will not affect the safety-related facilities associated with HAR 2 and HAR 3.

The drainage area for Zone C is 23.8 ac. Zone C is graded at a higher elevation; however, the area is graded and sloped such that runoff due to PMP on this area will drain east to the Main Reservoir and the flow from this zone does not affect the power block area in Zones A and B. Therefore, backwater computation for Zone C was not performed using the HEC-RAS program. However, the maximum water level at the upstream end of this zone was calculated using a conservative analysis based on one typical cross-section and downstream critical depth. The computed peak runoff for this zone was 714 cfs. The computed maximum water level in this zone was 80.3 m (263.6 ft.), which was below the HP elevation of 80.6 m (264.5 ft.). This HP acts as a barrier between this zone and the adjacent Zone A. Therefore, flow from this zone will drain east to the Main Reservoir and will not enter the adjacent areas in Zones A and B. The safety-related facilities in Zones A and B (HAR 2 and HAR 3) are not affected by PMP runoff in Zone C.

The drainage area for Zone D is 20.0 ac. Zone D is graded at a higher elevation; however, Zone D is graded to drain to the north through the low area between the HAR 3 switchyard and the existing landfill and eventually drains to the Main Reservoir. PMP flow from this zone does not affect the power block area in Zones A and B. As a result, backwater computation for Zone D was not performed using the HEC-RAS program. However, the maximum water level at the upstream end of this zone was calculated using a conservative analysis based on one typical cross section and downstream critical depth. The computed peak runoff for this zone was 750 cfs. The computed maximum water level in this zone was 82.0 m (269.0 ft.), which is below the HP elevation of 82.3 m (270.0 ft.). This HP acts as a barrier between this zone and the adjacent Zone C. Therefore, flow from this zone will drain east to the Main Reservoir and will not enter the adjacent area in Zone C.

The water levels, flow velocities, and Froude Numbers at different cross-sections for Zones A and B obtained from HEC-RAS computation are shown in Table 2.4.2-205.

The velocity of overland flow due to local PMP, in the east area near the power block, is less than 0.64 m/sec (2.1 ft/sec) and the maximum velocity downstream of the power block area is less than 1.22 m/sec (4.0 ft/sec). Similarly for the west area, the maximum flow velocity in the power block area is less than 0.30 m/sec (1 ft/sec) and less than 1.22 m/sec (4.0 ft/sec) for areas downstream. Therefore, the potential for erosion near the safety-related facilities is minimal. Minor erosion due to 1.22 m/sec (4.0 ft/sec) velocity will occur at locations far away from the safety-related facilities and therefore will not impact the plant buildings and the safety-related facilities in the power block area.

A Froude number value of 1.0 or less indicates critical or sub-critical flow. As shown in Table 2.4.2-205, the Froude number is nearly equal to 1 or less for all of the cross-sections and therefore, there is no potential for a hydraulic jump near any cross-section.

During the final design of the site grading and drainage, any roads and railroads in the path of surface runoff from a local PMP event that could adversely affect the water levels near the safety-related facilities will be lowered to elevation 79.2 m (260 ft.) NGVD29 or lower, so that those facilities will not be affected by local PMP on the site area.

The storm drainage ditches, which carry normal stormwater runoff from the site, are not considered in the PMP analysis. However, the local drainage system will be maintained to provide conveyance as designed throughout the operational life of the HAR units.

A review of historical rainfall records from the National Weather Service (NWS) Cooperative Observer Station No. 317069 in Raleigh, North Carolina, for the period 1971 to 2000, indicates monthly mean precipitation ranges between 7.11 and 10.90 cm (2.80 and 4.29 in.) and the annual mean precipitation is 109.35 cm (43.05 in.) (Reference 2.4-221). Monthly extremes were calculated from 1948 to 2001 from the station's available digital record. The highest daily precipitation of 12.60 cm (4.96 in.) occurred on September 6, 1996, and the highest monthly precipitation of 55.35 cm (21.79 in.) occurred during September 1999. Subsection 2.4.3.1 provides information pertaining to the PMP for the Buckhorn Creek Drainage basin. Because the HAR 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

HAR COL 2.4-2 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 2.4-222). The PMF represents an estimated upper bound on the maximum runoff potential for a given watershed. 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 PMFs for the HAR site were developed using the following steps:

a. The crest elevation of the existing uncontrolled ogee spillway on the Main Dam is 67.1 m (220 ft.) NGVD29. It is proposed that this crest will be raised to elevation of 73.2 m (240 ft.) NGVD29. In addition, an emergency spillway will be provided west of the existing spillway. The first step for determining the PMF is to delineate the sub-basins of the Buckhorn Creek drainage basin above the Main Dam. Considering these proposed modifications in the spillway of the Main Dam, the drainage basin above the dam is 182.1 km^2 (70.3 mi.^{2}) (Figure 2.4.3-201), wherein the area inundated is 30.8 km^2 (11.9 mi.^{2}) or about 16 percent of the entire basin. The Buckhorn Creek drainage basin above the Main Dam was divided into seven sub-basins (Sub-basin IV through Sub-basin X).

b. The unit-hydrograph theory was used as the runoff model for developing runoff hydrographs for each sub-basin, except the Auxiliary Reservoir and Main Reservoir pool surfaces. Therefore, various parameters required for developing unit hydrographs for the sub-basins were determined. Using these parameters, unit hydrographs were developed for each sub-basin. For the Auxiliary Reservoir and Main Reservoir pool surfaces, the direct rainfall was assumed to be equal to the runoff without any loss and lag.

c. The PMP storm hyetograph was determined for the Buckhorn Creek drainage basin using criteria and step-by-step instructions given in HMR 51 (Reference 2.4-223) and HMR 52 (Reference 2.4-218). The developed PMP storm hyetograph was applied to the unit hydrographs with the appropriate infiltration losses to develop the estimated flood hydrographs for each sub-basin, as well as for the entire drainage basin.

d. Based on the requirements of American National Standards Institute/American Nuclear Society (ANSI/ANS)-2.8-1992, Section 9.2.1.1, an antecedent 72-hour storm having a volume of 40 percent of the PMP was assumed (Reference 2.4-222). This antecedent storm was assumed to be followed by 72 hours of no rain. Then the full 72-hour PMP storm followed. This was the complete PMP storm that was applied to the unit hydrographs with appropriate infiltration losses to develop the estimated flood hydrograph for each sub-basin. These flood hydrographs were used to estimate the PMF stillwater level in the Main Reservoir and in the Auxiliary Reservoir.

e. Inflow hydrographs from various sub-basins upstream of the Main Dam were added together without conducting reach routing using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model to determine the combined inflow to the Main Dam (Reference 2.4-224).

f. After obtaining the combined inflow hydrograph, the PMF hydrograph was routed through the reservoir, spillway, and outlet works using the level pool reservoir routing method to estimate the maximum PMF stillwater level in the reservoirs.

The following discussions are based on the guidance presented in Regulatory Guide 1.206, Revision 0.

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 2.4-225). In other words, the PMP is the estimated depth of precipitation for which there is virtually no risk of exceedance (Reference 2.4-222). The PMP depths used in this study were calculated using the criteria and step-by-step instructions given in HMR 51 (Reference 2.4-223) and HMR 52 (Reference 2.4-218).

Generally, a three-step process is followed for determining PMP in nonorographic regions: moisture maximization, transposition, and envelopment.

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, 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 are available in the form of generalized plots on Figures 18 through 47 in HMR 51 (Reference 2.4-223).

The drainage area for the Buckhorn Creek watershed above the Main Dam (Figure 2.4.3-201) is 182.1 km^2 (70.3 mi.^{2}) and the location of the centroid of the

basin is approximately 35°38'00" N, 78°57'22'' W. Using HMR 52 as a guide, the PMP for the Buckhorn Creek drainage basin was developed using the following steps (Reference 2.4-218):

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 Basin above the Main Dam.

f. Development of Design Storm for Drainage Basin above the Auxiliary Dam.

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 2.4-223) were obtained for various-sized areas, both larger and smaller than the drainage area under study for the Buckhorn Creek watershed. Table 2.4.3-201 provides the 6-hour incremental depth-area-duration data taken from Figures 18 through 47 of HMR 51 (Reference 2.4-223). Using the data presented in Table 2.4.3-201, the smooth depth-area-duration curves for the Buckhorn Creek drainage basin above the Main Dam were plotted on Figure 2.4.3-202.

This initial plotting of the basic input data serves the following two functions:

- It eliminates reader errors from basic misinterpretation of values in the figures in HMR 51 (Reference 2.4-223).
- 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 10, 25, 50, 100, 175, 300, and 450 $mi.^2$. Table 2.4.3-203 tabulates the depth-area-duration values for the selected standard isohyet areas.

The depth-area-duration data for the selected standard areas from Table 2.4.3-203 were plotted on a linear paper, and smooth curves were fitted as shown on Figure 2.4.3-203. From Figure 2.4.3-203, the PMP values corresponding to an 18-hour duration were read as tabulated in Table 2.4.3-204. Incremental differences for the first three 6-hour periods were obtained by

successive subtraction of the values contained in Tables 2.4.3-203 and 2.4.3-204. Table 2.4.3-205 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 on Figure 2.4.3-204. Using the smooth curves from Figure 2.4.3-204, the data in Table 2.4.3-206 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 particular geographic location. That orientation is related to the general movement of storm systems and the direction of moisture-bearing winds. Based on contours of preferred orientation shown on Figure 8 of HMR 52, the preferred orientation for storms at the location having its latitude 35°38’00” N and longitude 78°57’22” W was 200 degrees (Reference 2.4-218). The orientation of the storm pattern to produce maximum precipitation volume in the watershed was found to be approximately 215 degrees. The angular difference in the orientations is 15 degrees, which is less than 40 degrees. This indicates that no adjustment is required in the incremental storm pattern 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 Buckhorn Creek drainage basin above the Main 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 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 2.4-218) 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 259 km^2 (100 $mi.^2$).

2.4.3.1.4 Distribution of Storm-Area Averaged PMP over the Drainage Basin

It was concluded that the maximum volume occurs for a PMP pattern near 259 km^2 (100 $mi.^2$) when placed over the Buckhorn Creek watershed. 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 Buckhorn Creek watershed with a 259-km^2 (100-$mi.^2$) 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 2.4-222). Total rainfall for the 72-hour duration was found to be 99.7 cm (39.24 in.). The resulting hourly PMP rainfall distribution is tabulated in Table 2.4.3-216 and plotted on Figure 2.4.3-205.

2.4.3.1.5 Development of Design Storm for Basin above the Main Dam

Using the PMP rainfall distribution tabulated in Table 2.4.3-216 and shown on Figure 2.4.3-205, 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 2.4-222). This design storm, which was used as the rainfall input in the hydrologic modeling, has 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 these three components, Figure 2.4.3-206 shows the resulting design storm rainfall data that were developed for the basin above the Main Dam.

2.4.3.1.6 Development of Design Storm for Drainage Basin above the Auxiliary Dam

The total drainage area of the Auxiliary Reservoir watershed is 7.8 km^2 (3 $mi.^2$). The smallest area considered in HMR 52 is 26.0 km^2 (10 $mi.^2$), with a 72-hour PMP of about 119.6 cm (47.10 in.) (Reference 2.4-218). Extrapolating depth-area-duration curves of Figure 2.4.3-202 for a drainage area of 7.8 km^2 (3 $mi.^2$), the 72-hour PMP for the drainage basin above the Auxiliary Dam was found to be 126.62 cm (49.85 in.). Using the temporal distribution of the design storm above the Main Dam (Figure 2.4.3-206), the design storm for the drainage basin above the Auxiliary Dam was determined. The resulting hourly PMP rainfall distribution for the Auxiliary Dam is presented in Table 2.4.3-216 and plotted on Figure 2.4.3-207. Combining the three components described in Subsection 2.4.3.1.5, Figure 2.4.3-208 depicts the resulting design storm rainfall input that was developed.

2.4.3.2 Precipitation Losses

This subsection describes the methodology used to assign 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 amount of rainfall loss can be characterized by various methods; the loss methods and their parameters are selected in accordance with recognizable characteristics of the drainage basin under study. The HEC-HMS model offers several methods for estimating precipitation losses. However, the exponential loss rate method that was used in the HNP FSAR is not included among the available loss methods. Thus, the optimized loss parameters used in the HNP FSAR could not be used in the present study.

The traditional initial and constant loss rate method for PMF computations was selected from the HEC-HMS model precipitation loss methods based on Federal Energy Regulatory Commission (FERC) recommendations (Reference 2.4-226). The following assumptions were made:

- Saturated antecedent conditions existed in the entire watershed prior to the start of the PMP.
- The initial loss for the sub-basins was zero inches (conservative assumption).
- Infiltration occurs at the minimum rate (for consistency with saturated soil conditions).

To determine the minimum infiltration rate, the average soil type for each sub-basin was determined. The land use in the study basin is primarily forested game lands throughout the watershed, with some transitional and urban areas well beyond the major watershed. FSAR Subsection 2.4.12.1.2.1 provides a detailed description of soil types in the study basin. Table 2.4.12-201 briefly summarizes the soil types in the Buckhorn Creek watershed. The study basin contains primarily three soil types: Creedmoor, Mayodan, and White Store. The U.S. Department of Agriculture (USDA) soil texture can be described approximately as sandy clay loam that falls into hydraulic soil group “C.” The HAR site is classified as heavy industrial, and the remaining area of the Buckhorn Creek watershed can be classified as approximately 85 percent forest and 15 percent transitional lands.

The USDA soil texture at the HAR site can be described as approximately sandy clay loam that falls into hydraulic soil group “C.” Based on TR-55 (References 2.4-227 and 2.4-228), the range of infiltration rates for the hydrologic soil group “C” is 0.05 to 0.15 in/hr. To ensure that the PMF estimate is both conservative and representative of the site, the PMF analysis is performed by taking credit for an initial infiltration loss of 0.15 in/hr, which then decreases linearly to zero at the end of 72 hours of the antecedent storm. During the hours of 72 through 144

when there is no rainfall but soil is still saturated and depressions are full, the infiltration loss rate is assumed to be zero. During the full PMP event, which includes the hours of 72 and after, the infiltration loss rate is assumed to be zero.

Given these conditions, the maximum potential loss rate due to infiltration, *Aloss*, is described by the following formula:

$$Aloss = 0.15\left(1 - \frac{t}{72}\right) \quad for \ 0 \le t \le 72$$

$$Aloss = 0.0 \quad for \ t > 72$$

where

Aloss = infiltration loss rate (in/hr), and

t = time (hr).

The above equation gives the maximum potential infiltration loss rate. Further, the actual rate of loss due to infiltration during the 72-hour antecedent storm is then calculated as the minimum of (1) the maximum potential infiltration rate given by the above formula, and (2) the rate of rainfall during a given hour of the 72-hour antecedent storm:

$Loss_{actual}$ = min[Aloss(t), rainfall(t)]

Figure 2.4.3-209 depicts the actual infiltration rate during the 72-hour antecedent storm and the two succeeding 72-hour periods of the PMP event for both the design storms associated with the Main Reservoir and the Auxiliary Reservoir drainage basins. It can be noted from Figure 2.4.3-209 that the actual infiltration rate is constrained only by precipitation levels for the first 24 hours of the antecedent storm. However, from hours 24 through 72, the infiltration rate is constrained by the potential maximum rate, as previously described. The infiltration rate is assumed to be zero after hour 72.

The infiltration rate represents a precipitation loss. The traditional initial and constant loss rate method was used as the precipitation loss method while using the HEC-HMS model for conducting the PMF computations. However, it is difficult to incorporate the infiltration loss given by the above equations in a single continuous run of the HEC-HMS model. Therefore, the effective rainfall has been calculated outside the HEC-HMS model. Effective precipitation values are determined by subtracting infiltration from actual precipitation as follows:

Effective Precipitation = Actual Precipitation – Infiltration Loss

Actual and effective precipitations, calculated using the above procedure, for both the Main and Auxiliary drainage basins are shown on Figures 2.4.3-210 and 2.4.3-211. Note that actual and effective precipitations are equal when the

infiltration rate is zero, which is the case for hour 72 and after. Therefore, only the 72-hour antecedent storm is shown on Figures 2.4.3-210 and 2.4.3-211. Table 2.4.3-217 shows a tabular list of the effective rainfall values for both the Main and Auxiliary Reservoirs. Total losses due to infiltration as a percentage of precipitation were 22.6 percent (3.55 in.) and 19.5 percent (3.88 in.) for the Main and Auxiliary Reservoirs, respectively.

This procedure was not used for the pool areas of the Main and Auxiliary Reservoirs where 100 percent of the rainfall was converted into direct runoff.

As previously described, the infiltration loss rate was applied outside the HEC-HMS model. The input infiltration loss rate parameters for various sub-basins were assumed to be zero in the HEC-HMS model, as shown in Table 2.4.3-218. In addition, Table 2.4.3-218 lists loss parameters for various sub-basins of the Buckhorn Creek watershed above the Main Dam.

2.4.3.3 Runoff and Stream Course Models

A runoff model is used to transform excess precipitation into surface runoff. For the purpose of this analysis, runoff was modeled using two different methods: one for rain falling on land surfaces, and a second for rain falling directly on reservoir pool surfaces. The runoff modeling approach is generally described as follows:

- Land Surface Areas - Unit hydrographs were applied to transform excess rainfall over land surface areas into runoff.
- Reservoir Pool Surface Areas - Precipitation falling directly over reservoir pool areas was converted into runoff without considering any infiltration loss or lag time.
- No reach routing was used; traveling time of runoff from land areas into the reservoir was neglected.
- Level pool routing was used to determine the PMF elevations in both the Main and Auxiliary Reservoirs.

2.4.3.3.1 Runoff Model

An overland runoff model 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 sub-basin. Unit hydrographs are combined with precipitation data to determine the direct runoff hydrograph for a given storm event in a particular basin. Thus, separate unit hydrographs are developed for each sub-basin using their specific hydrologic parameters.

Several different methods can be used to develop a unit hydrograph for a given sub-basin. Selection of an appropriate method depends on knowledge of its hydrologic response characteristics. Based on the hydrologic characteristics of the Buckhorn Creek drainage basin, the Snyder hydrograph method was selected as acceptable. The required hydrologic parameters for developing the Snyder's synthetic unit hydrographs were readily available. The HNP FSAR calculated the required generalized values of the shape coefficients that are empirical in nature. The other parameters of the Snyder's method can be determined from the geometry of each sub-basin.

The following information summarizes the Snyder's synthetic hydrograph method. The Snyder unit hydrograph relationships define only the unit hydrograph peak discharge (Q_P) and the lag time (t_L) that are defined as (Reference 2.4-226):

$$t_L = CC_t\left(LL_C\right)^{0.3} \tag{1}$$

$$Q_P = \frac{640 C_P A}{t_L} \tag{2}$$

where

L = flow path length from outlet to the hydraulically farthest point (basin divide),

L_C = flow path length from outlet to sub-basin centroid,
C_t = Snyder basin lag coefficient, and

C_P = Snyder peaking coefficient.

The parameters C_t and C_P are strictly empirical values often recommended as applicable to a specific region. C_t accounts for storage and shape of the watershed, and C_P is a function of flood-wave velocity and storage. The generalized values of C_t and C_P as given in the HNP FSAR are 3.91 and 0.75, respectively.

To apply the unit hydrograph approach to the Buckhorn Creek drainage basin, unit hydrographs were developed for three surfaces: (1) Main Reservoir pool surface, (2) Auxiliary Reservoir pool surface, and (3) Residual Land Surface around the Main Reservoir and the seven sub-basins in the Buckhorn Creek drainage basin above the Main Dam. Figure 2.4.3-201 shows Buckhorn Creek drainage sub-basin areas above the Main Dam. This figure illustrates that Sub-basins I, II, and III fall below the Main Dam spillway. Therefore, these sub-basins were not considered in the drainage area at the Main Dam. Excluding these sub-basins, the total drainage area at the Main Dam is 182.1 km^2 (70.3 $mi.^2$). This area also includes the drainage area at the Auxiliary Reservoir.

Table 2.4.3-219 lists the drainage areas of the Auxiliary Reservoir Surface, Main Reservoir Surface, Residual Land Surface, and Sub-basins IV, V, VI, VII, VIII, IX, and X.

A unit hydrograph has meaning only in connection with a specific duration of runoff. A sub-basin may have many different unit hydrographs, each associated with a different duration of runoff. Haan et al. recommend that the duration D of a unit hydrograph should be between $T_P/5$ and $T_P/3$, where T_P is the time to peak (Reference 2.4-229). Further, T_P is a function of D and catchment lag time T_L, defined as $T_P = T_L + D/2$ (Reference 2.4-260). However, for the Snyder's synthetic unit hydrograph, $D = T_L/5.5$. The catchment lag is a parameter used in unit hydrograph theory to provide a global measure of the response time of a catchment area. Since this global parameter incorporates various basin characteristics, such as hydraulic length, gradient, drainage density, and drainage patterns to determine these characteristics, it is necessary to delineate the sub-basins according to their drainage pattern as shown on Figure 2.4.3-201. Table 2.4.3-220 lists various watershed parameters, along with the Snyder Hydrograph parameters used in the HEC-HMS model.

More conservative alternate parameters were used for the residual area. A lag time of 10.6 hours was obtained by substituting the geometric characteristics associated with the land area surrounding the Main Reservoir in the Snyder's unit hydrograph equations. To increase conservatism, the calculated lag time was reduced from 10.6 hours to 1.7 hours by assuming a coefficient of L = 0.4 and of L_C = 0.15 in Equation (1). By decreasing the lag time, the peak flow increases from 796 cfs to 4,992 cfs within the residual area.

Using the standard Snyder hydrograph parameters and the more conservative lag time and peak flow parameters for the residual area presented in Table 2.4.3-220 as input in HEC-HMS model, 1-hour unit hydrographs were developed, as shown on Figure 2.4.3-212. The parameters associated with the 1-hour hydrographs for each basin are provided in Table 2.4.3-221.

2.4.3.3.2 Hydrograph Peaking

In order to ensure safety of the HAR site against flooding, the degree of conservatism associated with the peak flow calculation was determined. For this purpose, several storm events smaller than the PMP storm, but sufficient to cause out-of-bank flooding, were used. For these storm events, both rainfall and expected runoff are known. Using the hyetographs of these storm events, along with unit hydrograph parameters used in the PMF inflow calculation as presented in Table 2.4.3-221, peak flows were determined using the developed HEC-HMS model used for the PMF analysis. The obtained peak flows were compared with peak flows determined using the peak flow equations developed by the USGS for rural basins in North Carolina (Reference 2.4-261). The predictive error associated with these equations is known. In order to produce the most conservative estimate, the peak flows generated by these equations have been corrected by adding the known predictive errors (that is, erring in the positive direction). The resulting peak

flow values were then compared with the results generated by the HEC-HMS model for various storm events (Table 2.4.3-222) without making any change in the HEC-HMS parameters used for the PMP storm event.

As shown in Table 2.4.3-222, the estimated magnitude of peak flow events generated by the HEC-HMS model exceeds the corrected peak flows predicted using the USGS equations by more than 30 percent in all cases. This comparison serves to emphasize the degree to which the HEC-HMS computed peak flows are conservative. In other words, the difference between the magnitude of peak flows generated by the HEC-HMS model and peak flows obtained using USGS flow equations (Reference 2.4-261) can be considered as the implicit peaking factors.

In order to comply with the recommendation of ER 1110-8-2(FR) (Reference 2.4-262), the 1-hour base unit hydrographs that were developed for the FSAR analysis using the Snyder method were peaked. That is, the unit hydrographs (Figure 2.4.3-212) were adjusted such that the peak flows were increased by 25 percent, while the unit volume of each unit hydrograph was maintained. Given these adjustments, the appropriate time base and lag times of the peaked unit hydrographs were determined. The revised parameters associated with the peaked unit hydrographs are listed in Table 2.4.3-223. (Refer to Table 2.4.3-221 for the 1-hour base unit hydrograph parameters.)

A comparison of the base unit hydrograph to the peaked unit hydrograph for Sub-basin X is shown on Figure 2.4.3-213, while the peaked unit hydrographs for all sub-basins are shown on Figure 2.4.3-214.

Using the PMP storms above the Main Dam (Figure 2.4.3-206) and Auxiliary Dam (Figure 2.4.3-208), along with the developed unit hydrographs for various sub-basins for both cases considering peaking and no peaking, inflow hydrographs to the Auxiliary and Main Reservoirs were determined using the HEC-HMS model. Figures 2.4.3-215 and 2.4.3-216 show inflow hydrographs to the Auxiliary and Main Reservoirs for both cases with and without considering hydrograph peaking. Table 2.4.3-224 provides a summary of inflow results for both the Auxiliary and Main Reservoirs considering 25 percent peaking and no peaking for the PMP event.

2.4.3.3.3 Basin Data

Basin data include the elements of the basin, their connectivity, runoff, storage, and discharge relationships of hydraulic structures, and routing parameters of stream reaches and reservoirs. Figure 2.4.3-217 presents a schematic of the Buckhorn Creek drainage basin above the Main Dam and its elements, along with their connectivity.

Figures 2.4.1-206 and 2.4.1-207 present the stage-storage-area curves for the Main Reservoir and the Auxiliary Reservoir, respectively. Survey data from the Topographic Maps and Digital Ortho Photos from Barton Aerial Technologies

(BAT) were used to develop the stage-storage-area curves for the Main Reservoir. The stage-storage-area curves for the Auxiliary Reservoir were obtained from the HNP FSAR.

2.4.3.3.3.1 Existing Spillways

Both the Main Dam and the Auxiliary Dam have uncontrolled ogee spillways. The crest of the Main Dam spillway is at elevation 67.1 m (220 ft.) NGVD29, and the crest of the Auxiliary Dam spillway is at elevation 76.8 m (252 ft.) NGVD29. The elevation of the top of both dams is 79.2 m (260 ft.) NGVD29. The spillway crest at the Main Dam has a net length of 15.2 m (50 ft.) with a pier at its mid-length, while the spillway crest at the Auxiliary Dam has a length of 51.8 m (170 ft.). Both spillways are ogee-shaped and designed with a design head (H_0) and the upstream dam height (P) of 3.0 m (10 ft.) and 9.1 m (30 ft.), respectively, for the Main Dam spillway, while the corresponding values for the Auxiliary Dam spillway are 1.5 m (5 ft.) and 2.1 m (7 ft.), respectively.

2.4.3.3.3.2 Modification to Existing Spillway for the Main Dam

It is proposed that the normal water level (NWL) of the Main Reservoir be raised to an elevation of 73.2 m (240 ft.) NGVD29 from the existing NWL elevation of 67.1 m (220 ft.) NGVD29. In order to ensure protection of safety-related structures against external flooding and dynamic effects of wave action due to wind-generated activity, several engineering solutions were considered to modify the spillway design of the Main Dam so that the maximum water level due to coincidental effects of wind-generated setup and wave activity superimposed on PMF stillwater elevation will be below the top of Main Dam elevation of 79.2 m (260 ft.) NGVD29. The selected spillway modification for Main Reservoir includes raising Main Dam ogee spillway crest from elevation 67.1 m (220 ft.) to elevation 73.2 m (240 ft.) and providing 172.3 m (565 ft.) long emergency spillway with a crest elevation of 74.3 m (243.7 ft.). A plan and profile of the proposed modified ogee spillway are presented in Figures 2.4.3-238 and 2.4.3-239, respectively.

2.4.3.3.3.3 Emergency Spillway and Spillway Channel

To control the maximum PMF water level including wind-wave activity below the top of the Main Dam, an emergency spillway is provided along with the proposed modification to the existing Main Dam spillway. Several options for the location of the emergency spillway both on the west and east sides of the Main Dam were considered. The west abutment of the Main Dam west of the existing ogee spillway was considered an appropriate location for the emergency spillway. This location is sufficiently far from the Main Dam and the discharge from the emergency spillway and the downstream channel does not affect the Main Dam and the existing spillway. The location of the spillway and the downstream discharge channel is determined such that the discharge from the emergency spillway will flow into the existing valley or ravine and finally join Buckhorn Creek, which also acts as the downstream discharge path for the existing Main Dam spillway.

As stated in the Bureau of Reclamation's Design Standard No. 14 (Reference 2.4-276), emergency spillways are intended for use under extreme flood conditions to avoid the overtopping of dams and dikes. The crest elevation of an emergency spillway is usually designed at or above the 100-year flood level. The 100-year flood level in Harris Lake with outflow over the modified ogee spillway with a crest elevation at 73.2 m (240 ft.) NGVD29 is 74.3 m (243.66 ft.) NGVD29. Therefore the crest elevation of the emergency spillway is provided at elevation 74.3 m (243.7 ft.) NGVD29.

Using the design crest level of emergency spillway at elevation 74.3 m (243.7 ft.) NGVD29, a PMF routing analysis was performed together with the modified Main Dam ogee spillway at elevation 73.2 m (240 ft.) NGVD29. A crest length of 172.2 m (565 ft.) for the emergency spillway was estimated such that the maximum water level including wind-wave effect at Main Dam will be below the top of Main Dam elevation of 79.2 m (260 ft.) NGVD29. The emergency spillway is designed as a broad crested weir structure with top width of 6.1 m (20 ft.) and embankment slope of 5 horizontal to 1 vertical. A layout plan of the emergency spillway and downstream channel is presented in Figure 2.4.3-240. A profile along the centerline of the emergency spillway and downstream channel, and typical cross sections are provided in Figures 2.4.3-241 and 2.4.3-242.

The spillway channel crosses the Norfolk Southern Railroad alignment at approximately 152.4 m (500 ft.) downstream of the spillway. The width of emergency spillway discharge channel reduces from 172.2 m (565 ft.) immediately downstream of the emergency spillway to 76.2 m (250 ft.) at the railroad crossing. This reduction in the channel width is provided to reduce the length of the railroad bridge over the channel. The reduced 76.2 m (250 ft.) width of emergency spillway channel continues downstream of the railroad crossing until the channel daylights in the valley downstream.

To avoid the submergence effect of the emergency spillway weir due to the downstream water level, the downstream channel bottom elevation was lowered to an elevation 71.0 m (233 ft.) NGVD29, which is 3.3 m (10.7 ft.) below the crest elevation of the emergency spillway. The difference in bed elevation of the spillway channel between the upstream and downstream ends is 5.2 m (17 ft.) (elevation 233 ft. to 216 ft.) in length of approximately 335.3 m (1100 ft.), which corresponds to a uniform slope of 1.5 percent for entire length of the spillway channel. A drop structure upstream of the railroad crossing is provided to obtain a relatively flatter longitudinal slope for the channel. A channel slope of approximately 0.9 percent upstream of the drop structure and a 0.3 percent slope downstream of the drop structure are provided. A stilling basin downstream of the drop structure is provided for energy dissipation.

2.4.3.3.3.4 Discharge Rating for the Spillways

The discharge over an ogee crest is given by the following equation (Reference 2.4-230):

$$Q = CLH_e^{3/2} \tag{3}$$

where

Q = discharge (cfs),

L = effective length of crest (ft.),

H_e = total head on the spillway crest including velocity of approach (ft.), and

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 the spillway (L) is determined using the following relationship:

$$L = L' - 2(NK_P + K_a)H_e \tag{4}$$

where

L' is the net length of the spillway,

N is the number of piers, and

K_P and K_a are pier and abutment contraction coefficients, respectively. For the Main Dam spillway, $K_P = K_a = 0.01$ and N = 1. Further, $K_P = K_a = 0.01$ and N = 0 for the Auxiliary Dam.

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 2.4-230). To determine the discharge coefficients, the following relationships were developed and used in the calculations:

$$\mathrm{C} = C_0\left[0.86242043 + 0.13731086\sqrt{\frac{\mathrm{H_e}}{\mathrm{H_0}}}\right]^2 \tag{5}$$

where, C_0 is the discharge coefficient when $H_e = H_0$ and is given as:

$$C_0 = \frac{3.115674587 + 5.584120225\left(\frac{P}{H_0}\right) - 37.803292\left(\frac{P}{H_0}\right)^2 + 59.93051634\left(\frac{P}{H_0}\right)^3}{1 + 0.542416723\left(\frac{P}{H_0}\right) - 8.21524481\left(\frac{P}{H_0}\right)^2 + 14.50132694\left(\frac{P}{H_0}\right)^3 + 0.102735247\left(\frac{P}{H_0}\right)^4}$$

(6)

Using Equation (6), the corresponding values of P/H_0 and C_0 for the Main Dam are 3 and 3.95, respectively; the corresponding values of P/H_0 and C_0 for the Auxiliary Dam are 1.4 and 3.92, respectively. The discharge coefficient C was determined by substituting values of C_0 and H_e/H_0 in Equation (5).

A broad-crested weir formula was used to develop the stage-discharge relationship for the emergency spillway. The discharge over broad-crested weir is given by the following equation (Reference 2.4-220):

$$Q = CLH^{3/2} \qquad (7)$$

Where

Q = discharge (cfs)

L = crest length of emergency spillway (ft.)

H = head of water above the crest (ft.)

C = discharge coefficient

Using a crest width of 6.1 m (20 ft.) and assumed PMF water depth of about 2.74 m (9 ft.) above crest, a discharge coefficient of 2.7 was obtained from Reference 2.4-220. This coefficient was verified to be applicable based on the water depth of 2.76 m (9.06 ft.) corresponding to the calculated PMF water level of 77.0 m (252.76 ft.). Using the discharge coefficient of 2.7 and emergency spillway crest length of 172.2 m (565 ft.), stage-discharge relationship for emergency spillway was developed. A stage-discharge curve for the Main Reservoir was developed by combining the stage-discharge relationship for the Main Dam spillway and emergency spillway. A stage-discharge relationship curve for Main Dam spillway, emergency spillway and the combined curve are shown in Figure 2.4.3-218.

During the PMF event, the rate of discharge from the Auxiliary Dam may be impacted due to tail water effects from the Main Reservoir. The magnitude of impact in the discharging capacity of an outlet structure is governed by the degree of submergence from tail water. In order to correct the discharge coefficient of the Auxiliary Dam ogee spillway, the ratios of discharge coefficients at various tail water submergence levels were obtained from Figure 9.28 in Section 9.13 of *Design of Small Dams* (Reference 2.4-230).

The crest elevation of ogee spillway for the Auxiliary Dam is at 76.8 m (252 ft.)

NGVD29. During the PMF, water level in the Main Reservoir downstream of the Auxiliary Dam will be above 76.8 m (252 ft.) for some period during which there will be a submergence effect on discharge capacity of the Auxiliary Reservoir. To consider this submergence effect in the PMF analysis, the stage-discharge relationship for the Auxiliary Dam spillway was modified with considering the conservative downstream water level in the Main Reservoir corresponding to the maximum PMF level. The stage-discharge relationship for Auxiliary Reservoir considering submergence effect is presented in Figure 2.4.3-219. The stage-discharge relationship considering the submergence effect for the Auxiliary Dam spillway was used for the PMF analysis.

2.4.3.3.4 Backwater Analysis

In order to assess the impact of the PMF event, including backwater effects at the HAR site, an unsteady state HEC-RAS model was developed (Reference 2.4-265). The geometric data necessary to develop such a model include the following:

- Stream system connectivity between the Auxiliary and Main Reservoirs and their tributaries.
- Cross-section data for various tributaries.
- Hydraulic structure data for the Main Dam.

The stream system data for the Main Reservoir were determined using Geographic Information System (GIS)-based data. The reach lengths were determined by demarcating the uppermost and lowermost points on the stream and then measuring the length along the stream between two successive demarcated points. For calculating average width, stream width was determined using GIS-based data at several locations on the stream. These values were averaged to get the representative stream width. This information was used to define the river system on a reach-by-reach basis and to establish junctions at the intersections of two or more reaches. Figure 2.4.3-220 shows an overview of the study area and various sub-basins, along with various stream segments assessed in the model. The location of the HAR plant site is indicated by the shaded square shown in the inset.

PEC performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir. Figure 2.4.3-243 presents the plan view of the bathymetry contours in 5-ft. intervals for the Main Reservoir. The survey data were compiled into a single GIS point coverage. ArcGIS 3-D analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data. This surface was further processed using ArcGIS to generate 1-ft. contour lines from an elevation of 67.1 m (220 ft.) NGVD29 to the approximate bottom of the Main Reservoir at 46.9 m (154 ft.) NGVD29. Two-ft. contours above the 67.1-m (220-ft.) NGVD29 lake level elevation were extracted from the Chatham and Wake County GIS databases. Using ArcGIS Append, the contour features greater than

67.1 m (220 ft.) NGVD29 located within the Buckhorn Creek drainage basin were combined with the existing contours below 67.1 m (220 ft.) NGVD29 to generate a comprehensive elevation file for the basin.

Since profiles were required, the contours were converted into a 3-D shapefile to facilitate the conversion to a 3-D computer-aided design (CAD) format. In addition, an overlap analysis was performed to determine where any contour line might cross another. The resulting contour files were then post-processed in the CAD environment to generate the ground surface profiles and cross-sections in each of the basin's stream reaches.

The CAD-generated data were imported into the HEC-RAS model to define the geometric data of the Main Reservoir. Each cross-section was defined by a series of points that consist of an X-value, which establishes distance from the left bank (looking downstream), and a Y-value for elevation. Once each cross-section was established, characteristics describing the downstream channel (the stream reach between the current cross-section and the next downstream cross-section) were defined, including the following:

- Manning's n values (Left Overbank [LOB] = 0.1, Main Channel = 0.045, Right Overbank [ROB] = 0.1)
- Main Channel Bank Stations (most were defined to an elevation of up to 67.1 m [220 ft.] NGVD29
- Contraction and Expansion Coefficients (contraction = 0.3 and expansion = 0.6)

Following the determination of downstream channel characteristics, data describing hydraulic structures were incorporated. In this case, the hydraulic structures to be defined were the Main Dam spillway and the emergency spillway, which are located at the southwestern-most downstream portion of the Main Reservoir.

In addition to geometric data, unsteady flow data consisting of the boundary and initial conditions are required input for the HEC-RAS model. For this requirement, PMF inflow hydrographs generated from each sub-basin of the Main Reservoir by considering the 25 percent peaking factor were used. Figure 2.4.3-221 shows various PMF inflow hydrographs used as boundary conditions within the HEC-RAS model.

In addition to the 25 percent peaking factor, the HEC-RAS model was initiated with other conservative assumptions, such as an initial stillwater elevation of 73.3 m (240.36 ft.) NGVD29 instead of 73.2 m (240 ft.) NGVD29. This initial stillwater elevation is 0.11 m (0.36 ft.) above the proposed uncontrolled ogee crest of the Main Dam spillway.

2.4.3.4 Probable Maximum Flood Flow

PMF hydrographs for the various sub-basins and the entire Buckhorn Creek drainage basin were developed using the HEC-HMS model incorporating: (1) application of the 1-hour incremental effective PMP values tabulated in Table 2.4.3-217 to the unit hydrographs of various sub-basins considering 25 percent peaking, as presented in Figure 2.4.3-214 and Table 2.4.3-223, and (2) values of initial loss and infiltration parameters, listed in Table 2.4.3-218. The HEC-HMS model is flexible and offers many options to input precipitation, to estimate runoff hydrographs, and to manipulate and route hydrographs. HEC-HMS has been used extensively throughout the U.S. to predict stream flows in both gauged and non-gauged watersheds (Reference 2.4-224).

Based on the HNP FSAR, the base flow of the Buckhorn Creek watershed is insignificant (1.1 cfs per square mile of the drainage area) in comparison to the PMF flow. Thus, no base flow was considered for this study. Further, it was assumed that both the Main Reservoir and the Auxiliary Reservoir were completely full. In order to determine the most conservative PMF elevation, the most critical rainfall scenario was selected. The following two PMP storms were considered from the PMP storms tabulated in Table 2.4.3-217:

- Case 1: Using the PMP corresponding to the entire basin (that is, the PMP storm given on Figure 2.4.3-206 for the entire basin).
- Case 2: Using two different PMP storms. Figure 2.4.3-206 was used as the PMP for the Main Dam watershed, and Figure 2.4.3-208 was used as the PMP for the Auxiliary Dam watershed.

In Case 1, the PMP storm corresponds to the Main Reservoir and Auxiliary Reservoir drainage basins. In this case, it was assumed that the same storm would be occurring over the entire drainage area. In Case 2, the PMP storm was considered as a mixture of two different PMP storms: one for the Auxiliary Reservoir drainage basin and a second for the Main Reservoir drainage basin. The most critical PMP storm scenario, which generates the higher peak and inflow volume, was selected. Table 2.4.3-225 provides model results in terms of both the peak flow and total inflow volume corresponding to the Case 1 and Case 2 PMP storm scenarios. Based on the results presented in Table 2.4.3-225, Case 2 was selected as the most critical rainfall scenario and has been used in the PMF analysis.

2.4.3.4.1 Probable Maximum Flood Flow from Drainage Basin above the Auxiliary Dam

The PMP storm corresponding to the Auxiliary Reservoir drainage area of 7.8 km^2 (3.0 $mi.^2$), as shown in Figure 2.4.3-208, was used to estimate the PMF flow for the Auxiliary Reservoir. Figure 2.4.3-222 presents the PMF inflow and outflow hydrographs for the Auxiliary Reservoir. The peak inflow and outflow for

the Auxiliary Reservoir are 197.1 m^3/s (6962 cfs) and 177.3 m^3/s (6261 cfs), respectively.

2.4.3.4.2 Probable Maximum Flood Flow from Drainage Basin above the Main Dam

The drainage area that contributes runoff to the Main Reservoir is 182.1 km^2 (70.3 $mi.^2$). This area includes the 7.8-km^2 (3.0-$mi.^2$) drainage area above the Auxiliary Dam, as described in FSAR Subsection 2.4.3.4.1. As described in FSAR Subsection 2.4.3.4, to determine the PMF inflow and outflow hydrographs, the Case 2 PMP storm was considered. Case 2 uses two different PMP storms: Figure 2.4.3-206 was used as the PMP for the Main Reservoir watershed, and Figure 2.4.3-208 was used as the PMP for the Auxiliary Reservoir watershed. Figure 2.4.3-223 presents the inflow and outflow hydrographs for the Main Reservoir. The peak inflow and outflow for the Main Reservoir are 3545.0 m^3/s (125,190.5 cfs) and 1410.4 m^3/s (49,807.1 cfs), respectively.

2.4.3.5 Water Level Determinations

Using the level pool routing technique, along with the stage-storage curve and storage-out flow curve of dam outlet works for the Auxiliary and Main Reservoirs within the HEC-HMS model, PMF stillwater elevations were determined for both the Auxiliary and Main Reservoirs. Figures 2.4.3-224 and 2.4.3-225 present the stillwater elevations for the Auxiliary Reservoir and the Main Reservoir, respectively. The peak stillwater elevations in the Auxiliary Reservoir and Main Reservoir were found to be 78.2 m (256.50 ft.) NGVD29 and 77.0 m (252.48 ft.) NGVD29, respectively.

As discussed in FSAR Subsection 2.4.3.3.4, the HEC-RAS model was initiated with conservative initial conditions. The results of the HEC-RAS model run for the PMF event are shown on Figures 2.4.3-226, 2.4.3-227, 2.4.3-228, and 2.4.3-229. Figure 2.4.3-226 presents time series of stage (ft. NGVD29) and flow (cfs) values for Thomas Creek, a tributary of the Main Reservoir adjacent to the HAR plant site. Based on this plot, the stage peaked on day 9 at an elevation of 77.0 m (252.76 ft.) NGVD29.

Figures 2.4.3-227 and 2.4.3-228 show cross-sections depicting the maximum stage at two locations immediately upstream and downstream of the HAR plant site. Again, the maximum stage at these locations during the PMF event is 77.0 m (252.76 ft.) NGVD29. Similarly, Figure 2.4.3-229 presents the maximum stage for all cross-section locations from the downstream end of the Main Reservoir (Main Dam) to the upstream end (Thomas Creek, upstream of the HAR plant site). The maximum stage during the PMF event for all locations along the Main Reservoir and Thomas Creek, including backwater effects, is 77.0 m (252.76 ft.) NGVD29. These values are provided in tabular format in Table 2.4.3-226.

The final stillwater PMF elevations obtained using HEC-RAS and HEC-HMS models are summarized in Table 2.4.3-227. The maximum stillwater elevations in the Auxiliary Reservoir and Main Reservoir are 78.2 m (256.50 ft.) NGVD29 and 77.0 m (252.76 ft.) NGVD29, respectively.

Based on the flood routing analysis, the PMF discharge over the emergency spillway is approximately 1217.6 m^3/s (43,000 cfs). An analysis to determine the water surface profile in the emergency spillway channel was performed using the PMF discharge of 1217.6 m^3/s (43,000 cfs). The water level immediately downstream of the emergency spillway is 73.6 m (241.4 ft.) NGVD29. This elevation is lower than the spillway crest elevation of 74.3 m (243.7 ft.) NGVD29 and therefore will not affect the flow over the emergency spillway due to submergence. Water levels and velocities in the emergency spillway channel immediately downstream of the emergency spillway, at upstream and downstream of the drop structure and at upstream and downstream of the railroad crossing, are presented in Table 2.4.3-239.

2.4.3.6 Coincident Wind-Wave Activity

As discussed in FSAR Subsection 2.4.2.2, safety-related structures and facilities for the HAR site are protected against floods and flood waves caused by probable maximum events, such as the PMF and the PMH. Coincident wind-wave activity was evaluated at HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. In the context of the HAR site, the Auxiliary Dam and Main Dam are not safety-related structures. However, in the context of HNP, the Auxiliary Dam is a safety-related structure, whereas the Main Dam is not a safety-related structure. In this analysis, however, both the Auxiliary and Main Dams have been considered for evaluating the coincident wind-wave activity. For the wind-wave activity analyses, the USACE's *Coastal Engineering Manual, Engineer Manual 1110-2-1100* (Part II) was strictly followed (Reference 2.4-263).

In order to determine wind setup and wave runup for a given site, the following data were required:

- Water body bathymetry data
- Critical fetch distances
- Over-wind speed averaged for an appropriate duration
- Site characteristics, such as protection type and material and slope

2.4.3.6.1 Bathymetry Data

PEC performed detailed bathymetric surveys to establish the current geometry of the Main Reservoir with thousands of depth-to-bottom measurements collected during the study. These data were compiled into a single GIS point coverage.

ArcGIS 3-D analyst Kriging sampling interpolation was used to generate a 3-D surface from the mass point data.

The locations of interest for determining the wind-wave activity for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are shown on Figures 2.4.3-230, 2.4.3-231, 2.4.3-232, and 2.4.3-233. For wind and wave calculation purposes, water depths at various locations were determined using: (1) the bottom elevation of the lake near the location at which wind-wave activity had been determined, and (2) the stillwater PMF elevation for a given scenario and option. Table 2.4.3-228 presents the lake bottom elevations that were used in the wind-wave activity analysis.

2.4.3.6.2 Determination of Fetch for the Main Reservoir and the Auxiliary Reservoir

Fetch is the length of water surface exposed to wind during the generation of waves. Fetch is an important characteristic of open water because longer fetch can result in larger wind-generated waves. For this analysis, straight line fetch distances were used in the wave runup calculations, as detailed in the EM 1110-2-1100 (Part II) (Reference 2.4-263). Selected straight line fetches associated with HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are shown on Figures 2.4.3-230, 2.4.3-231, 2.4.3-232, and 2.4.3-233. Tables 2.4.3-229, 2.4.3-230, 2.4.3-231, and 2.4.3-232 provide the over water fetch distances. The critical fetch distances for the HAR 2, HAR 3, HNP, Auxiliary Dam, and Main Dam locations are 1.50, 1.37, 6.97, 6.90, and 6.90 km (0.93, 0.85, 4.33, 4.29, and 4.29 mi.), respectively.

2.4.3.6.3 Over Water Wind Speed

According to ANSI/ANS 2.8-1992 (Reference 2.4-222), a 2-year wind speed should be used while conducting the coincident wind-wave activity analysis. Therefore, the 2-year wind speed was obtained from ANSI/ANS 2.8-1992 (Reference 2.4-222) and used for conducting the coincident wind-wave activity analysis at the HAR site. The 2-year wind speed at the HAR site is 50 mph. Before using this wind speed in the calculation of wave runup, several adjustments were applied following the procedure outlined in the EM 1110-2-1100 (Part II) (Reference 2.4-263). The step-by-step procedure is as follows:

1. Standard measurements should be collected at 10 meters above ground surface. Since the wind speed obtained from the ANSI/ANS 2.8.1992 (Reference 2.4-222) was measured at 10 m (30 ft.) above ground, no adjustment was applied.

2. The averaging duration associated with the 2-year wind speed obtained from ANSI/ANS 2.8-1992 is assumed to be 1 hour (Reference 2.4-222). Using a 2-year wind speed of 50 mph, its averaging duration, and Figure II-2-1 of EM 1110-2-1100 (Part II)

(Reference 2.4-263) (Figure 2.4.3-234), the 1-hour wind speed was calculated as 50.0 mph.

3. Overwater wind speeds at various locations were then determined by applying a correction for transition from land to water. This correction factor was determined using Figure II-2-7 of EM 1110-2-1100 (Part II) (Reference 2.4-263) (Figure 2.4.3-235). (Note: The HAR site is approximately 225 km (140 mi.) from the coast line.)

   According to this figure (Figure 2.4.3-235), the correction factor R_L is given as follows (Reference 2.4-263):

   $$U_W = R_L * U_L \tag{8}$$

   where

   U_W is the over-water wind speed,
   U_L is the overland wind speed, and
   R_L is a correction factor which is equal to 0.9 for U_L > 41.5 mph.

   In this case, the overland wind speed, U_L, is 50.0 mph (20.1 m/s). As such, the correction factor, R_L, is equal to 0.9. However, in an effort to be conservative, a correction factor, R_L, equal to 1.0 was used for this analysis.

4. Finally, the wind speed was corrected according to the appropriate averaging duration. When a sustained wind with essentially constant direction over a fetch for sufficient time achieves steady-state, fetch-limited values, simplified wave predictions can provide accurate estimates of wave conditions. The time required to accomplish fetch-limited wave development for short fetches was calculated as follows (Reference 2.4-263):

   $$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \tag{9}$$

   where

   $t_{x,u}$ is the time required for waves crossing a fetch of length x under a wind of velocity u to become fetch-limited.

   The resulting averaging time interval, $t_{x,u}$ was then used in conjunction with Figure II-2-1 of EM 1110-2-1100 (Part II) (Figure 2.4.3-234) in order to determine the appropriate wind speed for various locations. Table 2.4.3-233 presents calculated corrections of wind averaging intervals for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam.

2.4.3.6.4 Site Characteristics Such as Type and Material of Protection and Slope

The HAR is surrounded by the Thomas Creek Branch of the Main Reservoir on the east side and by the Auxiliary Reservoir on the west side. Figure 2.4.1-205 shows the planned site drainage plan and indicates that the HAR will have permeable natural land area between the developed site and the water bodies. Using site-specific topographic and bathymetry data, the land slope adjacent to the Thomas Creek Branch of the Main Reservoir and Auxiliary Reservoir were determined; these slopes are 0.09 and 0.13 for the east and west sides of the HAR, respectively.

The upstream faces of the Main Dam and Auxiliary Dam are protected by riprap with slopes of 1(V):2(H) and 1(V):2.5(H), respectively. On the plant island, the embankments located on the Auxiliary Reservoir (Line 1 as shown on Figure 2.4.3-233, which is directed toward the plant island from the Auxiliary Reservoir) and the Main Reservoir (Lines 2 and 3 as shown on Figure 2.4.3-233, which are directed towards the plant island from the Main Reservoir) are protected by sacrificial spoil fill. The embankment intersected by Line 1 has a slope of 1(V):10(H) whereas the embankment intersected by Lines 2 and 3 have a slope of 1(V):10(H).

2.4.3.6.5 Wave Runup

Having determined the estimate of winds for wave prediction, wave runup for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam were calculated according to the step-by-step procedure given in the EM 1110-2-1100 (Part VI) (Reference 2.4-263). The step-by-step procedure is as follows:

1. Using the previously determined fetch lengths and wind speeds, estimates of significant wave heights were obtained using the deepwater nomogram for the fetch limited wave heights given in the EM 1110-2-1100 (Part II) (Figure 2.4.3-236).

2. Similarly, estimates of peak wave periods were obtained using the deepwater nomogram for the fetch limited wave periods given in the EM 1110-2-1100 (Part II) (Figure 2.4.3-237).

3. The peak wave periods for various fetch lines calculated above were compared with the shallow-water limit. According to the CEM (Reference 2.4-263), the shallow-water limit is given by the following equation:

$$T_P \approx 9.78\left(\frac{d}{g}\right)^{\frac{1}{2}} \tag{10}$$

where

T_P = the limiting wave period in sec,

d = the water depth in meter, and

g = the gravitational acceleration in meter/sec^2 .

If the predicted peak wave period for a given fetch line is greater than the limiting value, then the predicted wave period was reduced to the limiting wave period. Conversely, if the predicted wave period was less than the limiting value, the predicted deepwater wave period was retained and used for further calculations.

4. Wave runup was calculated using the wave runup equation on permeable slopes given in Chapter 5 of *EM 1110-2-1100* (Part VI) (Reference 2.4-263). According to the CEM, the runup equation for various levels of percentage exceedances is given as:

$$R_{ui\%}/H_S = A\xi_{om} \quad for \;\; 1.0 < \xi_{om} \le 1.5 \tag{11a}$$

$$R_{ui\%}/H_S = B\xi_{om}^{C} \quad for \;\; 1.5 < \xi_{om} \le (D/B)^{1/C} \tag{11b}$$

$$R_{ui\%}/H_S = D \; for \;\; (D/B)^{1/C} < \xi_{om} \le 7.5 \tag{11c}$$

where, ξ_{om} is the surf-similarity parameter for irregular waves defined as:

$$\xi_{om} = \frac{\tan\alpha}{\sqrt{s_{om}}} \tag{12}$$

In which α is an angle defined by arctangent of the slope of structure (dam)/embankment or stream or reservoir bank, and S_{om} is the fictitious wave steepness. S_{om} is the ratio between the statistical wave height at the structure and representative deepwater wavelengths and is defined as:

$$s_{om} = \frac{H_s}{L_{om}} = \frac{2\pi}{g}\frac{H_s}{T_m^2} \tag{13}$$

where

H_s = significant wave height of incident waves at the toe of the structure,

L_{om} = deepwater wavelength,

T_m = mean wave period, and

T_P = wave period corresponding to the peak of the wave spectrum.

The coefficients A through D in Equations 11a, 11b, and 11c for runup of irregular head-on waves on impermeable and permeable rock armored slopes are given in Table 2.4.3-234 (Reference 2.4-263). Using Steps 1 through 4 for various fetch lines, runup was calculated. Table 2.4.3-235 presents the runup results.

2.4.3.6.6 Wind Setup

Sustained wind over a water body exerts a horizontal stress on the water surface in the wind direction. In an enclosed water body, this wind effect results in a surplus of water at the leeward end and a decrease in water level at the windward end. This effect is called wind setup. According to EM 1110-2-1420 (Reference 2.4-264), the wind setup in lakes and reservoirs can be estimated using the Zeider Zee equation given as:

$$S = \frac{U^2 X}{1400d} \tag{14}$$

where

S = the setup (ft.) above the stillwater level,

U = the wind speed (mph),

X = the fetch length (mi.), and

d = the water depth corresponding to the PMF level.

Table 2.4.3-236 presents the setup calculation results using Equation (14) for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam locations.

2.4.3.6.7 Overall PMF Elevation

The values of stillwater elevation, wave runup, and wind setup for various fetch lines were added together to determine the PMF elevation coincident with

wind-wave activity at HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. Table 2.4.3-237 presents the overall PMF elevations for these locations.

2.4.3.6.8 Maximum PMF Elevation due to Coincident Wind-Wave Activity

The maximum PMF elevations for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are summarized in Table 2.4.3-238. None of the PMF elevations exceeded the target elevation of 79.2 m (260 ft.) NGVD29. Therefore, no potential hazard exists to the plant's safety-related facilities as a result of the effect of the PMF.

2.4.3.7 Erosion Protection for Emergency Spillway and Downstream Channel

As shown in Figure 2.4.3-242, the emergency spillway and the upstream approach channel will be protected with concrete lining. In addition, the concrete apron downstream of the spillway will be extended 6.1 m (20 ft.) below the channel bottom to an elevation of approximately 64.0 m (210 ft.) NGVD29 at a slope of 3 horizontal to 1 vertical. This apron will provide additional protection for the downstream slope of the emergency spillway, even if the downstream riprap is eroded during the very high discharges, including the PMF.

The drop structure, the stilling basin, and the portion of the spillway channel at the railroad crossing will be provided with concrete lining, which will provide protection against erosion and scour due to the high flow velocities. The rest of the spillway channel will be protected with riprap placed over a bedding layer and geotextile.

The bridge structure for the railroad crossing will be supported by concrete piers in the spillway channel and abutments at each end. The bridge structure foundations will be protected from scour by providing a bottom concrete liner in the vicinity of the bridge.

2.4.4 POTENTIAL DAM FAILURES

HAR COL 2.4-2
HAR COL 2.5-15

There are no existing dams upstream or downstream of Harris Reservoir that can affect the HAR site safety-related facilities or the availability of the cooling water supply. The National Hydrography Dataset was reviewed to identify impoundments in the Buckhorn Creek Drainage Basin (Reference 2.4-266). All impoundments other than the Auxiliary and Main Reservoirs were less than or equal to 10 ac. in size and were not considered to be large enough to affect HAR safety-related facilities or the availability of the cooling water supply.

Furthermore, HAR 2 and HAR 3 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 under conditions of Main Dam failure. 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

HAR COL 2.4-2

A PMH could cause a water level change in the Main Reservoir and the Auxiliary Reservoir. The resulting high water levels, if not considered in the project design, could affect the safety of the Main Dam, Auxiliary Dam, or safety-related structures located at the plant site. The following subsections discuss the design considerations afforded these facilities related to flooding. Preceding the PMH, the assumed stillwater level elevations in the Main Reservoir and Auxiliary Reservoir are 73.2 m (240 ft.) NGVD29 and 76.8 m (252 ft.) NGVD29, respectively.

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The meteorological characteristics used to calculate the PMH were obtained from NOAA Technical Report NWS 23 (Reference 2.4-234). According to this report, 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 specific coastal location. From values of the parameters, a wind field is specified, which is termed the “PMH wind field.” The following are the over-water PMH wind field parameters taken from NOAA Technical Report NWS 23 corresponding to the Milepost 2200 and Latitude of 35.6 degrees (Figure 1.1 of NOAA, Reference 2.4-234):

- Coriolis parameter (f) = 14.584x10^{-5} Sin(35.6) = 0.31 hr^{-1}
- Peripheral pressure (P_w) = 30.12-in.
- Central pressure (P_0) = 26.4-in.
- Radius of maximum wind (R) = 9 nautical miles for small storms, and 25 nautical miles for large storms
- Forward speed (T) = 10 knots (KT) for small storms, and 34 knots for large storms
- Density coefficient, K = 68.7 KT-in

Using the above parameters, the maximum gradient wind speed (V_{gx}) was calculated for a stationary hurricane using the following relationship (Reference 2.4-234):

$$V_{gx} = K\left(P_w - P_0\right)^{\frac{1}{2}} - \frac{fR}{2} \qquad (15)$$

The obtained value of V_{gx} is 128.7 KT (148.2 mph). The value of V_{gx} is adjusted to the maximum 10-meter, 10-minute value (V_{xs}) for a stationary hurricane using the following relationship (Reference 2.4-234):

$$V_{xs} = 0.95 V_{gx} \tag{16}$$

The obtained maximum 10-meter, 10-minute wind speed for a stationary hurricane, V_{xs} is 122.2 KT (140.8 mph). In order to determine wind speed for a moving hurricane, the stationary hurricane wind speed needs to be adjusted for asymmetry due to storm forward speed (T). The asymmetry factor (AF) is given as (Reference 2.4-234):

$$AF = 1.5 T^{0.63} T_0^{0.37} Cos(\beta) \tag{17}$$

where

AF is the asymmetry factor in KT,

T is the forward wind speed of the storm in KT,

T_0 = 1 when AF and T are in KT, and

β is the angle between track direction and the surface wind direction.

To be conservative, β was assumed to be zero giving $Cos(\beta)$ a value of 1. Substituting values of T_0 and $Cos(\beta)$, the maximum value of asymmetry factor is:

$$AF = 1.5 T^{0.63} \tag{18}$$

The maximum value of T is 34 KT (39.2 mph). Substituting T = 34 KT in Equation (18), the maximum value of asymmetry factor (AF) is 13.8 KT (15.9 mph). Adding the stationary hurricane wind speed (V_{xs}) and asymmetry factor (AF) together, the wind speed for a moving hurricane is 136.1 KT (156.8 mph).

When the center of a hurricane crosses the coast, over-water wind speeds are reduced because of filling by a factor that decreases with travel time after landfall. Kaplan and Demaria (Reference 2.4-267) developed a mathematical model for predicting decay of maximum sustained surface winds after storm landfall using a combination of physical and empirical considerations. In the simplest version of this model, the maximum winds inland are a function of the maximum winds at landfall and of the travel time after landfall. With the assumption of a track perpendicular to the coastline, it can be used to estimate the maximum inland penetration of winds of a given speed using the storm's landfall intensity and speed of motion. This decay model is given as (Reference 2.4-267):

$$V(t) = V_b + \left(R_f V_0 - V_b\right) \exp(-\alpha t) \tag{19}$$

where

$V(t)$ is the inland storm wind speed on traveling overland for time (t) hours after landfall,

V_b is the background wind speed,

R_f is the initial decay factor just after the landfall,

V_0 is the storm wind speed just before the landfall,

and

α is a coefficient.

The values of R_f, V_b, and α are 1.0, 26.7 KT, and 0.095 hr^{-1}, respectively (Reference 2.4-267). Substituting these values in Equation (19), the decay model can be written as:

$$V(t) = 26.7 + (V_0 - 26.7)e^{-0.095t} \tag{20}$$

In the above Equation, V(t) and V_0 are in knots and t is in hours. The HAR site is located about 225.3 km (140 mi.) inland from the coastline. With a forward speed of 34 knots (39.2 mph), the overland travel time is 3.6 hours.

Substituting the value of the overland travel time t = 3.6 hours and V_0 =136.1 KT into Equation (20), the maximum 10-meter, 10-minute overland wind speed at the HAR site is 94.9 KT (109.3 mph).

2.4.5.2 Surge and Seiche Water Levels

For the HAR site, the only dynamic mechanism considered to be credible for the production of high water levels is the probable maximum wind (PMW) discussed in Subsection 2.4.5.1. Subsection 2.4.5.3 discusses the effects of the PMW on the plant reservoirs and the resulting activity.

2.4.5.3 Wave Action

In order to determine impact of wind-wave action due to PMW, straight line fetch distances were used in wave runup and setup calculations. Conservatively, straight line fetch distances shown in Figures 2.4.3-230, 2.4.3-231, 2.4.3-232, and 2.4.3-233 and tabulated in Tables 2.4.3-229, 2.4.3-230, 2.4.3-231, and 2.4.3-232 were used for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam locations.

2.4.5.3.1 Wind Speed Corrections

As presented in Subsection 2.4.5.1, the maximum 10-meter, 10-minute overland wind speed at the HAR site is 94.9 KT (109.3 mph). Based on Figure 2.4.3-234, the ratio of wind speed of any duration (V_t) to the 1-hour wind speed (V_{3600}) is given as:

$$\frac{V_t}{V_{3600}} = \begin{cases} 1.277 + 0.296\tanh\left[0.9\log_{10}\left(\frac{45}{t}\right)\right] & t \leq 3600 \\ -0.15\log_{10} t + 1.5334 & 3600 < t < 36{,}000 \end{cases} \tag{21}$$

Substituting the maximum 10-meter, 10-minute overland wind speed of 94.9 KT (109.3 mph) into Equation (21), the maximum 10-meter, 1-hour overland wind speed is 90.4 KT (104.1 mph). Before using this wind speed in the calculation of wave runup and wind setup, several adjustments such as corrections for transition from land to water and averaging durations based on various fetch distances needs to be applied using the procedure outlined in the U.S. Army Corps of Engineers' Coastal Engineering Manual, Engineer Manual 1110-2-1100 (Part II) (Reference 2.4-263). Subsection 2.4.3.6.3 provides a step-by-step procedure for calculating over-water wind speed.

Over-water wind speed is determined by applying a correction factor for transition from land to water. Based on Figure 2.4.3-235 and an overland wind speed of 90.4 KT (104.1 mph), the correction factor is 0.9. Therefore, the over-water wind speed is 90.4 KT x 0.9 = 81.3 KT (93.7 mph). Following the procedure presented in Subsection 2.4.3.6.3, the wind speed was corrected for averaging duration. Substituting over-water wind speed of 93.7 mph (150.8 km/hr) and over-water fetch distances for various locations in Equation (9), the averaging time durations ($t_{x,u}$) were determined for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. Fetch distances corresponding to PMF conditions were used to determine PMH water levels, thus increasing straight line fetch lengths and creating more conservative conditions than those corresponding to normal water levels. The calculated averaging durations for various locations have been listed in Table 2.4.5-201. Using these calculated averaging durations in conjunction with Figure II-2-1 (Figure 2.4.3-234) of *EM 1110-2-1100 (Part II)* (Reference 2.4-263), the appropriate wind speeds for various fetch lines were determined. Table 2.4.5-201 presents both the calculated correction factors for various winds averaging durations and corrected wind speed for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam.

2.4.5.3.2 Wave Runup, Wave Height, and Wave Period

After determining estimates of PMH wind for wave prediction, the wave runup for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam were calculated using the step-by-step procedure given in *EM 1110-2-1100* (Reference 2.4-263) as described in Steps 1 through 4 of Subsection 2.4.3.6.5. Table 2.4.5-202 presents the results of wave runup along with significant wave heights and wave periods.

2.4.5.3.3 Wave Setup

Using the procedure described in Subsection 2.4.3.6.6, wave setups were determined for various fetch lines directed toward HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. Table 2.4.5-203 presents results of wave setup.

2.4.5.3.4 Overall PMH Elevation

The values of stillwater elevation, wave runup, and wind setup for various fetch lines were added together to determine the PMH water elevation coincident with wind-wave activity at HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam. Table 2.4.5-204 presents the overall PMH elevations for these locations.

2.4.5.3.5 Maximum PMH Elevation due to Coincident Wind-Wave Activity

The maximum PMH elevations for HAR 2, HAR 3, HNP, the Auxiliary Dam, and the Main Dam are summarized in Table 2.4.5-205. As indicated by Table 2.4.5-205, none of the PMH elevations exceeded the target elevation of 79.2 m (260 ft. NGVD29). Therefore, no potential hazard exists to the plant's safety-related facilities as a result of the effect of the PMH.

2.4.5.4 Resonance

In order to discuss a possibility of oscillations of waves at natural periodicity such as lake reflection and harbor resonance phenomena and any resulting effects at the site, two sinusoidal waves of the same amplitude and wavelength traveling in opposite directions are considered and their interference can be studied through Equations (22) and (23):

$$y(x,t) = y_0 \sin(kx - \omega t) \tag{22}$$

$$y(x,t) = y_0 \sin(kx + \omega t) \tag{23}$$

Where y_0 is the initial magnitude of a wave, k is the wave number represented as $2\pi/\lambda$, in which λ is a wavelength of oscillations generated by an external perturbation. The symbol ω represents the angular frequency and is represented as $2\pi/T$, in which T is the time period, and x and t represent location and wave travel time. According to the superposition principle, the resultant wave is simply the sum of the two waves, i.e.:

$$y_{Resonance}(x,t) = y_0 \sin(kx - \omega t) + y_0 \sin(kx + \omega t) \tag{24}$$

Using trigonometry identities, the sum of two sines is given by:

$$y_{Resonance}(x,t) = 2y_0 \sin(kx)\cos(\omega t) \tag{25}$$

The amplitude of the resonance wave is $2y_0 \sin(kx)$, which varies with position x. Alternatively, Equation (25) can also be written as:

$$y_{\mathrm{Re}\,sonance}(x,t) = 2y_0 \sin\left(\pi \frac{2x}{\lambda}\right)\cos\left(\pi \frac{2t}{T}\right) \tag{26}$$

Well defined points exist where the magnitude of the resonance wave is zero. Such as at x = 0, $\lambda/2$, λ, $3\lambda/2$, 2λ, etc; these points are called the nodes. Similarly, points of maximum magnitude oscillations, called the antinodes, are the locations of maximum amplitude oscillations such as at x = $\lambda/4$, $3\lambda/4$, $5\lambda/4$, etc. At these locations the two waves undergo constructive interference and result in resonance with magnified magnitude. This indicates that the length of a water body over which waves are generated should be a multiple of $\lambda/2$. This multiple is also known as the mode or harmonic of an oscillating wave. The oscillation resonance mode *n* can be determined as (Reference 2.4-272):

$$n = \frac{2L}{\lambda} \tag{27}$$

The corresponding resonance period T_n is given as (Reference 2.4-273 and Reference 2.4-274):

$$T_n = \frac{2L}{n\sqrt{gh}} \tag{28}$$

where *L* is the length, *h* the average depth of the body of water, and *g* the acceleration of gravity. Using the physical parameters of the Main and Auxiliary Reservoirs provided in Tables 2.4.3-235 and 2.4.3-236, the numbers of modes (*n*) at which resonance may occur were determined as given in Table 2.4.5-206. As presented in Table 2.4.5-206, all modes are over 100.

The amplitude and persistence of a wave depends not only on the magnitude of energy source but also on the energy losses within a water body. Such losses include dissipative effects resulting from friction on the sides or bottom of the basin. If a wave is generated by an external impulsive event such as a sudden change in atmospheric pressure gradient, the amplitude is seen to decay by nearly constant fraction with each successive period or mode. In general, the rate of decay is greater for basins that are shallow or have narrow constrictions and complex topography (Reference 2.4-274).

Using geometric progression and assuming a constant rate loss of *r*, the magnitude of resonance wave A_n after n modes is given as:

$$A_n = 2y_0(1-r)^{n-1} \tag{29}$$

Equation (29) indicates that only the first few modes are important as *n* increases *A* tends to decrease. At a large value of *n*, A_n becomes insignificant. Based on literature, the decay rate *r* in Lake Geneva and Lake Erie has been estimated to be about 3 percent and 32 percent, respectively, with each successive wave period (Reference 2.4-274). Both the Main and Auxiliary Reservoirs are very shallow and have narrow constrictions with complex topography. Therefore, it is expected that the decay rate *r* will be large. In the absence of observed data, a conservative decay rate of 10 percent was assumed and the magnitude of resonance wave after *n* modes A_n were determined as shown in Table 2.4.5-207.

As presented in Table 2.4.5-207, the magnitude of resonance wave for all fetch lengths is zero. Therefore, wave amplification due to resonance will not occur on the Auxiliary or Main Reservoirs at the HAR site, because the wind fetch is approximately 100 times longer than the significant wave length. The resonance due to such a high mode, if it does occur, would not have an appreciable effect due to the fact that only the first few modes of resonance are of concern for wave amplification.

2.4.5.5 Protective Structures

All safety-related structures at the plant site are protected from high water levels up to elevation 79.6 m (261 ft.) NGVD29, which is higher than anticipated flood levels due to wave runup in the reservoirs or direct rainfall at the plant site.

The Auxiliary Dam is a safety-related structure for HNP and the Main Dam is built as Seismic Category I. However, the Main Dam and Auxiliary Dam do not serve any safety-related purpose for HAR 2 and HAR 3. Subsection 2.4.1.1 provides additional information pertaining to the dams and spillways.

The upstream face of the Main Dam and both upstream and downstream faces of the Auxiliary Dam are protected by riprap designed for the worst postulated wave action. Subsection 2.5.6 of the HNP FSAR provides an additional description of the riprap design. The downstream face of the Main Dam is protected by a layer of oversized rock. As discussed in Subsection 2.4.2.2, the backwater effects of the Cape Fear River on the downstream face of the Main Dam are not expected to be significant. However, protection of the downstream face, as described in Subsection 2.5.6.4 of the HNP FSAR, serves as an additional safety precaution.

2.4.6 PROBABLE MAXIMUM TSUNAMI HAZARDS

HAR COL 2.4-2

Coastal areas bordering the Pacific Ocean and U.S. territories in the Caribbean, notably Puerto Rico and the U.S. Virgin Islands, are most susceptible to tsunamis (Reference 2.4-235). The HAR site is located approximately 225.3 km (140 mi.) inland from the Atlantic coast, where tsunami hazards are relatively low. Therefore, the HAR site is not subjected to the effects of tsunami flooding.

The potential for a slope failure into the Main or Auxiliary Reservoirs causing a tsunami-like wave is negligible based on the extensive site-specific investigations associated with topography, geology, seismicity, and groundwater (Subsections 2.5.0.1.2, 2.5.1, 2.5.2, 2.5.4.1, 2.5.3, 2.5.0.3, 2.5.3.6, and 2.4.12.1). In addition, the current land use is not conducive to landslide activity (Subsection 2.4.3.2) and no observed or recorded land slippage of any kind has occurred along the shore of either the Main or Auxiliary Reservoirs since the reservoirs were filled in late 1980.

2.4.7 ICE EFFECTS

HAR SUP 2.4-2

A review of historical temperature records from the NWS Cooperative Observer Station No. 317069 in Raleigh, North Carolina, for the period 1971 to 2000 indicates monthly average minimum temperatures for the months of December, January, and February as being 0.33 degrees Celsius (°C), -1.33°C, and -0.06°C (32.6 degrees Fahrenheit [°F], 29.6°F, and 31.9°F), respectively (Reference 2.4-221). The monthly mean temperatures for the same months are 6.11°C, 4.3°C, and 6.1°C (43.0°F, 39.7°F, and 43.0°F), respectively. Ice formation in this locality on large bodies of water in Central North Carolina is expected to be limited to minor freezing along shorelines. 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 the cooling water canals and reservoirs are needed.

The 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

There is no historical evidence of channel diversion above the Main Dam within Buckhorn Creek, Tom Jack Creek, Thomas Creek, Little White Oak Creek, White Oak Creek, or Cary Creek. Examination of USGS 1:24,000-scale topographic maps associated with the Buckhorn Creek drainage basin did not reveal

evidence of natural channel diversions (e.g., oxbow lakes or broad, well developed floodplains). Creeks and streams within the watershed generally occur in well-defined valleys and, therefore, limit the possibility of water diversion into adjacent drainage basins.

Topographic characteristics and geological features of the drainage basin indicate there is no possibility for the occurrence of a landslide blocking or limiting streamflow into Harris Lake.

Because ice effects are expected to be limited to minor freezing, they are not expected to create flow diversion during winter months.

2.4.10 FLOODING PROTECTION REQUIREMENTS

HAR COL 2.4-2 The flooding effects of a PMF on Harris Lake and a local PMP at the plant site are the design bases for flood protection. Subsection 2.4.2.2, Flood Design Considerations, discusses considerations for selecting the PMF as the design flood. Subsection 2.4.3, Probable Maximum Flood on Streams and Rivers, discusses the effects of the PMF and coincident wind-wave activity on the reservoirs.

The DCD, Section 2.0, Table 2-1 specifies the key site parameters for the design of safety-related structures, systems, and components for the AP1000 design. DCD, Section 3.4 provides the flood design and protection requirements and DCD, Section 3.8 provides the design of seismic Category I structures, including the design loads and load combinations.

2.4.11 LOW WATER CONSIDERATIONS

HAR SUP 2.4-3 Conditions such as limited flow rates and low cooling water elevations resulting from severe droughts will not affect the ability of the safety-related facilities associated with the AP1000 design, particularly the ultimate heat source, to perform adequately. HAR 2 and HAR 3 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 Main Reservoir is only used to provide HAR 2 and HAR 3 with cooling water for normal operations to produce electricity.

To demonstrate that HNP, HAR 2, and HAR 3 can continue to operate during low flow conditions, two evaluations were performed. The first hypothetical situation evaluates the ability of the three units to operate under low flow conditions. The second hypothetical situation evaluates how long the three plants could operate without withdrawing water from the Cape Fear River.

2.4.11.1 Hypothetical Operation of HNP, HAR 2, and HAR 3 under Low Flow Conditions

This hypothetical evaluation determines if a normal pool elevation of 73.2 m (240 ft.) NGVD29 within the Main Reservoir could sustain the HNP, HAR 2, and HAR 3 during severe drought conditions. For the evaluation, historical inflows, meteorological data, and projected consumptive use were used to compute the water balance and the Main Reservoir water level on a monthly basis for a period from October 1939 to September 2004. Inputs into Harris Lake included inflow from the Buckhorn Creek drainage basin above the Main Dam, precipitation onto the Main Reservoir and the Auxiliary Reservoir, and inflow of makeup water from the Harris Lake makeup water system to the Main Reservoir. Multiple makeup water flow rates from the Cape Fear River were considered in the evaluation to determine a reasonable makeup flow. Outputs consisted of spillage above a proposed normal pool elevation of 73.2 m (240 ft.) NGVD29, consumptive use due to plant operation (forced evaporation), makeup water pumping to the Auxiliary Reservoir from the Main Reservoir, seepage, and natural evaporation.

Two outage periods of 15 days each were considered in the evaluation. The first outage period included the HNP and one proposed AP1000 unit; the second period was staggered by 6 months for the second proposed AP1000 unit. These outages were repeated once every 18 months. During these periods, the plant consumptive use was considered zero for the respective units in outage.

Water balance computations for a proposed Main Reservoir normal pool elevation of 73.2 m (240 ft.) NGVD29 were completed to determine the minimum makeup water flow from the Cape Fear River required for a three-unit operation so that the minimum water level would be equal to 67.1 m (220 ft.) NGVD29. This effort required trial and error computations using different assumed makeup water flows, with a resulting determination that a makeup water flow rate of 1.1 m^3/s (40.3 cfs) supports the minimum lake water level criteria.

Results from the evaluation include the following:

- Four severe drought periods occurred during the period of analysis from October 1939 to September 2004 as observed by changes in reservoir water levels: 1940 to 1943, 1950 to 1957, 1966 to 1972, and 1985 to 1997. The worst drought period occurred between 1985 and 1997.

- For this computation only, the net total consumptive water use for the HNP is 0.8 m^3/s (27.2 cfs) as specified in Subsection 2.4.11.6 of the HNP FSAR and assumed to be 1.9 m^3/s (66.8 cfs) for HAR 2 and HAR 3.

Therefore, the total assumed consumptive use for the HNP, HAR 2, and HAR 3 is 2.7 m^3/s (94 cfs).

- Streamflow in the Cape Fear River near the confluence of Buckhorn Creek is regulated upstream by the B. Everett Jordan Dam. A review of streamflow data from the Cape Fear River indicates regulated releases have been occurring since 1982. Therefore, it was determined that the 7-day, 10-year (7Q10) low flow in the river near the confluence of the Buckhorn Creek for the period after 1982 would be relevant for use in this analysis. The 7Q10 low flow for the period 1982 to 2004 near the confluence of the Buckhorn Creek and Cape Fear River was estimated as 10.8 m^3/s (382 cfs). The minimum available makeup water flow from the Cape Fear River during low flow periods is assumed to be 20 percent of the 7Q10 low flow or about 2.2 m^3/s (76.4 cfs).

- Assuming plant outages, a proposed normal pool elevation within the Main Reservoir of 73.2 m (240 ft.) NGVD29, and a continuous Cape Fear River makeup water flow rate of 1.1 m^3/s (40.3 cfs), the minimum Main Reservoir water elevation during the period of October 1939 to September 2004 was 67.1 m (220 ft.) NGVD29 with the HNP and two proposed AP1000 units operating. Using these parameters, the computed average monthly downstream releases from the Main Dam would be 0.3 m^3/s (10.3 cfs) for the period from 1939 to 2004.

Based on this evaluation, a proposed normal pool elevation of 73.2 m (240 ft.) NGVD29 within the Main Reservoir and a continuous makeup water flow rate from the Cape Fear River of 1.1 m^3/s (40.3 cfs) would be acceptable for the operation of HNP, HAR 2, and HAR 3 during historical low water periods.

2.4.11.2 Hypothetical Operation of HNP, HAR 2, and HAR 3 without Makeup Water from the Cape Fear River

This hypothetical evaluation was performed to estimate the length of time the storage volume of the Main Reservoir could sustain the HNP, HAR 2, and HAR 3 during drought conditions without makeup water flow from the Cape Fear River. The assumed water level elevation of the Main Reservoir was 73.2 m (240 ft.) NGVD29 at the start of the period in which no water is being withdrawn from the river.

The following assumptions were made in the computation:

- No net makeup flow from the Cape Fear River will be available for the entire period.

- No water will be released from the Main Reservoir other than that required to meet minimum flow requirements (no net consumption from the Cape Fear River).

- For the operation of HNP, HAR 2, and HAR 3, the lowest permissible water level in the Main Reservoir is 67.1 m (220 ft.) NGVD29.
- Outage periods are not considered.
- Main Reservoir area at elevation 67.1 m (220 ft.) NGVD29 = 3561.44 ac.
- Main Reservoir area at elevation 73.2 m (240 ft.) NGVD29 = 7179.33 ac.
- Average area for elevation range between 67.1 m (220 ft.) NGVD29 and 73.2 m (240 ft.) NGVD29 is (3561.44 ac. + 7179.33 ac.) / 2 = 5370.39 ac.
- Storage volume between elevations 67.1 m (220 ft.) NGVD29 and 73.2 m (240 ft.) NGVD29 is 177,563 ac-ft - 73,000 ac-ft = 104,563 ac-ft.
- Average monthly inflow volume into the Main Reservoir during a drought period is 1309 ac-ft/month.
- Average monthly volume of net evaporation is 979.4 ac-ft/month.
- Average monthly seepage volume is 446.25 ac-ft/month.
- Makeup pumping from the Main Reservoir to the Auxiliary Reservoir is 265.15 ac-ft/month.
- Average monthly volume of consumptive use from the Main Reservoir is 5596 ac-ft/month.
- Total monthly use volume from Main Reservoir is 979.4 ac-ft/month + 446.25 ac-ft/month + 265.15 ac-ft/month + 5596 ac-ft/month = 7287 ac-ft/month.
- Net monthly water use from the reservoir is 7287 ac-ft/month - 1309 ac-ft/month = 5978 ac-ft/month.

Using the assumptions above, the estimated length of time that the storage volume of the Main Reservoir between elevations 73.2 m (240 ft.) NGVD29 and 67.1 m (220 ft.) NGVD29 could sustain the HNP, HAR 2, and HAR 3 without makeup water flow from the Cape Fear River is 17.5 months (104,563 ac-ft / 5978 ac-ft/month = 17.5 months).

2.4.12 GROUNDWATER

HAR COL 2.4-4

The HAR site is located within the Harris Lake drainage basin (HUC-11) of the Piedmont Province immediately west of the physiogeographic boundary that separates the Coastal Plain and Piedmont provinces. The Piedmont Province is

characterized by crystalline bedrock overlain by varying thicknesses of unconsolidated material. The physical characteristics of the bedrock and regolith largely determine the water supply potential and quality of the groundwater system. This subsection presents groundwater conditions, sources, and usage of the aquifer in the region and at the HAR site.

2.4.12.1 Description and On-Site Use

2.4.12.1.1 Regional Groundwater Systems

Regional groundwater is associated with non-productive aquifers (producing little water) in the early Mesozoic, Durham basin of the Newark Supergroup (upper Triassic Series) (Reference 2.4-236). Geologic units of the Durham basin consist of claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate. These units have low effective porosity and poorly interconnected pores. Groundwater flows primarily along joints, fractures, and bedding planes, which create anisotropic conditions where most water movement is parallel to the strike of the beds (Reference 2.4-236). Exceptions to the Durham basin lithology are thin, vertically oriented, diabase dikes. These dikes, which are characterized by very low, primary porosity, generally yield little water. However, in some locations, the strata adjacent to the dikes have been fractured by the intrusion and water wells in these areas produce higher yields (Reference 2.4-236). The regolith associated with the Durham basin consists of a thin layer of dense, clayey soils and highly weathered bedrock.

The primary permeability of the Triassic age, sedimentary bedrock aquifer is very low. However, a secondary permeability occurs within the aquifer based on interconnected fractures. These fractures are common to depths of 30.5 m (100 ft.) below ground surface (bgs), but become less prevalent with increased depth. At depths greater than 121.9 m (400 ft.), the fractures are closed and sealed to water flow. Some interbedded lenses of relatively higher permeability exist. However, these units are not extensive and are commonly bound, both above and below, by materials with relatively lower permeability.

Results from a pilot study conducted within Triassic age sedimentary bedrock approximately 4.5 km (2.8 mi.) to the southwest of the HAR site, indicated fractures above 61.0 m (200 ft.) bgs are mostly parallel to the bedding plane and most fractures are steeply dipping (Reference 2.4-238). Results also indicated that hydraulically significant fractures along bedding planes generally occur along contacts where large contrasts in physical properties exist (e.g., a contact between coarse-grained to conglomeratic sandstones and underlying siltstones or claystones).

The regolith associated with the Durham basin consists of a thin layer of dense, clayey soil grading into highly to moderately weathered bedrock with increased depth. The thickness and texture of the regolith is largely dependent on the composition of the parent rock. Because the bedrock provides very low storage

volumes, most groundwater (by volume) is stored in the unconsolidated materials overlying the bedrock.

Recharge in the region occurs by percolation of precipitation through the overburden. However, most of the precipitation either is returned to the atmosphere through evapotranspiration or becomes surface runoff. The predominance of surface and near-surface deposits with extremely low permeability results in rapid runoff of precipitation. An average of 15 percent of precipitation within Wake County, North Carolina, recharges the groundwater aquifer; within the Triassic basin (includes Durham basin), recharge values are 8 percent or lower (Reference 2.4-239). Therefore, natural recharge to the aquifer occurs at a very low rate. The low percentage of precipitation that percolates downward is confined laterally by the diabase dikes and vertically by the absence of open fractures or joints at depth.

Maximum well yields in the Triassic age sedimentary rocks are typically less than 94.6 l/min (25 gpm), with average yields less than 37.9 l/min (10 gpm) (Reference 2.4-239). The use of groundwater in the region is limited because of the low well yields. A few communities in the area use the Triassic rocks as a source of water. However, total groundwater usage is still small, as discussed in Subsection 2.4.12.2.

2.4.12.1.2 Site Groundwater Systems

Investigations conducted in the vicinity of the HAR site reveal that geologic and hydrologic conditions are essentially the same as the regional conditions described in Subsection 2.4.12.1.1. The HAR site is located on a ridge bounded by the Auxiliary Reservoir to the west (upper Tom Jack Creek Branch), and the Main Reservoir to the southwest (lower Tom Jack Creek Branch), southeast (White Oak Creek Branch), and east (Thomas Creek Branch). Currently, the normal pool elevations of the Auxiliary Reservoir and the Main Reservoir are 76.8 m (252 ft.) and 67.1 m (220 ft.) NGVD29, respectively. The proposed normal pool elevation for the Main Reservoir is 73.2 m (240 ft.) NGVD29. The Main Reservoir was formed after PEC constructed an earthen dam across Buckhorn Creek in 1980. Buckhorn Creek enters the Cape Fear River about 11.3 km (7 mi.) south-southwest of the HAR site. The HNP site has been graded to an elevation of 79.2 m (260 ft.) NGVD29. The pre-graded site elevations ranged from about 64.0 m (210 ft.) to 85.3 m (280 ft.) NGVD29; the land surface generally sloped towards the east and southeast on the southern half and towards the north and northeast on the northern half of the site. The stratigraphy beneath the HAR site was evaluated through the completion of additional work by CH2M HILL in 2006 and a review of previously collected site data and data obtained from literature.

A site investigation was conducted at the HAR site during the summer of 2006 to characterize the regolith and bedrock. Sixty-eight boreholes were advanced to characterize the subsurface conditions at the HAR 2 and HAR 3 structure locations (Boreholes BPA-1 through BPA-50 and BGA-1 through BGA-18). At

each borehole, rotary drilling with standard penetration testing (SPT) was advanced through soil to the SPT refusal criteria depth (50 blows over 7.62 cm [3 in.]). Rock coring was then initiated, using double-tube wireline coring methods to the borehole termination depth. Borehole depths ranged from approximately 12.2 to over 61.0 m (40 to over 200 ft.) bgs.

2.4.12.1.2.1 Regolith

The regolith consists of residual soils formed in place by weathering of the underlying sedimentary bedrock of the Triassic basin, and the underlying highly to moderately weathered bedrock. The residual soil is predominantly lean clay and sand, with subordinate amounts of silt and fat clay. The regolith extends to depths ranging from approximately 0.6 to 4.6 m (2 to 15 ft.) bgs at HAR 2, and from approximately 3.0 to 9.1 m (10 to 30 ft.) bgs at HAR 3.

While constructing the HNP, the existing regolith was removed and stockpiled during site grading activities and used as fill for areas below the HNP nominal plant grade elevation of 79.2 m (260 ft.) NGVD29. No fill soil was required from outside locations that might have consisted of different soil types. During construction of the HAR, the same procedures are assumed for site preparation. Therefore, the existing and future regolith for HNP and HAR sites will consist of a mixture of native soil types.

Figure 2.4.12-201 shows the NRCS's classification of soils at the HAR site as the White Store–Creedmoor–Mayodan type (Reference 2.4-207). Table 2.4.12-201 summarizes physical soil and engineering properties by soil type. Characteristics of the White Store–Creedmoor–Mayodan soil types indicate a high percentage of fine soil textures with relatively high porosity but low saturated hydraulic conductivity. These soil characteristics are indicative of relatively impervious surfaces with limited infiltration and percolation.

2.4.12.1.2.2 Triassic Rocks

The HAR site and peripheral lands are underlain by Newark Supergroup (upper Triassic Series) bedrock, which consists predominantly of siltstone and sandstone, with subordinate amounts of claystone, shale, and conglomerate. The tightness of the rock formations, as a result of compaction and cementation, is evident from most of the cores extracted from the 2006 site investigation borings. Surface percolation of precipitation is controlled by the location of joints and fractures in the rocks. The low permeability of the Triassic age, sedimentary bedrocks suggests that any movement of groundwater within the rocks is controlled by the interconnecting patterns of joints and fractures.

Before the HNP was constructed, groundwater elevations across the site occurred principally within jointed, fractured bedrock, generally at depths of 9.1 to 27.4 m (30 to 90 ft.) below the original ground surface. Following completion of the HNP, surface water pressure from the Main Reservoir and the Auxiliary

Reservoir created new groundwater elevations at the HAR site. Groundwater depths now range between 0.6 and 9.1 m (2 and 30 ft.) bgs.

Water is supplied to the reservoirs by streamflow, direct precipitation, runoff, and a small amount of groundwater. Because of the impervious nature of the soils and bedrock, there is only insignificant interchange of water between the reservoirs and the aquifer.

2.4.12.1.3 On-Site Use of Groundwater

Groundwater beneath the site is not used for plant operation or drinking water. Given the low yield of the materials underlying the site, groundwater will not be used for HAR 2 or HAR 3.

2.4.12.2 Sources

2.4.12.2.1 Present and Future Groundwater Use

It is not likely that groundwater will be used at the HAR site during construction activities. No groundwater will be used for facility operation.

In September 2006, PEC performed a water use survey as part of the annual HNP Land Use Census Survey for the HNP. The closest residents located relative to the HAR site were surveyed concerning drinking water sources (groundwater, surface water, or public water supply) and well details, if known (Figure 2.4.12-202). Table 2.4.12.-202 lists results from the survey. The closest surveyed resident is about 1.9 km (1.2 mi.) from the HAR site within the north-northwest direction. Visual observations confirmed all surveyed residents had water wells located on the associated property. Private water wells ranged from 22.9 to 109.7 m (75 to 360 ft.) bgs in depth and were completed within bedrock aquifer systems. No other water well details or usage rates were available from private residents.

Table 2.4.12-203 lists water wells associated with public water supply users within 8 km (5 mi.) of the HAR site. Only two communities (one located in New Hill, North Carolina, and one located in Fuquay-Varina, North Carolina) use groundwater as a public water supply source within 8.0 km (5 mi.) of the HAR site. Both communities are located in Wake County. Water wells associated with these cities are located within the Carolina Slate Belt and not the Triassic basin. In the plant area, the same crystalline rocks are buried a few thousand feet beneath the Triassic sediments. Four transient, non-communities using a total of five water wells are located in New Hill, North Carolina, and Raleigh, North Carolina, about 0.97 to 5.97 km (0.6 to 3.71 mi.) from the HAR site (Reference 2.4-207). No well details or usage rates were available for the public water supply users.

Table 2.4.12-204 lists the past and projected future Cape Fear River drainage basin water use for the years of 1997 and 2010.

2.4.12.2.2 Groundwater Levels and Movement

Configuration of the potentiometric surface in the immediate vicinity of the HAR site was determined by measuring water levels in piezometers and monitoring wells installed after completing the HNP and during the HAR site investigation conducted from June through August of 2006. On June 6 and 7, 2006, a well survey and gauging event were conducted at the HNP to determine the status of post-construction HNP monitoring wells and piezometers. An additional 21 monitoring wells were installed during the HAR site investigation to more accurately characterize the potentiometric surface, gradient, and flow pathways within the vicinity of HAR 2 and HAR 3. Nine nested well pairs (18 out of 21 wells) were installed during the investigation to determine the connectivity between the surficial and bedrock aquifers. Shallow monitoring wells were screened within the regolith directly above the residual soil/bedrock interface. Deep monitoring wells were screened completely within the Newark Supergroup (upper Triassic Series) bedrock. Groundwater gauging events were conducted quarterly (August 2006, November 2006, February 2007, and May 2007) to account for seasonal and long-term variations. Table 2.4.12-205 summarizes well construction details; Figures 2.4.12-203, 2.4.12-204, 2.4.12-205, 2.4.12-206, 2.4.12-207, 2.4.12-208, 2.4.12-209, and 2.4.12-210 show potentiometric contour maps for each of the quarterly events.

Table 2.4.12-206 provides recent groundwater elevations for the period of August 2006 through May 2007. Historically, water level measurements in the bedrock aquifer collected before the construction of the HNP indicated the groundwater flow direction beneath the site was southeast toward White Oak Creek (pre-reservoir conditions). The Main Reservoir, which began filling in December 1980, reached its current normal operating level of 67.1 m (220 ft.) NGVD29 in January 1983. The Auxiliary Reservoir filled to its operating level of 76.2 m (250 ft.) NGVD29 in March 1983. The operating level has since been changed to 76.8 m (252 ft.) NGVD29, which is the crest elevation of the Auxiliary Spillway.

Current groundwater conditions are heavily influenced by surface water pressure from the Main Reservoir and the Auxiliary Reservoir. The HAR site and the HNP are bounded by the Auxiliary Reservoir to the northwest, west, southwest, and south (Emergency Service Water Intake Channel) and the Main Reservoir to the northeast, east, southeast, and south (Cooling Tower Makeup Water Intake Channel). The Emergency Service Water Discharge Channel separates the HAR site from the HNP on the western half of the plant site. The only area not bound by a surface water body is north of the HAR site. This area is characterized as a topographic high (maximum ground surface elevation of approximately 91.4 m [300 ft.] NGVD29). The water table in the vicinity of the HAR site is influenced by the topographic high and occurs as a ridge-like mound northwest of HAR 3. The position of the groundwater divide marks a recharge area from which groundwater flows west toward the Auxiliary Reservoir, south toward the Emergency Service Water Discharge Channel, and east toward the Thomas Creek Branch of the Main Reservoir. Groundwater south of the Emergency

Service Water Discharge Channel, which is influenced by the Auxiliary Reservoir, generally flows to the southeast and east toward the Thomas Creek Branch of the Main Reservoir. The current direction of groundwater flow beneath the site in the surficial/overburden and bedrock aquifers is east in the proposed locations of HAR 2 and HAR 3, and east and southeast at the HNP.

Nine nested well pairs were installed during the 2006 HAR site investigation to determine the vertical gradient between the surficial and bedrock aquifers. Shallow monitoring wells were screened within the regolith directly above the residual soil/bedrock interface. Deep monitoring wells were screened completely within the bedrock. Six of the nine nested well pairs had a greater hydraulic head within the surficial aquifer than the bedrock aquifer; this condition creates a downward vertical gradient (Table 2.4.12-207). Nested well pairs MWA-3S/D and MWA-8S/D located within the footprint of the safety-related structures for HAR 2 and HAR 3 had downward vertical gradients with elevation head differences as measured in the field on August 28, 2006 of 1.01 and 2.74 m (3.3 and 9 ft.), respectively (Table 2.4.12-206). Only three nested well pairs MWA-4S/D, MWA-9S/D, and MWA-10S/D installed immediately upgradient of suspected localized diabase dikes had an upward vertical gradient; elevation head differences as measured in the field on August 28, 2006 were 0.6 m, 2.0 m, and 0.3 m (1.9 ft., 6.4 ft., and 0.9 ft.), respectively. In monitoring well MWA-9D, the groundwater elevation within the bedrock aquifer was above the ground surface elevation creating flowing artesian conditions. Vertical gradients between the surficial and bedrock aquifers remained consistent for all nested well pairs during each quarterly gauging event except nested well pair MWA-10S/D. Nested well pair MWA-10S/D had an upward vertical gradient during the August 2006 and November 2006 gauging events but a downward vertical gradient during the February 2007 and May 2007 gauging events.

“Typical” seasonal variations (higher groundwater levels in the spring, lower groundwater levels in the fall and summer) are not consistent within the shallow or deep monitoring wells.

2.4.12.2.3 Site Hydrogeologic Characteristics

In the vicinity of the HAR site, a thin regolith overlying Triassic age, sedimentary rock consists of clayey soils and highly weathered bedrock, which have low groundwater yields. The Triassic rocks, which are thick and widespread in extent, constitute the principal aquifer in the area. However, because of compaction and cementation of individual rock layers, the bedrock aquifer produces little water, and is a minor groundwater source.

Numerous borings were advanced during initial field investigations conducted for the HNP to gather geologic information in the plant site and reservoir areas. The bedrock below the regolith and highly weathered bedrock zones appears to have two distinct components of permeability. The primary permeability of the bedrock matrix is very low. However, a secondary permeability occurs within the aquifer based on interconnected fractures. This principal component is measured as

permeability during hydrogeologic tests at the site. On-site borings confirm that fractures within the Triassic rocks are filled with water below the water table. These fractures are common to depths of 30.5 m (100 ft.), but become less prevalent with increased depth. At a depth greater than 121.9 m (400 ft.), the fractures are closed and sealed to water flow.

Permeabilities determined from the down-hole pressure tests within the Triassic rocks were conducted during past site investigations in borings located in the vicinity of the HNP. Three borings were tested in 3.0 m (10 ft.) intervals under pressures up to 110 pounds per square inch (psi) at depth intervals ranging from 3.0 to 44.2 m (10 to 145 ft.) bgs. Down-hole pressure test results ranged from 4.7×10^{-7} to 2.37×10^{-4} centimeter per second (cm/sec) (0.47 to 2.54 feet per day [ft/day]) in fine sandstone, 6.7×10^{-6} to 4.2×10^{-4} cm/sec (6.71 to 12.93 ft/day) in shaley siltstone, and 1.31×10^{-6} to 2.91×10^{-6} cm/sec (1.31 to 2.91 ft/day) in siltstone. Test results indicated zones with small water losses under high pressure were vertically positioned between dense, impervious rock layers. These impermeable intervals showed no water losses during pressure testing and ranged in thickness from 3.0 to 4.6 m (10 to 50 ft.)

Yields from known wells in the area generally range up to 75.7 l/min (20 gpm), but average only about 18.9 l/min (5 gpm) or about 0.03 gallons per minute per foot (gpm/ft) of well. Of 57 wells with an average depth of 48.2 m (158 ft.) constructed in the Triassic formation in western Wake County, 16 percent yield less than 3.8 l/min (1 gpm) (Reference 2.4-240). Such relatively low permeability also explains why the Triassic formation is the lowest producing groundwater source in the region.

Generally, the principal areas of groundwater storage in the Triassic basin are found near diabase dikes that have intruded the Triassic sediments. During construction of the HNP, 20 water wells were installed near the diabase dikes to provide water for use during construction activities. These water wells were abandoned or removed from service during HNP's operational status. Based on a total capacity of 757 l/min (200 gpm) for seven wells completed in 1973 and a total capacity of 946 l/min (250 gpm) for eight wells completed over the 1977 to 1979 period, the average discharge rate for the 15 wells was approximately 113.6 l/min (30 gpm). Additional information from six site wells located in the proximity of dikes yielded specific capacity values from 24-hour driller's tests that ranged from 0.16 to 0.59 gpm/ft. The specific capacity values correspond to transmissivity values of about 3.7 to 12.1 square meters per day (m^2/day) (40 to 130 square feet per day [ft^2/day]). According to observed behavior of water in a diabase dike fracture system during pumping tests at existing wells W-13 and W-15, it is possible that measurable changes in the water level may occur a few hundred feet from the reservoir in such fracture systems.

Hydrophysical logging methods were used to locate conducting features and estimate flow capacities within Triassic age, sedimentary bedrock. Logging was performed at a pilot study site located approximately 4.5 km (2.8 mi.) to the southwest of the HAR site. Flow-rate values measured at the site ranged from

0.004 to 4.01 l/min (0.001 to 1.06 gpm) (Reference 2.4-238). The distribution of conductive zones was concentrated within the weathered zone, along strata concordant fractures, and within an enhanced fracture zone at a fault. Most of the conductive zones were located within the weathered zone. In addition to hydrophysical logging, packer tests were performed at targeted conductive intervals. Results from packer tests indicated transmissivity values within the weathered zone ranged from 1 x 10^{-4} to 1 x 10^{-2} square centimeters per second (cm^2/sec) (0.009 to 0.9 ft^2/day) and within the unweathered zone from 1 x 10^{-5} to 1 x 10^{-1} cm^2/sec (0.0009 to 9 ft^2/day). Strata-concordant or fault fractures within the unweathered zone were typically more transmissive with values of 1 x 10^{-2} to 1 x 10^{-1} cm^2/sec (0.9 to 9 ft^2/day) (Reference 2.4-238).

To confirm the permeability results of the site investigation for the HNP, the slug test method was used within 18 monitoring wells at the HAR site. This method was used to determine in-situ permeability or hydraulic conductivity values for the surficial and bedrock aquifers. Table 2.4.12-208 summarizes the test results. Average horizontal permeability values range from 5.1 x 10^{-5} cm/sec (0.1 ft/day) to 1.9 x 10^{-3} cm/sec (5.4 ft/day) in the surficial aquifer and 8.6 x 10^{-7} cm/sec (0.002 ft/day) to 3.0 x 10^{-4} cm/sec (0.8 ft/day) in the bedrock aquifer. These values are indicative of low permeability conditions and reflect the results from site investigations for the HNP.

Linear groundwater velocity and Darcy flux estimates for the surficial and bedrock aquifers were calculated using site parameters for HAR 2 and HAR 3. Table 2.4.12-209 shows the results for the seepage velocity and Darcy flux. Nested monitoring wells were selected both upgradient and downgradient, where possible, for each proposed Unit, to more accurately compare the surficial and bedrock aquifers. For HAR 2, the seepage velocity and Darcy flux for the surficial aquifer between monitoring wells MWA-3S and MWA-5S are about 0.5 ft/day and 0.052 cubic feet per day (ft^3/day), respectively; for the bedrock aquifer, the seepage velocity and Darcy flux between monitoring wells MWA-3D and MWA-5D are about 0.09 ft/day and 0.0046 ft^3/day, respectively.

Similar estimates were calculated for HAR 3. The seepage velocity and Darcy flux for the surficial aquifer between monitoring wells MWA-7S and MWA-9S are about 1.2 ft/day and 0.1 ft^3/day, respectively. For the bedrock aquifer, the seepage velocity and Darcy flux between monitoring wells MWA-7D and MWA-9D are about 0.3 ft/day and 0.013 ft^3/day, respectively. Monitoring wells MWA-7D and MWA-9D are upgradient and downgradient of HAR 3. When comparing nested monitoring wells MWA-8S/D and MWA-9S/D, the bedrock values dramatically change. The seepage velocity and Darcy flux for the surficial aquifer between monitoring wells MWA-8S and MWA-9S are about 1.5 ft/day and 0.15 ft^3/day, respectively. For the bedrock aquifer, the seepage velocity and Darcy flux between monitoring wells MWA-8D and MWA-9D are about 0.002 ft/day and 0.000087 ft^3/day, respectively. A small gradient between the bedrock monitoring wells creates a numerical change in magnitude for the bedrock aquifer downgradient of HAR 3. MWA-9D had a measured

depth-to-water above the ground surface that created artesian conditions. These conditions are assumed to be associated with localized diabase dikes.

2.4.12.2.4 Effects of Groundwater Usage

The population in the vicinity of the plant is small and groundwater usage is minimal because well yields are low. Only a limited supply of groundwater is obtainable from the proximity of diabase dikes. Therefore, any increase in groundwater usage will be limited because of the poor permeability and storage characteristics of the aquifer. In addition, PEC has acquired most of the land within a 3.2-km (2-mi.) radius, and some beyond this distance. The future population in the plant vicinity should not greatly increase, and groundwater usage will remain essentially the same.

It is not likely that groundwater will be used at the HAR site during construction activities. No groundwater will be used for facility operation.

Limited dewatering of site excavations is anticipated during construction activities. The effects of dewatering on groundwater gradients and flow pathways within the shallow and bedrock aquifers are considered minimal and will not affect local groundwater users.

No potential exists from the local area to reverse or affect groundwater flow within the surficial or bedrock aquifers at HAR site.

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 Subsection 2.4.12.2, Sources. A conservative analysis of critical groundwater pathways for a liquid effluent release at the site is provided in Subsection 2.4.13, Accidental Releases of Radioactive Liquid Effluents in Ground and Surface Waters, along with the determination of groundwater and radionuclide travel times to the nearest downgradient groundwater user or surface body of water.

2.4.12.4 Monitoring or Safeguard Requirements

This section provides a brief summary of the monitoring programs to be used to protect the present and projected groundwater users in the vicinity of the HAR 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 HAR 2 and HAR 3, 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 Pre-application Monitoring Program for groundwater will support the assessment of site acceptability and establish background conditions for groundwater prior to the construction and operation of HAR 2 and HAR 3.
- A Construction Monitoring Program for groundwater will monitor and control potential effects caused by site preparation and construction activities.
- A Pre-operational Monitoring Program will establish a baseline database for identifying and assessing environmental effects attributable to the operation of HAR 2 and HAR 3.
- A limited Operational Monitoring Program will be implemented to document groundwater conditions and detect any unexpected effects to the groundwater system from the operation of HAR 2 and HAR 3. Based on the monitoring data for the HNP, the Operational Monitoring Program is anticipated to extend over a 5-year period, or until conditions appear to have stabilized, based on the trend analysis. Modifications to the monitoring program (e.g., changes in monitoring locations or collection procedures) will be assessed regularly over the duration of the program.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

HAR COL 2.4-1

The HAR site does not anticipate employing a permanent active dewatering system. Flood design and protection for the AP1000 design is provided in the 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 proposed nominal plant grade of 79.6 m (261 ft.) NGVD29.

A permanent dewatering system was not used during the construction of HNP. Groundwater elevations greater than 76.5 m (251 ft.) NGVD29 were measured in some areas of the plant site. During construction activities, a lack of significant inflow of groundwater into the completed plant block excavation indicated the groundwater in the bedrock occurs in widely separated joints and bedding planes. Rain or surface water accumulation during construction activities was removed with sump pumps.

Groundwater gauging events were conducted in August 2006, November 2006, February 2007, and May 2007 to account for seasonal and long-term variations in the surficial and bedrock aquifers at the plant site. Nested monitoring well pairs MWA-3S/D and MWA-8S/D were installed within the reactor locations of HAR 2 and HAR 3. Surficial aquifer monitoring wells MWA-3S and MWA-8S recorded the highest groundwater elevations, which ranged from 78.21 to 79.31 m

(256.60 to 260.19 ft.) NGVD29 to 77.86 to 78.36 m (255.45 to 257.08 ft.) NGVD29, respectively. Site-specific groundwater elevations did exceed an elevation of 78.9 m (259 ft.) NGVD29 during the February 2007 and May 2007 gauging events in monitoring well MWA-3S, which is located in the proposed location of HAR 2. The average groundwater elevation for the four monitoring events within MWA-3S is 78.63 m (257.99 ft.) NGVD29.

Final grading of the plant site will result in a hydrologic alteration, including the permanent change in groundwater levels within the plant site from site grading and a series of stormwater drainage ditches. North of the plant site, the area is characterized as a topographic high (maximum ground surface elevation of approximately 91.4 m [300 ft.] NGVD29). As specified in Subsection 2.4.12.2.2, the water table in the vicinity of the HAR site is directly influenced by this topographic high and occurs as a ridge-like mound northwest of HAR 3. The position of the groundwater divide marks a natural recharge area from which groundwater flows west toward the Auxiliary Reservoir, south toward the Emergency Service Water Discharge Channel, and east toward the Thomas Creek Branch of the Main Reservoir.

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 HAR facilities. Stormwater drainage ditches installed approximately 594.4 m (1950 ft.) and farther north of HAR 3 will have bottom elevations ranging from approximately 80.8 m (265 ft.) NGVD29 or lower, while drainage ditches as close as approximately 137.2 m (450 ft.) north of HAR 3 will have bottom elevations ranging from approximately 78.0 m (256 ft.) NGVD29 or lower (Figure 2.4.1-205). This network of stormwater drainage ditches will intersect the water table based on known groundwater elevations and effectively lower the existing water table within the vicinity of the HAR facilities.

The series of drainage ditches surrounding the plant construction areas and the HAR facilities will form a collective barrier for the flow of groundwater into and out of the HAR facility. Groundwater will migrate to the lower open elevations in the ditch bottoms, resulting in a final water table at or slightly higher than the ditch bottom elevations. The ditches encompass the plant facilities where the final grade elevations outside of the facility limits are higher than the final plant grade of 79.6 m (261 ft.) NGVD29. They will also intercept flow in the surficial aquifer towards HAR 2 and HAR 3. These ditches will act as a natural barrier to the groundwater flow, preventing it from passing into the plant area and raising the groundwater level above the ditch bottom elevations. The groundwater levels may rise during periods of intense precipitation, but these elevated levels will be temporary. Groundwater flow within the surficial material will be redirected towards these ditches and will ultimately discharge into the Main Reservoir. Groundwater elevations at HAR 3, which are expected to decrease to 78.0 m (256 ft.) NGVD29 or lower, 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.

A pre-operational and operational groundwater monitoring program will be designed and implemented to monitor the effectiveness of the surface water drainage system for HAR 2 and HAR 3. The groundwater monitoring program will monitor both up-gradient and side-gradient groundwater elevations in the vicinity of HAR 2 and HAR 3 to identify any potential groundwater mounding issues near the safety-related structures. This program will begin prior to operations and will be incorporated into the overall groundwater monitoring program for HNP and HAR. The results of the monitoring program will be continually evaluated, and a mitigation plan will be created if groundwater elevations near the safety-related structures threaten to exceed the DCD groundwater criteria.

2.4.13 ACCIDENTAL RELEASES OF RADIOACTIVE LIQUID EFFLUENT IN GROUND AND SURFACE WATERS

HAR COL 2.4-5

This subsection presents a conservative analysis of the effect of accidental release of liquid effluents to the groundwater environment. A release of the contents of the waste liquid system effluent hold-up tank is postulated. The groundwater transport of radionuclides through the surficial and bedrock aquifers is examined, although the surficial aquifer pathway is considered the more likely of the two scenarios. The resultant nuclide concentrations in the nearest public water supplies are evaluated and compared to 10 Code of Federal Regulations (CFR) 20 regulatory limits. Water “supplies” are defined as a well or surface water intake that is used for direct human consumption or indirectly through animals, crops, or food processing.

2.4.13.1 Groundwater Scenarios

Failure of the waste effluent hold-up tank is considered. Evaluations performed by Westinghouse determined that these tanks have the greatest potential radionuclide inventory of all waste effluent system tanks. There are two 105,992 liter (28,000 gal.) waste effluent hold-up tanks per Unit. One tank is postulated to fail in a Unit. The failed waste effluent hold-up tank is assumed to have nuclide maximum concentrations corresponding to 101 percent of the reactor coolant term with:

- A tritium source term of 1.0 microcuries per gram (μCi/g).
- Corrosion product concentrations for chromium isotope 51 (Cr-51), manganese isotopes 54 and 56 (Mn-54 and Mn-56), iron isotopes 55 and 59 (Fe-55 and Fe-59), and cobalt isotopes 58 (Co-58) and 60 (Co-60) taken from Table 11.1-2 of the DCD.
- All other isotopes are based on DCD Table 11.1-2 scaled to a defective fuel fraction of 0.12 percent as per NUREG-0017.

Table 2.4.13-201 shows the tank inventory released to the groundwater.

2.4.13.1.1 Surficial Aquifer

The contents of the waste effluent hold-up tank are assumed to be released immediately from the Auxiliary Building. No credit is taken for hold-up by the building's sealed walls. The effluent is released to the surficial aquifer, which has water table elevations ranging from about 0.7 to 9.1 m (2 to 30 ft.) below grade (Table 2.4.12-206). The release migrates eastward 365.8 m (1200 ft.) in the direction of decreasing hydraulic head to the Thomas Creek branch of the Main Reservoir. The potentiometric map in Figure 2.4.12-203 shows that the water table elevation decreases from about 78.3 m (257 ft.) NGVD29 near the HAR 2 and HAR 3 Auxiliary Buildings to 70.1 m (230 ft.) NGVD29 next to the Main Reservoir. The map also shows a hydraulic upgradient toward the Auxiliary Reservoir. The upgradient precludes significant groundwater flow westward to the Auxiliary Reservoir.

The release remains in the surficial aquifer between the point of release and the Main Reservoir. The bedrock below the surficial aquifer is essentially impermeable with relatively small volumes of groundwater residing in the network of narrow cracks in the bedrock. Recharge of the bedrock aquifer from the surficial aquifer/saturated regolith at the site is limited, as revealed by the small vertical hydraulic gradients (up and down directions) between surficial and bedrock aquifers shown in Table 2.4.12-207. As shown in Table 2.4.12-209, the horizontal hydraulic gradients are similar in magnitude to the vertical gradients in Table 2.4.12-207. Flow based on the potentiometric surface is essentially towards the Main Reservoir.

No water-supply wells withdraw water from the aquifer between the Units (HAR 2 and HAR 3) and the Main Reservoir. The site takes surface water from the Main Reservoir for domestic, process, and Cooling Tower make-up. Spillover from the reservoir's Main Dam will flow to the Buckhorn Creek before discharging to the Cape Fear River. The nearest downstream public user of surface water is Lillington, North Carolina, about 22.0 km (13.7 mi.) downstream from the confluence of the Cape Fear River and Buckhorn Creek.

Radionuclide concentrations are reduced during transport through the saturated regolith by adsorption, dispersion, and radioactive decay. In addition, hold-up and dilution occurs in the Main Reservoir with its storage capacity of 2.2 x 10^8 m^3 (177,563 ac-ft) at the nominal 73.2-m (240-ft.) water elevation. Flow from the Cape Fear River further dilutes the nuclide concentrations for surface water use at Lillington. Average monthly Cape Fear River flow above Lillington is 95.9 m^3/s (3387 cfs) (Table 2.4.1-204) (Reference 2.4-271). The minimum downstream release from the Main Dam is expected to be 0.57 m^3/s (20 cfs). Average annual flow in the Thomas Creek branch of the Main Reservoir is greater than 0.057 m^3/s (2 cfs).

2.4.13.1.2 Bedrock Aquifer

A substantial release of nuclides to the bedrock aquifer is unlikely due to the overlaying surficial aquifer and regolith. The radionuclides are expected to move through the surficial aquifer to the Main Reservoir before significant migration of nuclides to the bedrock aquifer occurs.

Water withdrawn by wells from the bedrock aquifer is supplied from the flow in the network of small cracks and fractures in the bedrock. The bedrock aquifer can extend to 121.9 m (400 ft.), although the first 30.5 m (100 ft.) is the most productive region for water supply to wells. Because bedrock is largely impermeable, the aquifer’s storage capacity is limited to the volume of the cracks and fractures. Recharging of the bedrock aquifer takes place over large regions that extend greatly beyond HAR 2 and HAR 3.

The potentiometric map in Figure 2.4.12-204 shows that the bedrock aquifer’s water table decreases eastward from the Auxiliary Building to elevation 77.7 m (225 ft.) NGVD29 near the Main Reservoir, similar to the surficial aquifer. Recharge between the Auxiliary Building and the Main Reservoir to the upper region of the bedrock would likely surface at the Main Reservoir. A small amount may be transported beyond the reservoir via cracks in the bedrock. In order to estimate impact to public wells near the site, the entire release is assumed into the bedrock aquifer. Tables 2.4.12-202 and 2.4.12-203 identify private and community well users. The nearest private well downgradient is 3.2 km (2 mi.) east of the site; the nearest community well is 7.9 km (4.9 mi.) east-southeast of the site.

2.4.13.1.3 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 aquifer’s saturated soil or bedrock. In this region, the water flows along meandering paths through the pores and around grains of the soil while some is trapped in microscopic voids. The dissolved radionuclides in solution moving through the soil 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 (Reference 2.4-241):

$$\partial C/\partial t = 1/R_d \left(-U_x\, \partial C/\partial x + Dx\, \partial^2 C/\partial x^2 + D_y\, \partial^2 C/\partial y^2 + D_z\, \partial^2 C/\partial z^2 \right) - \lambda C$$

where

C is the concentration of nuclide i,

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, and

λ is the radionuclide decay constant.

The retardation factor is given by

$$R_d = 1 + \rho/n_e K_d$$

where

ρ is the bulk dry density of the material through which the groundwater moves, and

n_e is the effective porosity, which is the effective volume of the material where water is actually free to move.

Kd is the distribution coefficient which is dependent on soil and chemical properties.

The seepage or pore velocity is calculated as

$U_x = v/n_e$

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 determined from the soil hydraulic conductivity and water table head gradient data in Table 2.4.12-209.

A general solution for the groundwater concentration C(x, y, z, t) in an aquifer is

$$C(x, y, z, t) = C_0/(n_e\, Rd)\, X(x, t)\, Y(y, t)\, Z(z, t)$$

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 (Reference 2.4-241).

The radionuclides transported to the Main Reservoir are estimated from the rate that activity crosses an imaginary plane perpendicular to the x-directed flow near the reservoir. After substituting in the Green's functions appropriate to a point source configuration and integrating out the transverse (y and z) dependencies, the rate that activity crosses a plane at distance x in the aquifer and flows into the surface water is

$$F(x, t) = C_0/R_d\, [U_x\, X(x, t) - D_x\, \partial X(x, t)/\partial x]$$

An activity balance that accounts for hold-up and dilution in the Main Reservoir is then written using F(x, t) to determine the reservoir's concentration. The resulting expression for C(x, t) is then maximized as $t \rightarrow xR_d/U_x$

$$C_{max}(x) = \frac{C^* V_T\, U_x \exp\{-\lambda x R_d/U_x\}}{2\, R_d\, (Q + \lambda V_R)\, (\pi\, \alpha_L\, x)^{1/2}}$$

The concentration $C_{max}(x)$ is then used to bound the reservoir's concentration and make comparisons to regulatory limits.

The maximum concentration at a well in the bedrock aquifer is taken as the aquifer's concentration at the distance downgradient from the point of release with vertical mixing assumed across 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

$$C_{max}(x) = \frac{C^* V_T \exp\{-\lambda\, x\, R_d/U_x\}}{4\, \pi\, n_e\, R_d\, h\, x\, (\alpha_L\, \alpha_T)^{1/2}}$$

where

V_R is the volume of the reservoir,

C^* is the concentration in the hold-up tank

V_T is the tank's volume,

Q is the net diluting flow to the reservoir,

h is the productive depth of the aquifer,

α_L and α_T are the longitudinal and transverse dispersivities.

2.4.13.1.4 Distribution Coefficient and Dispersivity

Screening evaluations showed that tritium, cesium, and strontium are the larger contributors to the effective concentration limits (ECLs) in 10 CFR 20, Appendix B, Table 2.

Distribution coefficients Kd for cesium and strontium were selected based on important soil and chemical properties affecting the adsorption of radionuclide contaminants. Kd values were selected using U.S. Environmental Protection Agency (USEPA) guidance for selection of conservative distribution coefficients (Reference 2.4-242).

Cesium's Kd correlates well to the cation exchange capacity of the soil or, alternatively, its pH and clay content. The Creedmoor–Mayodan–White Store composite above the bedrock includes clay soils in combination with varying amounts of silt and sand soil, as shown in Table 2.4.12-201. NRCS physical data show that clayey soils make up more than 20 percent of the composites, while chemical data show that the cation exchange capacity is greater than 5 milliequivalents per 100 grams (g) (Reference 2.4-243). These conditions yield a minimum Kd of 30 milliliters per gram (ml/g) for cesium (Reference 2.4-242).

pH is an additional constraint for the selection of a minimum Kd for strontium. The White Store–Creedmoor–Mayodan soils have pHs ranging from 3.5 to 6. The clay composition, cation exchange capacity, and pH result in a minimum Kd of 10 ml/g for strontium.

The bedrock consists mainly of sandstone and siltstone with secondary amounts of claystone. USGS data for the Piedmont bedrock aquifer indicate the aquifer water is slightly acidic, with a median pH of 6.7 (Reference 2.4-244). The minimum Kd are based on the USEPA lower estimates for minimal clay content and $5 < pH < 8$. The minimum Kd values are 10 ml/g for cesium and 2 ml/g for strontium (Reference 2.4-242). General confirmation of these distribution coefficients can be obtained from the compilation of Kd for sandy and clayey soils in Thibault et al. (Reference 2.4-245).

Adsorption is not credited for tritium or other radionuclides. Hence, Kd is 0 for all nuclides except cesium and strontium in this analysis.

Table 2.4.13-202 tabulates the transport times for the center of the nuclide plume to reach the Main Reservoir and well site. These effective times include the effect of adsorption 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 that longitudinal dispersivity ranges from $1 \text{ m} < \alpha_L < 130 \text{ m}$ for non-alluvial aquifers (Reference 2.4-241). This analysis uses the minimum value identified, $\alpha_L = 1$ m, in order to conservatively maximize the concentration reaching the reservoir, and $\alpha_L * \alpha_T = 1 \text{ m}^2$ (Reference 2.4-246) for well concentrations.

2.4.13.1.4.1 Confirmation of HAR Distribution Coefficients

Kd values for Cs and Sr were confirmed by testing at Argonne National Laboratory of HAR-specific samples (Reference 2.4-268). Twelve bedrock, six soil, and two diabase dike samples were taken from HAR core borings. Samples are identified in Table 2.4.13-206 while Figure 2.5.4-202 shows the locations of the bore holes. Surficial and bedrock groundwater from HAR wells MW-3S and MW-3D was used for saturation of samples and preparation of tracer solutions.

Table 2.4.13-206 gives the minimum Kd values determined for Cs and Sr. The Kd test results confirm that the Kd values in Table 2.4.13-202 used in the groundwater transport analysis are conservative for HAR.

2.4.13.1.4.2 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 Fe, Co, Ni and Mn. Transition metals typically have oxidation states +2, +3, +4 and +6 that allow the complex to form. With the exception of Cs and Sr, the HAR groundwater transport analysis does not credit retardation of radionuclides, that is, Kd = 0 ml/g. Thus, all other nuclide concentrations analyzed for the surficial and bedrock aquifer pathways are conservatively bounded by the existing analyses.

Unlike transition metals, aqueous complexing is not thought to greatly influence Cs and Sr behavior in most groundwater systems. Cs and Sr are alkaline and alkaline earth metals, respectively. There is little tendency for Cs or Sr to form aqueous complexes: Cs+ is mono-valent; studies of leachates from contaminated sites showed essentially all Sr existed as uncomplexed Sr+2 (References 2.4-242 and 2.4-270). Complexing of Sr+ and 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 confirmatory Kd testing described in FSAR Subsection 2.4.13.1.4 used site groundwater, soils, and bedrock samples. The results of the test implicitly included the effects of naturally occurring complexes should they be present near the HAR.

The waste effluent tank radionuclide inventory is from reactor coolant received from Chemical Volume and Control System (CVS) and sampling systems, and miscellaneous leakage. The use of chelates and complexing agents in the AP1000 reactor coolant system is not anticipated for routine operation. 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 buildup (AP1000 DCD Subsection 9.3.6.2.3.3). Hydrazine or hydrogen peroxide may be injected in small quantities at start-up or during cooldown, respectively (AP1000 DCD Table 5.2-2).

Chelating and complexing agents are not routinely used in other portions of the AP1000 liquid radwaste system, for example, the chemical waste or waste hold-up tanks. 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 HAR would need to be reviewed by PEC prior to implementation, as would any maintenance activity that would use EDTA or other industrial chelates.

PEC determined that EDTA and other chelating agents have not been spilled or released at the HAR or HNP site. However, trisodium phosphate (TSP), a possible complexant, has been used to remove Ca and Mg deposits in boilers at HNP in the 1990s. Only small quantities were used and large quantities were never stored on-site. There have been no spills of TSP. There is no potential impact from prior TSP usage.

2.4.13.1.5 Compliance with 10 CFR Part 20

Table 2.4.13-201 identifies the nuclide source term in the waste effluent hold-up tank and the public exposure ECL for liquid effluents given in 10 CFR 20, Appendix B, Table 2.

The most probable groundwater pathway is a release to the surficial aquifer where activity flows to the Main Reservoir. The Main Reservoir is used for domestic water supply at the plant. Discharge from the reservoir spillway flows downstream to Lillington, where water is withdrawn for the public water supply.

Table 2.4.13-203 shows the minimum dilution factors and maximum activity concentrations in the Main Reservoir. The dilution factor includes the effects of radioactive decay and dispersion in the aquifer. Extremely small or zero dilution factors are indicative of significant radioactive decay prior to nuclides reaching the Main Reservoir. Table 2.4.13-203 also gives the relative fraction for each nuclide's concentration to the ECL. Each nuclide's activity concentration in the reservoir is much less than its ECL.

The 10 CFR 20, Appendix B, Table 2 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 Table 2.4.13-203 and shows that the concentrations in the Main Reservoir are 0.8 percent of the effective dose equivalent regulatory limit. The maximum activity concentrations are diluted by the Cape Fear River flow prior to being withdrawn at Lillington's water supply. The dilution causes the activity concentrations to be less than 0.004 percent of the effective dose equivalent limit.

The site's domestic water intake is on the Thomas Creek Branch of the Main Reservoir. Table 2.4.13-204 gives the nuclide concentrations at this location. The activity concentrations are 14 percent of the effective dose equivalent limit allowed by 10 CFR 20.

A significant release to the bedrock aquifer is unlikely. However, the impact on public and private well 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 domestic supply well 3.2 km (2 mi.) east of the plant. The maximum activity concentrations are 0.4 percent of the effective dose equivalent limit for water withdrawn from wells in the bedrock aquifer. Doses to users at the nearest community well are orders of magnitude less than those at the private

well because of the 7.9 km (4.9 mi.) distance between the site and the community well.

It is concluded that an accidental release of effluents to groundwater would result in effective dose equivalents that are small fractions of the limits in 10 CFR 20 for water supplies derived from groundwater aquifers.

2.4.13.2 Surface Water

No outdoor tanks contain radioactivity in the AP1000 design. Therefore, no accident scenario could result in the significant release of liquid effluents directly to surface water.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

HAR COL 2.4-6 The HAR 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 AP1000 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 HAR 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 Subsection 2.4.2.3, Effects of Local Intense Precipitation. The effects of PMP on the Buckhorn Creek drainage basin and the resulting PMF (including wind setup, wave height, wave period and wave runup) are described in Subsection 2.4.3, Probable Maximum Flood on Streams and Rivers. The effects of wave generating wind activity from a PMH are described in Subsection 2.4.5, Probable Maximum Surge and Seiche Flooding.
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

HAR COL 2.4-1 This Combined License (COL) Item is addressed in Subsection 2.4.1.2.

2.4.15.2 Floods

HAR COL 2.4-2 This COL Item is addressed in Subsections 2.4.2, 2.4.3, 2.4.4, 2.4.5, 2.4.6, and 2.4.10.

2.4.15.3 Cooling Water Supply

HAR COL 2.4-3 This COL Item is addressed in Subsection 2.4.1.1.

2.4.15.4 Groundwater

HAR COL 2.4-4 This COL Item is addressed in Subsection 2.4.12.

2.4.15.5 Accidental Release of Liquid Effluents in Ground and Surface Water

HAR COL 2.4-5 This COL Item is addressed in Subsection 2.4.13.

2.4.15.6 Emergency Operation Requirement

HAR COL 2.4-6 This COL Item is addressed in Subsection 2.4.14.

2.4.16 REFERENCES

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Table 2.4.1-201 (Sheet 1 of 2)
Monthly Mean Streamflow Measurements for Buckhorn Creek, North Carolina

Buckhorn Creek Monitoring Station
Chatham County (Near Corinth)
USGS Station Identification #: 02102192
Hydrologic Unit Code: 3030004
Latitude: 35°33'35"
Longitude: -78°58'25"
Drainage Area: 76.3 mi.^{2}

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1972	ND	ND	ND	ND	ND	18.9	17.2	9.3	8.1	15.8	147.0	221.0
1973	135.0	344.0	158.0	178.0	35.1	153.0	32.5	15.4	4.8	2.8	4.9	25.9
1974	110.0	176.0	110.0	62.7	181.0	37.4	8.3	95.5	64.6	10.3	12.0	68.8
1975	299.0	188.0	209.0	52.1	24.4	12.7	291.0	8.5	34.4	24.7	48.5	95.3
1976	153.0	82.5	68.4	24.4	20.5	28.6	5.1	3.2	4.3	14.1	13.9	89.8
1977	150.0	40.2	279.0	66.1	8.5	4.9	1.7	3.5	12.4	22.8	46.7	57.5
1978	387.0	64.7	213.0	229.0	143.0	50.7	18.1	12.7	2.4	7.1	12.7	35.5
1979	178.0	270.0	105.0	137.0	165.0	44.1	38.7	10.8	160.0	28.7	224.0	41.1
1980	156.0	74.2	250.0	82.3	20.7	20.3	8.6	2.1	2.0	3.8	2.8	2.7
1981	2.5	8.9	2.7	1.8	2.0	0.7	0.3	1.1	0.9	0.7	1.9	3.0
1982	9.5	9.2	11.0	5.2	2.3	7.4	7.4	3.1	1.1	1.1	3.3	4.4
1983	9.8	215.0	249.0	240.0	73.7	35.0	9.6	1.3	1.4	3.4	7.2	143.0
1984	241.0	223.0	250.0	262.0	107.0	138.0	80.3	58.7	1.9	16.8	15.2	14.0
1985	46.7	187.0	46.3	12.1	4.5	1.9	11.0	24.9	3.9	1.3	37.3	38.5
1986	15.6	31.3	74.8	21.0	14.2	8.0	0.5	199.0	40.2	0.9	2.4	18.8
1987	219.0	159.0	248.0	149.0	44.6	3.2	1.7	1.1	1.8	0.9	2.0	3.8
1988	5.7	24.3	14.9	6.9	2.8	2.8	1.3	0.8	1.1	2.2	7.0	2.2
1989	2.6	46.5	335.0	168.0	184.0	48.3	102.0	28.0	10.6	60.5	48.7	121.0

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Table 2.4.1-201 (Sheet 2 of 2)
Monthly Mean Streamflow Measurements for Buckhorn Creek, North Carolina

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1990	89.7	120.0	104.0	130.0	34.8	14.6	1.0	1.2	1.2	7.3	6.3	7.4
1991	64.8	30.4	78.3	57.7	55.5	21.6	6.6	3.4	1.2	1.0	0.8	1.4
1992	2.5	1.4	1.7	1.1	1.6	42.9	56.3	1.7	1.8	2.1	100.0	105.0
1993	203.0	64.3	269.0	312.0	23.1	1.7	1.1	1.0	1.1	3.2	1.9	8.5
1994	6.3	28.8	150.0	57.8	15.1	1.6	4.6	1.3	1.2	9.7	18.8	20.8
1995	26.0	123.0	165.0	9.6	8.3	93.0	74.2	2.5	3.8	128.0	146.0	31.9
1996	83.0	156.0	101.0	57.0	51.7	18.3	8.9	4.3	335.0	66.5	49.7	85.6
1997	85.3	143.0	76.5	63.5	106.0	26.8	16.4	3.0	1.0	1.0	4.0	12.6
1998	153.0	348.0	421.0	138.0	104.0	12.0	1.2	0.7	1.5	2.6	3.0	2.4
1999	10.4	12.2	21.8	13.7	3.8	1.1	1.1	1.0	189.0	137.0	25.5	45.5
2000	125.0	215.0	55.7	31.3	10.4	8.7	2.9	8.4	6.6	2.8	1.9	2.3
2001	2.1	2.9	42.4	114.0	3.2	98.1	182.0	191.0	9.3	3.9	8.7	8.2
2002	83.3	63.6	7.9	44.4	1.5	0.9	0.6	0.3	0.7	5.4	58.3	154.0
2003	82.5	180.0	376.0	309.0	62.5	62.0	69.3	242.0	50.2	10.4	5.1	50.7
2004	21.3	94.3	73.4	18.7	65.0	26.1	15.7	72.4	98.6	ND	ND	ND
Mean of Monthly Streamflow (cfs)	98.7	116.0	143.0	95.5	49.4	31.7	32.6	30.7	32.1	18.7	33.4	47.6

Note:

ND = no data available for the given time period

Source: Reference 2.4-247

HAR COL 2.4-1

Table 2.4.1-202 (Sheet 1 of 2)
Yearly Maximum Average Daily Streamflow Measurements for Buckhorn Creek, North Carolina

Buckhorn Creek Monitoring Station
Chatham County (Near Corinth)
USGS Station Identification #: 02102192
Hydrologic Unit Code: 3030004
Latitude: 35°33'35"
Longitude: -78°58'25"
Drainage Area: 76.3 mi.^{2}

Year	Date	Streamflow (cfs)
1972	December 16, 1972	1530
1973	February 2, 1973	3130
1974	August 7, 1974	890
1975	July 5, 1975	2150
1976	January 28, 1976	891
1977	March 14, 1977	1680
1978	April 26, 1978	2820
1979	September 6, 1979	1740
1980	March 21, 1980	951
1981	February 20, 1981	58
1982	July 29, 1982	129
1983	April 16, 1983	470
1984	March 29, 1984	781
1985	April 10, 1985	319
1986	August 20, 1986	766
1987	March 1, 1987	889
1988	March 28, 1988	114
1989	March 7, 1989	562
1990	April 1, 1990	328
1991	May 20, 1991	216
1992	November 26, 1992	390
1993	April 6, 1993	770
1994	March 3, 1994	453
1995	February 19, 1995	401
1996	September 6, 1996	1940

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Table 2.4.1-202 (Sheet 2 of 2)
Yearly Maximum Average Daily Streamflow Measurements for Buckhorn Creek, North Carolina

Buckhorn Creek Monitoring Station
Chatham County (Near Corinth)
USGS Station Identification #: 02102192
Hydrologic Unit Code: 3030004
Latitude: 35°33'35"
Longitude: -78°58'25"
Drainage Area: 76.3 $mi.^2$

Year	Date	Streamflow (cfs)
1997	April 29, 1997	480
1998	March 19, 1998	1190
1999	September 30, 1999	913
2000	February 1, 2000	314
2001	August 12, 2001	828
2002	January 25, 2002	347
2003	April 11, 2003	982
2004	May 3, 2004	200
2005	March 18, 2005	284

Source: Reference 2.4-201

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Table 2.4.1-203
Cape Fear River Basin Monitoring Station Summary

USGS Monitoring Station[a]	River or Creek Location	USGS Station Identification	Hydrologic Unit Code	Lat.	Long. (W)	Drainage Area ($mi.^2$)	Distance from Confluence of Buckhorn Creek and Cape Fear River (river feet/mile)	Flood Stage (ft.)	Discharge at Flood Stage[b] (cfs)	Monitoring Period	Historical Water Records					
											High Water			Low Water		
											Date	Stage Height (ft.)	Daily Mean Discharge (cfs)	Date	Stage Height (ft.)	Daily Mean Discharge (cfs)
East Fork Deep River Near High Point, NC	Deep River	02099000	03030003	36° 02' 14"	79° 56' 44"	14.8	201,410 / 38.1(above)	ND	ND	October 1, 1928 - current	September 24, 1947	ND	1670	August 8, 2002	2.15	0.61
Deep River at Ramseur, NC	Deep River	02100500	03030003	35° 43' 35"	79° 39' 20"	349	137,935 / 26.1 (above)	ND	ND	April 1, 1923 - current	September 18, 1945	ND	27,800	November 29, 1941	ND	0.7
Deep River at Moncure, NC	Deep River	02102000	03030003	35° 37' 37"	79° 06' 58"	1434	19,587 / 3.7 (above)	20	39,377	August 1, 1930 - current	September 18, 1945	ND	66,400	October 9, 1954	ND	6
Haw River at Haw River	Haw River	02096500	03030002	36° 05' 14"	79° 21' 58"	606	81,841 / 15.5 (above)	18	9879	October 1, 1928 - current	September 7, 1996	26.61	42,000	September 6, 1930	ND	5
Haw River Near Bynum, NC	Haw River	02096960	03030002	35° 45' 55"	79° 08' 09"	1275	33,011 / 6.25 (above)	11	13,790	September 26, 1973 – current	September 6, 1996	ND	58,000	September 10, 1983	ND	0.18
Buckhorn Creek NR Corinth, NC	Buckhorn Creek	02102192	03030004	35° 33' 35"	78° 58' 25"	76.3	On Buckhorn Creek	ND	ND	June 1, 1972 - current	February 2, 1973	ND	3130	September 2, 1976	ND	0.04
Cape Fear River at Lillington, NC	Cape Fear River	02102500	03030004	35° 24' 22"	78° 48' 48"	3464	72,575 / 13.7 (below)	14	30,000[a]	January 1, 1924 - current	September 19, 1945	ND	140,000	October 14, 1954	ND	11
Cape Fear River at Fayetteville, NC	Cape Fear River	02104000	03030004	35° 03' 02"	78° 51' 30"	4395	264,840 / 50.2(below)	35	30,300	January 1, 1889 - current[c]	ND	ND	ND	ND	ND	ND
Cape Fear River R at Wilm O Huske Lock NR Tarheel, NC	Cape Fear River	02105500	03030005	35° 50' 45"	78° 49' 14"	4852	369,865 / 70.1 (below)	42	17,000	October 1, 1937 – current	September 21, 1945	ND	112,000	August 13, 1999	0.36	154
Cape Fear R at Lock #1 NR Kelly, NC	Cape Fear River	02105769	03030005	34° 24' 16"	78° 17' 37"	5255	666,983 / 126.3 (below)	24	45,830	July 1, 1969 - current	March 3, 1979	ND	57,100	August 10, 2002	14.00	179

Note:
ND = no data recorded for parameter

Sources:
a) Reference 2.4-249
b) Reference 2.4-250
c) Reference 2.4-249

HAR COL 2.4-1

Table 2.4.1-204 (Sheet 1 of 3)
Monthly Mean Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

Cape Fear River Monitoring Station
Harnett County
USGS Station Identification #: 02102500
Hydrologic Unit Code: 3030004
Latitude: 35°24'22"
Longitude: -78°48'48"
Drainage Area: 3464.0 mi.^{2}

	Monthly Mean Streamflow, in cfs											
Year	**January**	**February**	**March**	**April**	**May**	**June**	**July**	**August**	**September**	**October**	**November**	**December**
1924	4,222	5,896	4,322	4,765	3,565	1,785	4,825	2,574	4,525	4,510	1,409	2,450
1925	14,660	4,303	3,283	1,624	1,721	542	614	517	332	153	530	746
1926	2,833	8,058	4,961	4,185	550	730	1,705	912	287	89.8	300	2,200
1927	1,331	4,354	6,264	1,932	753	1,161	3,503	2,523	1,084	3,788	1,234	7,433
1928	1,342	3,816	2,743	8,142	4,420	2,842	1,947	5,770	22,970	2,410	958	928
1929	1,406	6,969	16,550	4,582	4,175	3,966	4,559	2,539	1,092	13,640	5,677	4,717
1930	4,591	5,380	2,416	2,097	1,054	1,648	1,064	405	166	101	396	2,007
1931	2,584	1,165	2,145	5,464	5,242	812	1,435	7,355	567	195	225	2,294
1932	7,201	4,100	6,761	2,451	1,463	3,081	441	553	360	3,183	3,891	9,350
1933	5,280	5,623	2,782	3,393	1,129	532	333	1,260	803	132	141	240
1934	458	1,280	4,410	6,137	1,811	5,201	2,594	1,637	5,154	1,061	2,032	5,639
1935	5,268	4,386	6,726	7,553	3,022	809	1,186	304	3,025	321	1,801	1,942
1936	14,940	11,650	9,427	13,730	877	3,138	2,150	2,685	782	3,750	1,065	6,382
1937	14,750	6,676	3,963	5,959	2,059	996	1,124	3,882	2,647	1,540	1,237	1,259
1938	3,451	1,603	2,617	3,171	1,342	3,104	7,063	1,386	778	347	1,753	3,136
1939	3,412	14,000	8,414	3,417	2,943	1,206	2,509	8,709	1,068	503	642	1,078
1940	2,147	6,787	4,002	3,862	1,593	1,723	720	3,817	632	167	3,289	1,809
1941	2,746	1,961	4,209	5,688	699	1,308	3,569	456	371	118	107	622
1942	569	3,320	5,829	1,539	2,935	2,387	723	2,039	2,063	1,300	1,865	4,203
1943	7,899	5,117	6,584	4,989	1,361	2,136	6,660	655	852	210	389	937
1944	5,384	7,889	11,250	7,976	2,373	507	3,978	1,805	1,610	6,402	2,036	3,247
1945	3,569	8,238	3,845	2,123	1,535	448	3,086	1,902	21,630	1,971	1,175	8,318
1946	7,529	10,410	2,751	2,809	5,110	2,652	3,965	3,279	1,107	2,108	2,143	1,707
1947	9,667	2,100	4,607	4,291	943	585	685	514	3,261	1,610	6,874	2,044
1948	4,023	13,070	6,719	5,426	1,872	1,825	944	1,654	509	2,126	9,188	8,583
1949	5,656	6,730	3,518	3,562	5,815	1,018	3,196	5,939	2,576	3,302	4,748	2,257

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Table 2.4.1-204 (Sheet 2 of 3)
Monthly Mean Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

	Monthly Mean Streamflow, in cfs											
Year	January	February	March	April	May	June	July	August	September	October	November	December
1950	3,000	2,504	4,092	1,507	4,838	1,525	4,525	915	777	899	629	2,181
1951	1,440	2,025	3,209	5,891	922	1,249	592	743	170	115	353	3,274
1952	4,412	6,246	15,020	3,934	2,427	990	512	2,713	6,583	604	4,291	2,595
1953	8,533	9,656	9,353	3,980	1,681	1,689	522	254	525	164	175	1,804
1954	10,600	2,819	4,957	4,209	1,906	690	373	279	150	4,872	1,002	3,482
1955	1,935	6,768	3,340	3,750	1,056	478	1,419	4,684	4,690	2,067	1,018	671
1956	688	6,705	5,787	3,775	2,259	1,079	2,055	666	2,042	3,492	1,796	3,916
1957	1,818	9,604	6,793	3,040	1,716	3,536	1,401	2,244	2,006	2,373	7,981	5,577
1958	8,434	6,533	5,504	9,901	8,605	1,275	1,580	1,085	300	691	540	2,634
1959	3,101	5,552	3,469	10,650	1,721	2,140	3,793	1,913	2,911	6,529	3,050	3,597
1960	5,108	17,170	10,140	9,480	2,861	995	1,066	2,092	1,046	859	567	921
1961	1,634	9,985	6,249	6,706	3,613	2,075	1,211	2,399	444	179	312	2,579
1962	10,010	6,747	6,968	8,709	960	3,693	2,189	870	590	494	4,655	4,391
1963	5,358	5,958	10,030	1,776	1,427	990	662	430	471	379	2,078	2,937
1964	6,784	7,660	5,099	6,375	1,005	750	1,007	2,243	3,909	7,518	1,263	5,122
1965	2,838	7,521	9,565	3,036	1,454	4,447	8,841	2,329	1,005	1,212	667	545
1966	2,068	8,933	8,732	1,582	3,549	1,102	424	969	878	494	577	1,154
1967	1,697	5,311	1,688	1,195	1,600	592	543	4,380	872	394	441	5,197
1968	7,549	1,630	4,635	1,597	1,607	1,234	989	322	96	535	1,480	1,535
1969	2,643	6,425	7,478	4,064	1,224	2,737	1,232	2,964	1,933	2,106	546	2,345
1970	1,949	6,873	4,448	5,480	2,181	571	701	2,887	376	552	2,424	1,613
1971	5,471	8,964	7,252	3,857	5,106	1,168	728	3,055	1,242	9,412	2,907	3,037
1972	4,093	8,188	2,612	3,594	5,190	4,708	1,297	1,313	662	1,759	5,669	10,470
1973	6,303	10,900	8,826	11,440	3,045	6,062	3,509	1,605	432	268	313	2,411
1974	6,403	6,929	3,407	4,122	4,837	1,762	836	2,272	4,258	587	674	4,034
1975	12,300	6,919	13,790	4,119	3,179	1,842	12,220	1,244	4,359	2,030	2,402	2,774
1976	5,712	3,774	2,468	1,433	1,464	3,610	495	295	221	1,549	760	4,947
1977	4,696	1,708	8,301	3,739	700	466	253	385	2,171	2,042	1,608	3,007
1978	15,350	4,069	7,639	5,155	10,630	2,458	1,833	2,211	832	416	603	2,564
1979	9,502	11,170	10,600	5,970	4,479	3,845	1,003	612	6,015	1,961	6,283	1,779

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Table 2.4.1-204 (Sheet 3 of 3)
Monthly Mean Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

	Monthly Mean Streamflow, in cfs											
Year	January	February	March	April	May	June	July	August	September	October	November	December
1980	6,387	3,426	10,110	4,220	2,349	2,114	1,349	325	368	531	822	820
1981	754	4,445	1,613	1,134	647	852	2,505	1,956	1,603	1,356	655	2,458
1982	6,865	7,398	6,934	3,222	3,641	12,509	2,288	1,864	691	980	1,001	4,408
1983	3,342	10,110	12,170	8,917	3,482	1,502	879	634	653	669	1,388	8,595
1984	10,060	11,570	13,350	11,010	3,583	2,683	5,351	5,082	723	705	711	954
1985	3,304	10,010	2,211	969	1,324	884	1,319	5,447	1,135	792	7,919	4,376
1986	1,373	1,860	4,016	985	824	702	654	1,796	696	640	683	1,408
1987	6,720	5,891	11,300	8,246	1,686	824	817	726	938	726	778	1,148
1988	4,390	2,927	1,628	2,138	1,055	734	680	831	864	1,281	2,147	885
1989	1,552	5,683	15,160	6,253	7,784	2,992	3,454	2,590	1,318	6,442	2,567	5,637
1990	5,679	9,055	4,456	5,619	5,669	1,831	658	643	596	5,393	1,337	3,333
1991	10,470	2,117	6,772	5,433	2,014	1,288	1,333	1,097	905	728	846	882
1992	2,962	2,535	4,083	4,392	1,198	3,686	1,428	1,085	643	971	4,377	3,713
1993	9,110	4,210	15,709	11,670	1,643	864	707	728	671	637	724	1,312
1994	3,399	5,494	8,369	4,778	983	1,045	1,091	1,530	1,036	791	653	723
1995	5,073	8,561	5,800	1,006	937	5,782	5,694	1,496	1,344	5,931	7,377	2,623
1996	6,982	6,425	5,250	3,185	2,770	1,350	827	1,744	13,919	3,829	1,765	4,220
1997	4,730	6,388	5,001	4,061	5,716	1,515	3,483	889	774	685	1,194	2,070
1998	11,750	16,440	14,900	6,373	3,356	986	607	645	673	621	522	812
1999	4,488	3,176	2,882	2,029	2,141	551	604	673	8,181	5,431	1,330	2,061
2000	3,922	6,843	3,859	3,648	1,286	622	1,531	1,080	2,186	1,059	713	746
2001	707	1,786	4,521	4,946	670	2,403	1,283	1,213	684	644	592	612
2002	2,805	1,617	1,749	1,547	642	584	360	274	772	4,735	6,127	7,756
2003	2,819	7,135	14,180	14,779	5,516	6,522	4,325	7,075	4,902	1,170	1,277	3,500
2004	1,406	4,614	3,174	1,885	1,947	1,034	1,074	2,687	5,373	ND	ND	ND
Mean of Monthly Streamflows (cfs)	5,178	6,294	6,441	4,782	2,597	1,997	2,058	1,970	2,270	1,979	1,996	3,046

Note:
ND = no data available for the given time period

Source: Reference 2.4-258

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Table 2.4.1-205 (Sheet 1 of 2)
Yearly Maximum Average Daily Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

Cape Fear River Monitoring Station
Harnett County
USGS Station Identification #: 02102500
Hydrologic Unit Code: 3030004
Latitude: 35°24'22"
Longitude: -78°48'48"
Drainage Area: 3,464.0 mi.^{2}

Year	Date	Streamflow (cfs)
1924	October 1, 1924	49,200
1925	January 12, 1925	43,700
1926	February 4, 1926	25,800
1927	December 5, 1927	37,300
1928	September 20, 1928	83,300
1929	October 3, 1929	96,100
1930	February 5, 1930	24,000
1931	April 7, 1931	24,800
1932	March 7, 1932	47,700
1933	January 10, 1933	13,200
1934	December 2, 1934	37,300
1935	September 6, 1935	30,100
1936	April 8, 1936	62,900
1937	January 21, 1937	32,400
1938	July 27, 1938	38,300
1939	August 29, 1939	42,400
1940	February 8, 1940	28,100
1941	April 6, 1941	23,600
1942	February 18, 1942	25,900
1943	January 19, 1943	36,200
1944	October 1, 1944	56,300
1945	September 19, 1945	140,000
1946	February 11, 1946	48,900
1947	January 21, 1947	34,400
1948	February 15, 1948	47,000
1949	August 29, 1949	37,400
1950	May 15, 1950	28,600
1951	April 9, 1951	32,400
1952	March 5, 1952	71,500
1953	February 16, 1953	40,900
1954	January 23, 1954	52,600
1955	September 4, 1955	39,400
1956	March 17, 1956	42,800
1957	November 26, 1957	44,600
1958	April 30, 1958	35,000
1959	April 13, 1959	34,600

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Table 2.4.1-205 (Sheet 2 of 2)
Yearly Maximum Average Daily Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

Year	Date	Streamflow (cfs)
1960	April 6, 1960	44,600
1961	February 24, 1961	31,600
1962	January 8, 1962	48,600
1963	March 7, 1963	40,100
1964	October 6, 1964	44,600
1965	July 28, 1965	39,500
1966	March 1, 1966	42,800
1967	December 29, 1967	25,100
1968	January 15, 1968	32,200
1969	February 3, 1969	29,300
1970	February 18, 1970	32,000
1971	March 4, 1971	40,800
1972	December 16, 1972	40,700
1973	February 3, 1973	50,400
1974	September 8, 1974	22,000
1975	July 16, 1975	45,000
1976	January 28, 1976	17,500
1977	January 11, 1977	23,400
1978	April 27, 1978	33,700
1979	February 26, 1979	46,600
1980	March 22, 1980	25,800
1981	February 12, 1981	20,200
1982	June 11, 1982	31,300
1983	March 19, 1983	28,200
1984	March 29, 1984	29,600
1985	November 22, 1985	25,700
1986	August 21, 1986	13,500
1987	March 1, 1987	33,500
1988	November 2, 1988	14,400
1989	March 24, 1989	24,200
1990	October 25, 1990	21,600
1991	January 12, 1991	24,600
1992	February 5, 1982	17,100
1993	April 6, 1993	29,900
1994	March 3, 1994	27,800
1995	February 17, 1995	26,800
1996	September 6, 1996	41,400
1997	July 25, 1997	23,600
1998	March 20, 1998	38,000
1999	September 16, 1999	25,000
2000	February 1, 2000	13,800
2001	April 5, 2001	16,000
2002	October 12, 2002	16,800
2003	April 11, 2003	38,300
2004	September 9, 2004	16,200
2005	March 18, 2005	14,300

Source: Reference 2.4-271

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Table 2.4.1-206 (Sheet 1 of 4)
Public Water Supply Users within 10 Miles of the HAR Site

Surface Water Users (S):	**5**
Groundwater Users (G):	**83**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Community	S	353010	S01	Cape Fear River	Sanford	NC	27330	Lee
Community	S	392020	S01	Jordan Lake	Apex	NC	27502	Wake
Non-Transient, Non-Community	S	392992	S01	Harris Lake	New Hill	NC	27562	Wake
Non-Transient, Non-Community	S	392992	S02	Harris Lake Aux Reserv	New Hill	NC	27562	Wake
Non-Transient, Non-Community	S	319414	S01	Haw River	Moncure	NC	27559	Chatham
Transient, Non-Community	G	392454	W01	Well #1	Holly Springs	NC	27540	Wake
Transient, Non-Community	G	392660	W01	Well #1	New Hill	NC	27562	Wake
Transient, Non-Community	G	392669	W01	Well #1	New Hill	NC	27562	Wake
Transient, Non-Community	G	4392439	W01	Well #1	Fuquay Varina	NC	27526	Wake
Transient, Non-Community	G	4392505	W01	Well #1	Apex	NC	27502	Wake
Transient, Non-Community	G	4392529	W01	Well #1	Cary	NC	27519	Wake
Transient, Non-Community	G	4392622	W01	Well #1	Apex	NC	27502	Wake
Transient, Non-Community	G	319457	S01	Well #1	Apex	NC	27502	Chatham
Transient, Non-Community	G	319470	S01	Well #1	Moncure	NC	27559	Chatham
Transient, Non-Community	G	343510	W01	Well #1	Fuquay Varina	NC	27526	Harnett
Transient, Non-Community	G	319428	S01	Well #1	Apex	NC	27502	Chatham
Transient, Non-Community	G	319428	S02	Well #2	Apex	NC	27502	Chatham
Transient, Non-Community	G	319456	W01	Well #1	Apex	NC	27502	Chatham
Transient, Non-Community	G	319479	W01	Well #1	New Hill	NC	27562	Chatham
Transient, Non-Community	G	4392446	W01	Well #1	Fuquay Varina	NC	27526	Wake
Transient, Non-Community	G	392448	W01	Well #1	Apex	NC	27502	Wake
Transient, Non-Community	G	319462	S01	Well #1	New Hill	NC	27562	Chatham
Transient, Non-Community	G	4392458	W01	Well #1	New Hill	NC	27562	Wake
Transient, Non-Community	G	319433	S01	Well #1	Pittsboro	NC	27312	Chatham
Transient, Non-Community	G	4392521	W01	Well #2	Raleigh	NC	27602	Wake

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Table 2.4.1-206 (Sheet 2 of 4)
Public Water Supply Users within 10 Miles of the HAR Site

Surface Water Users (S): 5
Groundwater Users (G): 83

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Transient, Non-Community	G	4392628	W01	Well #1	Fuquay Varina	NC	27526	Wake
Transient, Non-Community	G	392672	W01	Well #1	Apex	NC	27502	Wake
Transient, Non-Community	G	4392520	W01	Well #1	Raleigh	NC	27602	Wake
Transient, Non-Community	G	319424	W01	Well #1	Apex	NC	27502	Chatham
Transient, Non-Community	G	319463	S01	Well #1	Moncure	NC	27559	Chatham
Transient, Non-Community	G	392453	W01	Well #1	Fuquay Varina	NC	27526	Wake
Campground	G	319425	PC8	Well #8	Pittsboro	NC	27502	Chatham
Campground	G	319425	PC6	Well #6	Pittsboro	NC	27502	Chatham
Campground	G	319426	S01	Well #1	Pittsboro	NC	27502	Chatham
Campground	G	319427	W15	Well #15	Apex	NC	27502	Chatham
Campground	G	319427	W17	Well #17	Apex	NC	27502	Chatham
Campground	G	319427	W20	Well #20	Apex	NC	27502	Chatham
Campground	G	319427	W22	Well #22	Apex	NC	27502	Chatham
Campground	G	319427	W23	Well #23	Apex	NC	27502	Chatham
Campground	G	319138	W32	Well #32	Apex	NC	27502	Chatham
Campground	G	319138	W33	Well #33	Apex	NC	27502	Chatham
Community	G	392078	LS1	Well #1	Fuquay-Varina	NC	27519	Wake
Community	G	392078	LS2	Well #2	Fuquay-Varina	NC	27519	Wake
Community	G	392080	SR1	Well #1	Fuquay Varina	NC	27526	Wake
Community	G	392080	SR2	Well #2	Fuquay Varina	NC	27526	Wake
Community	G	392080	SR3	Well #3	Fuquay Varina	NC	27526	Wake
Community	G	392080	SF1	Sunset Forest Well #4	Fuquay Varina	NC	27526	Wake

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Table 2.4.1-206 (Sheet 3 of 4)
Public Water Supply Users within 10 Miles of the HAR Site

Surface Water Users (S):	**5**
Groundwater Users (G):	**83**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Community	G	392080	ST2	Stansted Well #2	Fuquay Varina	NC	27526	Wake
Community	G	392092	W01	Well #1	Fuquay-Varina	NC	27526	Wake
Community	G	392092	W02	Well #2	Fuquay-Varina	NC	27526	Wake
Community	G	392217	NG1	Northgate Well #1	Fuquay Varina	NC	27526	Wake
Community	G	392129	001	Well #1	Holly Springs	NC	27607	Wake
Community	G	392129	002	Well #2	Holly Springs	NC	27607	Wake
Community	G	392129	003	Well #3	Holly Springs	NC	27607	Wake
Community	G	392224	W01	Well #1	Raleigh	NC	27603	Wake
Community	G	392271	W02	Well #2	New Hill	NC	27562	Wake
Community	G	392271	W01	Well #1	New Hill	NC	27562	Wake
Community	G	392322	001	Well #1	Holly Springs	NC	27540	Wake
Community	G	392330	001	Well #1	Apex	NC	27502	Wake
Community	G	392361	HA1	Hallmark #1	Cary	NC	27519	Wake
Community	G	392361	HA2	Hallmark #2	Cary	NC	27519	Wake
Community	G	392361	HO3	Hollybrook #3	Cary	NC	27519	Wake
Community	G	392361	HO4	Hollybrook #4	Cary	NC	27519	Wake
Community	G	392383	BR3	Briarwood #3	Raleigh	NC	27606	Wake
Community	G	392383	BR4	Briarwood #4	Raleigh	NC	27606	Wake
Community	G	392399	W02	Well #2	Apex	NC	27607	Wake
Community	G	392383	KI1	Kildaire #1	Raleigh	NC	27606	Wake
Community	G	392383	KI2	Kildaire #2	Raleigh	NC	27606	Wake
Community	G	392383	BR2	Briarwood #2	Raleigh	NC	27606	Wake
Community	G	392395	W01	Well #1	Fuquay-Varina	NC	27526	Wake
Community	G	392395	W02	Well #2	Fuquay-Varina	NC	27526	Wake

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Table 2.4.1-206 (Sheet 4 of 4)
Public Water Supply Users within 10 Miles of the HAR Site

Surface Water Users (S):	**5**
Groundwater Users (G):	**83**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Community	G	392399	W01	Well #1	Apex	NC	27607	Wake
Community	G	4092040	HD1	Well #3	Holly Springs	NC	27502	Wake
Community	G	4092040	HD2	Well #4	Holly Springs	NC	27502	Wake
Community	G	4392114	OL1	Well #1 Olde Mills Lake	Holly Springs	NC	27540	Wake
Community	G	4392164	BP2	Well #2	Holly Springs	NC	27540	Wake
Community	G	4392164	BP1	Well #1	Holly Springs	NC	27540	Wake
Community	G	4392229	HG1	Well #1	Cary	NC	27502	Wake
Community	G	4092001	CH3	Well #3	Holly Springs	NC	27502	Wake
Community	G	4092001	CH2	Well #2	Holly Springs	NC	27502	Wake
Community	G	4092001	CH1	Well #1	Holly Springs	NC	27502	Wake
Community	G	4092005	ME5	Well #5	Apex	NC	27502	Wake
Community	G	4092005	ME2	Well #2	Apex	NC	27502	Wake
Community	G	319124	W01	Well #1	Apex	NC	27502	Chatham
Non-Transient, Non-Community	G	4392434	W01	Well #1	Holly Springs	NC	27540	Wake
Non-Transient, Non-Community	G	319490	A01	Well A	Apex	NC	27502	Chatham
Non-Transient, Non-Community	G	319490	B01	Well B	Apex	NC	27502	Chatham
Non-Transient, Non-Community	G	319490	C01	Well C	Apex	NC	27502	Chatham

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Table 2.4.1-207 (Sheet 1 of 3)
Public Water Supply Users within 10 Miles of the HAR Site by Water Type

Surface Water Type:	**4**
Groundwater Under the Direct Influence of Surface Water Type:	**0**
Groundwater Type:	**54**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Surface	Sanford WTP	Treatment Plant	Permanent	0353010	33,503	Sanford	NC	27330
Community	Surface	Chatham Co WTP	Treatment Plant	Permanent	0319126	7947	Pittsboro	NC	27312
Non-Transient, Non-Community	Surface	Harris WTP	Treatment Plant	Permanent	0392992	1200	New Hill	NC	27562
Non-Transient, Non-Community	Surface	Allied Sig WTP	Treatment Plant	Permanent	0319414	700	Moncure	NC	27559
Campground	Ground	Well #23	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #20	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #22	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #15	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #17	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #1	Well House	Permanent	0319426	241	Apex	NC	27502
Campground	Ground	Storage Fac	Storage	Permanent	0319425	264	Apex	NC	27502
Campground	Ground	Central Trtmt Pl	Treatment Plant	Permanent	0319138	470	Apex	NC	27502
Community	Ground	Well #1	Well House	Permanent	4392229	1	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392164	128	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392164	128	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392114	96	Cary	NC	27519
Community	Ground	Well #5	Well House	Permanent	4092005	84	Cary	NC	27519

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Table 2.4.1-207 (Sheet 2 of 3)
Public Water Supply Users within 10 Miles of the HAR Site by Water Type

Surface Water Type:	**4**
Groundwater Under the Direct Influence of Surface Water Type:	**0**
Groundwater Type:	**54**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Well #1	Well House	Permanent	4092001	25	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4092001	25	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392399	107	Raleigh	NC	27607
Community	Ground	Plant #1	Well House	Permanent	0392395	157	Raleigh	NC	27606
Community	Ground	Plant #1	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #5	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #4	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Hallmark #1	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #4	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #3	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #2	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hollybrook #2	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hollybrook #1	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392330	107	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392322	76	Cary	NC	27519

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Table 2.4.1-207 (Sheet 3 of 3)
Public Water Supply Users within 10 Miles of the HAR Site by Water Type

Surface Water Type:	**4**
Groundwater Under the Direct Influence of Surface Water Type:	**0**
Groundwater Type:	**54**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Plant #1	Well House	Permanent	0392271	51	New Hill	NC	27562
Community	Ground	Plant #2	Well House	Permanent	0392271	51	New Hill	NC	27562
Community	Ground	Plant #1	Well House	Permanent	0392224	60	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392217	81	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Well #3	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Well #2	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Plant #2	Well House	Permanent	0392092	172	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392092	172	Raleigh	NC	27603
Community	Ground	Well #3	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392078	42	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392078	42	Cary	NC	27519
Community	Ground	Plant #1	Treatment Plant	Permanent	0319125	70	Chapel Hill	NC	27514
Community	Ground	Plant #2	Well House	Permanent	0319124	70	Apex	NC	27502
Community	Ground	Plant #1	Well House	Permanent	0319124	70	Apex	NC	27502
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392434	200	Raleigh	NC	27606
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392448	25	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319492	70	Siler City	NC	27344
Transient, Non-Community	Ground	Well #1	Pumping Facility	Permanent	0319429	30	Apex	NC	27502
Transient, Non-Community	Ground	Plant #1	Well House	Permanent	0319428	75	Apex	NC	27502
Transient, Non-Community	Ground	Well	Well House	Permanent	0319424	100	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319420	100	Apex	NC	27502

Source: Reference 2.4-208

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Table 2.4.1-208 (Sheet 1 of 3)
Public Water Supply Users within 25 Miles of the HAR Site

Surface Water Users (S):	**7**
Groundwater Users (G):	**105**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Community	S	343045	S01	Cape Fear River	Lillington	NC	27546	Harnett
Community	S	353010	S01	Cape Fear River	Sanford	NC	27330	Lee
Community	S	392020	S01	Jordan Lake	Apex	NC	27502	Wake
Non-Transient, Non-Community	S	353130	S01	Deep River	Sanford	NC	27330	Lee
Non-Transient, Non-Community	S	392992	S01	Harris Lake	New Hill	NC	27562	Wake
Non-Transient, Non-Community	S	392992	S02	Harris Lake Aux Reserv	New Hill	NC	27562	Wake
Non-Transient, Non-Community	S	319414	S01	Haw River	Moncure	NC	27559	Chatham
Non-Transient, Non-Community	G	332590	W01	Well #1	Durham	NC	27703	Durham
Non-Transient, Non-Community	G	368469	W01	Well #1	Chapel Hill	NC	27516	Orange
Non-Transient, Non-Community	G	392272	W01	Well #1	Apex	NC	27539	Wake
Non-Transient, Non-Community	G	392720	W03	Well #3	Raleigh	NC	27628	Wake
Non-Transient, Non-Community	G	392720	W05	Well #5	Raleigh	NC	27628	Wake
Non-Transient, Non-Community	G	392720	W06	Well #6	Raleigh	NC	27628	Wake
Non-Transient, Non-Community	G	392720	W01	Well #1	Raleigh	NC	27628	Wake
Non-Transient, Non-Community	G	392720	W02	Well #2	Raleigh	NC	27628	Wake
Non-Transient, Non-Community	G	392774	S01	Well #1	Fuquay Varina	NC	27526	Wake
Non-Transient, Non-Community	G	392880	W01	Well #1	Raleigh	NC	27613	Wake
Non-Transient, Non-Community	G	392880	W02	Well #2	Raleigh	NC	27613	Wake
Non-Transient, Non-Community	G	392892	W01	Well #1	Raleigh	NC	27612	Wake
Non-Transient, Non-Community	G	392980	W01	Well #1	Raleigh	NC	27611	Wake
Non-Transient, Non-Community	G	4392404	001	Well #1	Apex	NC	27505	Wake
Non-Transient, Non-Community	G	4392409	S01	Well #1	Raleigh	NC	27606	Wake
Non-Transient, Non-Community	G	4392412	W01	Well #1	Garner	NC	27529	Wake
Non-Transient, Non-Community	G	4392420	W01	Well #1	Garner	NC	27529	Wake
Non-Transient, Non-Community	G	4392431	W01	Well #1	Raleigh	NC	27617	Wake
Transient, Non-Community	G	351925	S01	Well #1	Angier	NC	27501	Johnston
Transient, Non-Community	G	353428	W01	Well #1	Sanford	NC	27330	Lee
Transient, Non-Community	G	368423	S01	Well #1	Chapel Hill	NC	27516	Orange
Transient, Non-Community	G	368444	S01	Well #1	Durham	NC	27707	Orange
Transient, Non-Community	G	368459	S01	Well #1	Chapel Hill	NC	27516	Orange
Transient, Non-Community	G	368480	W01	Well #1	Chapel Hill	NC	27516	Orange
Transient, Non-Community	G	392416	W01	Well #1	Apex	NC	27502	Wake
Transient, Non-Community	G	392420	W01	Well #1	Morrisville	NC	27560	Wake
Transient, Non-Community	G	392427	W01	Well #1	Durham	NC	27713	Wake
Transient, Non-Community	G	392454	W01	Well #1	Holly Springs	NC	27540	Wake
Transient, Non-Community	G	392460	W01	Well #1	Raleigh	NC	27606	Wake
Transient, Non-Community	G	392609	001	Well #1	Raleigh	NC	27603	Wake
Transient, Non-Community	G	392614	W01	Well #1	Raleigh	NC	27603	Wake
Transient, Non-Community	G	392660	W01	Well #1	New Hill	NC	27562	Wake
Transient, Non-Community	G	392669	W01	Well #1	New Hill	NC	27562	Wake

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Table 2.4.1-208 (Sheet 2 of 3)
Public Water Supply Users within 25 Miles of the HAR Site

Surface Water Users (S):	**7**
Groundwater Users (G):	**105**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Transient, Non-Community	G	392810	W01	Well #1	Raleigh	NC	27613	Wake
Transient, Non-Community	G	392587	W01	Well #1	Willow Springs	NC	27592	Wake
Transient, Non-Community	G	392591	W01	Well #1	Willow Springs	NC	27592	Wake
Transient, Non-Community	G	392597	W01	Well #1	Willow Springs	NC	27592	Wake
Transient, Non-Community	G	392597	W02	Well #2	Willow Springs	NC	27592	Wake
Transient, Non-Community	G	392598	W01	Well #1	Willow Springs	NC	27592	Wake
Transient, Non-Community	G	392759	W01	Well #1	Raleigh	NC	27606	Wake
Transient, Non-Community	G	392786	W01	Well #1	Raleigh	NC	27603	Wake
Transient, Non-Community	G	392787	W01	Well #1	Cary	NC	27511	Wake
Transient, Non-Community	G	392806	W01	Well #1	Garner	NC	27529	Wake
Transient, Non-Community	G	392924	W01	Well #1	Raleigh	NC	27603	Wake
Transient, Non-Community	G	392951	S02	Well #2	Garner	NC	27610	Wake
Campground	G	319138	W33	Well #33	Apex	NC	27502	Chatham
Campground	G	319427	W15	Well #15	Apex	NC	27502	Chatham
Campground	G	319427	W17	Well #17	Apex	NC	27502	Chatham
Campground	G	319138	W32	Well #32	Apex	NC	27502	Chatham
Campground	G	319427	W20	Well #20	Apex	NC	27502	Chatham
Campground	G	319427	W22	Well #22	Apex	NC	27502	Chatham
Campground	G	319427	W23	Well #23	Apex	NC	27502	Chatham
Campground	G	319425	PC6	Well #6	Pittsboro	NC	27502	Chatham
Campground	G	319425	PC8	Well #8	Pittsboro	NC	27502	Chatham
Campground	G	319426	S01	Well #1	Pittsboro	NC	27502	Chatham
Community	G	332130	S01	Well #1	Durham	NC	27705	Durham
Community	G	332130	S02	Well #2	Durham	NC	27705	Durham
Community	G	332441	S02	Well #2	Durham	NC	27713	Durham
Community	G	351150	W01	Well #1	Willow Springs	NC	27592	Johnston
Community	G	351154	W01	Well #1	Clayton	NC	27629	Johnston
Community	G	351154	W02	Well #2	Clayton	NC	27629	Johnston
Community	G	351156	W01	Well #1	Garner	NC	27529	Johnston
Community	G	351156	W02	Well #2	Garner	NC	27529	Johnston
Community	G	351161	00W	Well #1	Garner	NC	27529	Johnston
Community	G	351164	W01	Well #1	Willow Springs	NC	27592	Johnston
Community	G	351186	CS2	Well #2	Clayton	NC	27520	Johnston
Community	G	351186	CS3	Well #3	Clayton	NC	27520	Johnston
Community	G	351190	IX2	Well #2	Clayton	NC	27520	Johnston
Community	G	351413	W01	Well #1	Garner	NC	27529	Johnston
Community	G	332106	W01	Well #1	Durham	NC	27703	Durham
Community	G	332106	W02	Well #2	Durham	NC	27703	Durham
Community	G	332106	W03	Well #3	Durham	NC	27703	Durham
Community	G	332106	W04	Well #4	Durham	NC	27703	Durham
Community	G	343030	W01	Well #1	Buies Creek	NC	27506	Harnett

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Table 2.4.1-208 (Sheet 3 of 3)
Public Water Supply Users within 25 Miles of the HAR Site

Surface Water Users (S):	**7**
Groundwater Users (G):	**105**

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County
Community	G	343030	W02	Well #2-Tennis Court	Buies Creek	NC	27506	Harnett
Community	G	343030	W03	Well #3-Campbelltown	Buies Creek	NC	27506	Harnett
Community	G	343030	W07	Well #7-Horse Barn	Buies Creek	NC	27506	Harnett
Community	G	343030	W08	Well #8-Golf Course	Buies Creek	NC	27506	Harnett
Community	G	343030	W09	Well #9-Pool	Buies Creek	NC	27506	Harnett
Community	G	343030	W10	Well #10-Faculty	Buies Creek	NC	27506	Harnett
Community	G	343030	W11	Well #11-Keith Hills	Buies Creek	NC	27506	Harnett
Community	G	351104	CH1	Well #1	Garner	NC	27519	Johnston
Community	G	351104	CH2	Well #2	Garner	NC	27519	Johnston
Community	G	351104	CH3	Well #3	Garner	NC	27519	Johnston
Community	G	351167	SL1	Well #1	Clayton	NC	27519	Johnston
Community	G	351168	SH2	Southhills #2	Clayton	NC	27520	Johnston
Community	G	351168	SH3	Southhills #3	Clayton	NC	27520	Johnston
Community	G	351168	SH4	Southhills #4	Clayton	NC	27520	Johnston
Community	G	351168	SH5	Pleasant Woods #5	Clayton	NC	27520	Johnston
Community	G	351168	SW1	Southwoods #1	Clayton	NC	27520	Johnston
Community	G	351176	001	Well #1	Garner	NC	27529	Johnston
Community	G	351176	002	Well #2	Garner	NC	27529	Johnston
Community	G	351184	SF1	Well #1	Clayton	NC	27520	Johnston
Community	G	351185	SG1	Southgate #1	Clayton	NC	27520	Johnston
Community	G	351185	SG2	Southgate #2	Clayton	NC	27520	Johnston
Community	G	351186	CS1	Well #1	Clayton	NC	27520	Johnston
Community	G	353015	W01	Well #1	Broadway	NC	27505	Lee
Community	G	353015	W02	Well #2	Broadway	NC	27505	Lee
Community	G	353101	S04	Well #4	Sanford	NC	27330	Lee
Community	G	353101	S16	Well #16	Sanford	NC	27330	Lee
Community	G	353119	W01	Well #1	Sanford	NC	27330	Lee
Community	G	353122	PH1	Well #1	Sanford	NC	27330	Lee
Community	G	353122	PH2	Well #2	Sanford	NC	27330	Lee
Community	G	353123	W01	Well #1	Sanford	NC	27330	Lee
Community	G	353123	W02	Well #2	Sanford	NC	27330	Lee

Source: Reference 2.4-207

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Table 2.4.1-209 (Sheet 1 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	**9**
Groundwater Under the Direct Influence of Surface Water Type:	**3**
Groundwater Type:	**431**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Surface	OWASA WTP	Treatment Plant	Permanent	0368010	65000	Carrboro	NC	27510
Community	Surface	Lee County WTP	Treatment Plant	Permanent	0353130	780	Sanford	NC	27330
Community	Surface	Sanford WTP	Treatment Plant	Permanent	0353010	33503	Sanford	NC	27330
Community	Surface	Harnett Co WTP	Treatment Plant	Permanent	0343045	50000	Lillington	NC	27546
Community	Surface	Chatham Co WTP	Treatment Plant	Permanent	0319126	7947	Pittsboro	NC	27312
Community	Surface	Goldston GLF WTP	Treatment Plant	Permanent	0319025	1250	Goldston	NC	27252
Community	Surface	Pittsboro WTP	Treatment Plant	Permanent	0319015	3048	Pittsboro	NC	27312
Non-Transient, Non-Community	Surface	Harris WTP	Treatment Plant	Permanent	0392992	1200	New Hill	NC	27562
Non-Transient, Non-Community	Surface	Allied SIG WTP	Treatment Plant	Permanent	0319414	700	Moncure	NC	27559
Community	Groundwater Under Direct Influence of Surface Water	Well #1	Well House	Permanent	0368190	53	Greensboro	NC	27425
Community	Groundwater Under Direct Influence of Surface Water	Wells #1 & 1A	Treatment Plant	Permanent	0368182	224	Chapel Hill	NC	27514
Community	Groundwater Under Direct Influence of Surface Water	Storage Bldg	Storage	Permanent	0332130	87	Durham	NC	27704
Campground	Ground	Plant #1	Well House	Permanent	0392089	90	Raleigh	NC	27610
Campground	Ground	Well #23	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #20	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #22	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #15	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #17	Well House	Permanent	0319427	900	Apex	NC	27502
Campground	Ground	Well #1	Well House	Permanent	0319426	241	Apex	NC	27502
Campground	Ground	Storage Fac	Storage	Permanent	0319425	264	Apex	NC	27502
Campground	Ground	Central Trmt Pl	Treatment Plant	Permanent	0319138	470	Apex	NC	27502
Community	Ground	Well #1	Well House	Permanent	4392233	25	Garner	NC	27529
Community	Ground	Well #1	Well House	Permanent	4392229	1	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392225	25	Cary	NC	27519
Community	Ground	Well #8	Well House	Permanent	4392216	96	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	4392216	96	Cary	NC	27519
Community	Ground	Plant #4	Well House	Permanent	4392214	25	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392211	3	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392192	20	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	4392188	115	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392188	115	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392188	115	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392187	25	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392186	25	Fuquay Varina	NC	27526
Community	Ground	Lane Ridge #1	Well House	Permanent	4392183	140	Cary	NC	27519
Community	Ground	Old Stage #1	Well House	Permanent	4392183	140	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	4392181	175	Cary	NC	27519
Community	Ground	Well #5	Well House	Permanent	4392181	175	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392172	125	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392169	110	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392169	110	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	4392169	110	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392164	128	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392164	128	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392163	155	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392163	155	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392160	85	Cary	NC	27519

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Table 2.4.1-209 (Sheet 2 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	**9**
Groundwater Under the Direct Influence of Surface Water Type:	**3**
Groundwater Type:	**431**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Well #2	Well House	Permanent	4392158	112	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	4392157	195	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392157	195	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392157	195	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	4392151	123	Cary	NC	27519
Community	Ground	Holland Ridge	Well House	Permanent	4392150	328	Cary	NC	27519
Community	Ground	Lake Rand Well	Well House	Permanent	4392150	328	Cary	NC	27519
Community	Ground	Greenfield Manr	Well House	Permanent	4392150	328	Cary	NC	27519
Community	Ground	Holland Meadows	Well House	Deactivate	4392150	328	Cary	NC	27519
Community	Ground	Whittingham	Well House	Deactivate	4392150	328	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392147	87	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392145	74	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392142	207	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392142	207	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392141	25	Zebulon	NC	27597
Community	Ground	Plant #3	Well House	Permanent	4392140	350	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392140	350	Cary	NC	27519
Community	Ground	Plant 1	Well House	Permanent	4392140	350	Cary	NC	27519
Community	Ground	Plant #4	Well House	Permanent	4392140	350	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392139	45	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392135	99	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392134	102	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392132	53	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392131	75	Cary	NC	27519
Community	Ground	Plant 2	Well House	Permanent	4392129	122	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392128	211	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	4392128	211	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392128	211	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392124	97	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392124	97	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392122	160	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392122	160	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	4392120	160	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392120	160	Cary	NC	27519
Community	Ground	Well #1 (CRKWD)	Well House	Permanent	4392119	245	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392119	245	Cary	NC	27519
Community	Ground	Plant SW2	Well House	Permanent	4392118	114	Cary	NC	27519
Community	Ground	Plant SW1	Well House	Permanent	4392118	114	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392114	96	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392112	200	Cary	NC	27519
Community	Ground	Plant #1 OFF-L	Treatment Plant	Other	4392112	200	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392111	80	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4392105	89	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392102	241	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	4392102	241	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	4392102	241	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	4392101	143	Cary	NC	27519
Community	Ground	Plant 1	Well House	Permanent	4392101	143	Cary	NC	27519
Community	Ground	Holland Meadows	Well House	Permanent	4092039	493	Cary	NC	27519
Community	Ground	Whittingham	Well House	Permanent	4092039	493	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	4092006	25	Cary	NC	27519
Community	Ground	Well #5	Well House	Permanent	4092005	84	CARY	NC	27519
Community	Ground	Wells #1 & #2	Treatment Plant	Permanent	4092004	48	BENSON	NC	27504

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Table 2.4.1-209 (Sheet 3 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	9
Groundwater Under the Direct Influence of Surface Water Type:	3
Groundwater Type:	431

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Well #1	Well House	Permanent	4092001	25	CARY	NC	27519
Community	Ground	Well #2	Well House	Permanent	4092001	25	CARY	NC	27519
Community	Ground	Well #4	Well House	Permanent	4019004	25	CHAPEL HILL	NC	27514
Community	Ground	Plant #1	Well House	Permanent	0392399	107	RALEIGH	NC	27607
Community	Ground	Well #1	Well House	Permanent	0392398	342	CARY	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392398	342	CARY	NC	27519
Community	Ground	Well #4	Well House	Permanent	0392398	342	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392397	152	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392395	157	Raleigh	NC	27606
Community	Ground	Plant #1	Well House	Permanent	0392391	54	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392390	431	Benson	NC	27504
Community	Ground	Plant #2	Well House	Permanent	0392388	104	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392388	104	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392387	280	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392387	280	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392385	105	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #5	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Plant #4	Well House	Permanent	0392383	419	Cary	NC	27519
Community	Ground	Wedgewood Sqr 1	Well House	Permanent	0392381	70	Cary	NC	27519
Community	Ground	Crooked Brook 1	Well House	Permanent	0392381	70	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392376	435	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392376	435	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392375	115	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392375	115	Cary	NC	27519
Community	Ground	Heritage Point	Well House	Permanent	0392373	8462	Cary	NC	27519
Community	Ground	Shannon Woods 1	Well House	Permanent	0392373	8462	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392370	30	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392369	180	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392369	180	Raleigh	NC	27603
Community	Ground	Plant #4	Well House	Permanent	0392367	37	Garner	NC	27529
Community	Ground	Plant #1	Well House	Permanent	0392366	47	Raleigh	NC	27607
Community	Ground	Plant #1	Well House	Permanent	0392365	86	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392364	49	Raleigh	NC	27607
Community	Ground	Plant #1	Well House	Permanent	0392363	112	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392363	112	Cary	NC	27519
Community	Ground	Hallmark #1	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #4	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #3	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hallmark #2	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hollybrook #2	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Hollybrook #1	Well House	Permanent	0392361	688	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392358	135	CARY	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392358	135	Cary	NC	27519
Community	Ground	Plant #3	Well House	Other	0392357	605	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392357	605	Cary	NC	27519
Community	Ground	Springfield #1	Well House	Permanent	0392357	605	Cary	NC	27519
Community	Ground	Springfield #2	Well House	Permanent	0392357	605	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392357	605	Cary	NC	27519
Community	Ground	Willowbluffs #1	Well House	Permanent	0392355	520	Cary	NC	27519
Community	Ground	Willowbluffs #2	Well House	Permanent	0392355	520	Cary	NC	27519
Community	Ground	Willowbluffs #3	Well House	Permanent	0392355	520	Cary	NC	27519
Community	Ground	Springhaven #1	Well House	Permanent	0392355	520	Cary	NC	27519
Community	Ground	Middle Creek W#1	Well House	Permanent	0392355	520	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392353	235	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392353	235	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392353	235	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392352	34	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392350	116	Raleigh	NC	27607
Community	Ground	Well #1	Well House	Permanent	0392349	62	Garner	NC	27529
Community	Ground	Well #2	Well House	Permanent	0392345	376	Bailey	NC	27807
Community	Ground	Well #1	Well House	Permanent	0392345	376	Bailey	NC	27807
Community	Ground	Plant #1	Well House	Permanent	0392344	98	Raleigh	NC	27610
Community	Ground	Plant #2	Well House	Permanent	0392338	340	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392338	340	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392338	340	Cary	NC	27519

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Table 2.4.1-209 (Sheet 4 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	**9**
Groundwater Under the Direct Influence of Surface Water Type:	**3**
Groundwater Type:	**431**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Plant #1	Well House	Permanent	0392337	84	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392336	49	Cary	NC	27519
Community	Ground	Windhaven W#1	Well House	Permanent	0392335	268	Cary	NC	27519
Community	Ground	Windhaven S W#1	Well House	Permanent	0392335	268	Cary	NC	27519
Community	Ground	Windhaven S W#2	Well House	Permanent	0392335	268	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392332	34	Willow Springs	NC	27592
Community	Ground	Well #5	Well House	Permanent	0392331	599	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	0392331	599	Cary	NC	27519
Community	Ground	Plant #6	Well House	Permanent	0392331	599	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392330	107	Cary	NC	27519
Community	Ground	WELLS #1,2,3,4	Well House	Permanent	0392323	233	Raleigh	NC	27607
Community	Ground	Plant #1	Well House	Permanent	0392322	76	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392321	185	Raleigh	NC	27606
Community	Ground	Plant #1	Well House	Permanent	0392321	185	Raleigh	NC	27606
Community	Ground	Well #1	Well House	Permanent	0392319	92	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392315	50	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392314	93	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392313	59	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392313	59	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392311	32	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392308	82	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392307	40	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392303	105	Zebulon	NC	27597
Community	Ground	Well #1	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Well #6	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Well #7	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Wildwd Grn #1	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Brandon #10/#11	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Wildwd Grn #2	Well House	Permanent	0392298	1799	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392294	96	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392294	96	Cary	NC	27519
Community	Ground	Well #1 (GLENDL)	Well House	Permanent	0392293	584	Cary	NC	27519
Community	Ground	Well #3 (ROLLWD)	Well House	Permanent	0392293	584	Cary	NC	27519
Community	Ground	Well #4 (BRIGHT)	Well House	Permanent	0392293	584	Cary	NC	27519
Community	Ground	Well #5 (PEBBLE)	Well House	Permanent	0392293	584	Cary	NC	27519
Community	Ground	Well #2 (CHARI)	Well House	Permanent	0392293	584	Cary	NC	27519
Community	Ground	Plant #9 WV2	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #27 OC3	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #6 HA6	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #22 BB6	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #19 W13	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #3 HA1&2	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #21 BB1	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #12 WV6	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #4 HA3&4	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #11 WV5	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #7 HA7	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #13 WV7	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #20 HA8	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #14 WV8	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #17 W11	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #15 WV9	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #8 WV1	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #18 W12	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #5 HA5	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #16 W10	Well House	Other	0392291	3559	Cary	NC	27519
Community	Ground	Plant #10 WV4	Well House	Permanent	0392291	3559	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392280	123	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392280	123	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392274	42	Zebulon	NC	27597
Community	Ground	Plant #1	Well House	Permanent	0392271	51	New Hill	NC	27562
Community	Ground	Plant #2	Well House	Permanent	0392271	51	New Hill	NC	27562
Community	Ground	Well #1	Well House	Permanent	0392263	81	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392257	410	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392257	410	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392257	410	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392253	648	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392253	648	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392253	648	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	0392253	648	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392251	280	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392251	280	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392250	430	Garner	NC	27529
Community	Ground	Well #1	Well House	Permanent	0392250	430	Garner	NC	27529
Community	Ground	Well #2	Well House	Permanent	0392250	430	Garner	NC	27529
Community	Ground	Plant #1	Well House	Permanent	0392247	57	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392240	50	Burlington	NC	27216
Community	Ground	Well #1	Well House	Permanent	0392237	25	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392224	60	Raleigh	NC	27603
Community	Ground	Wells #1A &1B	Well House	Permanent	0392219	86	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392219	86	Cary	NC	27519

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Table 2.4.1-209 (Sheet 5 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	**9**
Groundwater Under the Direct Influence of Surface Water Type:	**3**
Groundwater Type:	**431**

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Plant #1	Well House	Permanent	0392217	81	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392213	77	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392212	84	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392206	60	Cary	NC	27511
Community	Ground	Well #1	Well House	Permanent	0392200	75	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392196	32	Raleigh	NC	27607
Community	Ground	Well #1	Well House	Permanent	0392195	60	Zebulon	NC	27597
Community	Ground	Plant #3	Well House	Permanent	0392190	285	Fuquay-Varina	NC	27526
Community	Ground	Plant #2	Well House	Permanent	0392190	285	Fuquay-Varina	NC	27526
Community	Ground	Plant #1	Well House	Permanent	0392190	285	Fuquay-Varina	NC	27526
Community	Ground	Plant #1	Well House	Permanent	0392187	45	Garner	NC	27607
Community	Ground	Plant #1	Well House	Permanent	0392186	67	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392179	177	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392179	177	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392178	120	Willow Springs	NC	27592
Community	Ground	Plant #1	Well House	Permanent	0392177	93	Raleigh	NC	27602
Community	Ground	Plant #1	Well House	Permanent	0392172	69	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392168	298	Charlotte	NC	28224
Community	Ground	Well #1	Well House	Permanent	0392168	298	Charlotte	NC	28224
Community	Ground	Plant #2	Well House	Permanent	0392166	91	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392166	91	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392164	54	Garner	NC	27529
Community	Ground	Plant #1	Well House	Permanent	0392163	166	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392163	166	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392160	731	Cary	NC	27519
Community	Ground	Plant #7	Well House	Permanent	0392160	731	Cary	NC	27519
Community	Ground	Plant #6	Well House	Permanent	0392160	731	Cary	NC	27519
Community	Ground	Plant #4	Well House	Permanent	0392160	731	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392160	731	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392151	125	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392150	58	Holly Springs	NC	27540
Community	Ground	Plant #1	Well House	Permanent	0392149	161	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392147	90	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392145	45	Raleigh	NC	27623
Community	Ground	Plant #1	Well House	Permanent	0392141	53	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Well #3	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Well #2	Well House	Permanent	0392129	325	Raleigh	NC	27607
Community	Ground	Well #1	Well House	Permanent	0392128	120	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392119	101	Cary	NC	27519
Community	Ground	Wells #1 & #2	Well House	Permanent	0392117	99	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392116	270	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392116	270	Cary	NC	27519
Community	Ground	Well #2	Well House	Emergency	0392116	270	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392113	270	Raleigh	NC	27603
Community	Ground	Well #2	Well House	Permanent	0392113	270	Raleigh	NC	27603
Community	Ground	Wells #1 & #2	Well House	Permanent	0392112	75	Raleigh	NC	27612
Community	Ground	Well #1	Well House	Permanent	0392111	600	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392111	600	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392111	600	Cary	NC	27519
Community	Ground	Well #4	Well House	Permanent	0392111	600	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392108	149	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392108	149	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392107	229	Zebulon	NC	27597
Community	Ground	Plant #3	Well House	Permanent	0392107	229	Zebulon	NC	27597
Community	Ground	Plant #3	Well House	Permanent	0392102	536	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Other	0392102	536	Raleigh	NC	27603
Community	Ground	Plant #2	Well House	Permanent	0392102	536	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392099	70	Charlotte	NC	28224
Community	Ground	Plant #1	Well House	Permanent	0392095	298	Charlotte	NC	28224
Community	Ground	Plant #2	Well House	Permanent	0392092	172	Raleigh	NC	27603
Community	Ground	Plant #1	Well House	Permanent	0392092	172	Raleigh	NC	27603
Community	Ground	Well #1	Well House	Permanent	0392091	63	Garner	NC	27529
Community	Ground	Plant #1	Well House	Permanent	0392090	178	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392090	178	Cary	NC	27519
Community	Ground	Plant #3	Well House	Permanent	0392087	280	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392087	280	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392087	280	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0392085	91	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0392085	91	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392080	325	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0392078	42	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0392078	42	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0368192	56	Chapel Hill	NC	27516
Community	Ground	Wells #3 #4 #5	Well House	Permanent	0368188	180	Greensboro	NC	27425
Community	Ground	Well #2	Well House	Deactivate	0368188	180	Greensboro	NC	27425
Community	Ground	Sedgefield #1	Well House	Permanent	0368185	688	Cary	NC	27519
Community	Ground	Stoneridge #4	Well House	Permanent	0368185	688	Cary	NC	27519
Community	Ground	Stoneridge #3	Well House	Other	0368185	688	Cary	NC	27519
Community	Ground	Creekwood #1	Well House	Permanent	0368185	688	Cary	NC	27519
Community	Ground	Stoneridge #1	Well House	Permanent	0368185	688	Cary	NC	27519
Community	Ground	Plant 1	Well House	Permanent	0368184	79	Chapel Hill	NC	27516
Community	Ground	Plant #2	Well House	Permanent	0368179	120	Durham	NC	27704
Community	Ground	Plant #1	Well House	Permanent	0368179	120	Durham	NC	27704
Community	Ground	Plant #1	Treatment Plant	Permanent	0368174	77	Chapel Hill	NC	27514
Community	Ground	Well #1	Well House	Permanent	0368164	33	Greensboro	NC	27425
Community	Ground	Well #3	Treatment Plant	Permanent	0368160	65	Chapel Hill	NC	27516
Community	Ground	Well #1	Well House	Permanent	0368145	340	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0368145	340	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0368144	56	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0368144	56	Cary	NC	27519

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Table 2.4.1-209 (Sheet 6 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type:	9
Groundwater Under the Direct Influence of Surface Water Type:	3
Groundwater Type:	431

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Community	Ground	Well D	Well House	Permanent	0368119	105	Hillsborough	NC	27278
Community	Ground	Well A & B	Well House	Permanent	0368119	105	Hillsborough	NC	27278
Community	Ground	Plant #1	Treatment Plant	Permanent	0368118	70	Hillsborough	NC	27278
Community	Ground	Well #1	Well House	Permanent	0368116	102	Cary	NC	27519
Community	Ground	Treatment House	Well House	Permanent	0368105	312	Durham	NC	27703
Community	Ground	Well #3	Well House	Permanent	0353127	137	Sanford	NC	27331
Community	Ground	Well #1	Well House	Permanent	0353127	137	Sanford	NC	27331
Community	Ground	Plant 1	Well House	Permanent	0353126	64	Sanford	NC	27330
Community	Ground	Wells #1 and #2	Treatment Plant	Permanent	0353123	305	Carthage	NC	28327
Community	Ground	Plant #2	Well House	Permanent	0353122	411	Sanford	NC	27331
Community	Ground	Plant #1	Well House	Permanent	0353122	411	Sanford	NC	27331
Community	Ground	Well #1	Well House	Permanent	0353119	103	Sanford	NC	27330
Community	Ground	Plant #16	Well House	Permanent	0353101	2432	Garner	NC	27529
Community	Ground	Plant #4	Well House	Permanent	0353101	2432	Garner	NC	27529
Community	Ground	Plant #2	Well House	Permanent	0353015	1476	Broadway	NC	27505
Community	Ground	Plant #1	Well House	Permanent	0353015	1476	Broadway	NC	27505
Community	Ground	Plant 1	Treatment Plant	Permanent	0351413	65	Henderson	NC	27536
Community	Ground	Well #1	Well House	Other	0351192	86	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0351190	53	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0351186	500	Cary	NC	27519
Community	Ground	WELLS #2 & #3	Well House	Permanent	0351186	500	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0351185	150	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0351185	150	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0351184	125	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0351176	125	Garner	NC	27529
Community	Ground	Southhills #3	Well House	Permanent	0351168	569	Cary	NC	27519
Community	Ground	Southhills #2	Well House	Permanent	0351168	569	Cary	NC	27519
Community	Ground	Pleasant Woods5	Well House	Permanent	0351168	569	Cary	NC	27519
Community	Ground	Southhills #4	Well House	Permanent	0351168	569	Cary	NC	27519
Community	Ground	Southwoods #1	Well House	Permanent	0351168	569	Cary	NC	27519
Community	Ground	Plant 1	Well House	Permanent	0351167	107	Cary	NC	27519
Community	Ground	Plant 1	Well House	Permanent	0351161	81	Raleigh	NC	27603
Community	Ground	Plant 1	Well House	Permanent	0351156	114	Cary	NC	27519
Community	Ground	Plant #2	Well House	Permanent	0351154	193	Cary	NC	27519
Community	Ground	Plant #1	Well House	Permanent	0351154	193	Cary	NC	27519
Community	Ground	Well #1	Well House	Permanent	0351150	69	Fuquay-Varina	NC	27526
Community	Ground	Well #1	Well House	Permanent	0351104	140	Cary	NC	27519
Community	Ground	Well #2	Well House	Permanent	0351104	140	Cary	NC	27519
Community	Ground	Well #3	Well House	Permanent	0351104	140	Cary	NC	27519
Community	Ground	Well #10	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #3	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #1	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #2	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #8	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #11	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #7	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #9	Well House	Permanent	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #6	Well House	Deactivate	0343030	4039	Buies Creek	NC	27506
Community	Ground	Well #1	Well House	Permanent	0319137	180	Durham	NC	27705
Community	Ground	Well #2	Well House	Permanent	0319137	180	Durham	NC	27705
Community	Ground	Well #1	Well House	Permanent	0319136	121	Chapel Hill	NC	27514
Community	Ground	Plant #1	Well House	Permanent	0319135	133	Greensboro	NC	27425
Community	Ground	Well #1	Well House	Permanent	0319134	70	Greensboro	NC	27425
Community	Ground	Plant #1	Well House	Permanent	0319133	41	Durham	NC	27704
Community	Ground	Plant #1	Well House	Permanent	0319132	154	Greensboro	NC	27425
Community	Ground	Plant #1	Well House	Permanent	0319130	100	Chapel Hill	NC	27510
Community	Ground	Plant #1	Well House	Permanent	0319128	84	Durham	NC	27704
Community	Ground	Plant #1	Well House	Permanent	0351164	48	Fuquay-Varina	NC	27526
Community	Ground	Plant #1	Treatment Plant	Permanent	0319125	70	Chapel Hill	NC	27514
Community	Ground	Plant #2	Well House	Permanent	0319124	70	Apex	NC	27502
Community	Ground	Plant #1	Well House	Permanent	0319124	70	Apex	NC	27502
Community	Ground	Well #1	Well House	Permanent	0319123	382	Greensboro	NC	27425
Community	Ground	Plant #1	Well House	Permanent	0319120	120	Durham	NC	27704
Community	Ground	Well #2	Well House	Permanent	0319110	143	Greensboro	NC	27425
Community	Ground	Well #1	Well House	Permanent	0319110	143	Greensboro	NC	27425
Community	Ground	Plant #3	Well House	Deactivate	0319104	360	Greensboro	NC	27425
Community	Ground	Plant #1	Well House	Deactivate	0319104	360	Greensboro	NC	27425
Community	Ground	Plant #1	Well House	Permanent	0319103	45	Chapel Hill	NC	27510
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392434	200	Raleigh	NC	27606
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392420	29	Garner	NC	27529
Non-Transient, Non-Community	Ground	Plant #1	Well House	Permanent	4392404	1299	Raleigh	NC	27610
Non-Transient, Non-Community	Ground	Plant #1	Well House	Permanent	0392980	76	Raleigh	NC	27611
Non-Transient, Non-Community	Ground	Well #3/Storage	Storage	Permanent	0392720	100	Raleigh	NC	27628
Non-Transient, Non-Community	Ground	Grade Sch Well	Well House	Deactivate	0368470	210	Chapel Hill	NC	27516
Non-Transient, Non-Community	Ground	Well #3	Well House	Permanent	0368470	210	Chapel Hill	NC	27516
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	0332590	60	Chapel Hill	NC	27514

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Table 2.4.1-209 (Sheet 7 of 7)
Public Water Supply Users within 25 Miles of the HAR Site by Water Type

Surface Water Type: 9
Groundwater Under the Direct Influence of Surface Water Type: 3
Groundwater Type: 431

Public Water Supply Type	Water Type	Source Name	Function	Availability	Public Water Supply ID	Population	City	State	Zip
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319438	25	Chapel Hill	NC	27515
Non-Transient, Non-Community	Ground	Well #2	Well House	Permanent	0319437	80	Moncure	NC	27559
Non-Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319437	80	Moncure	NC	27559
Non-Transient, Non-Community	Ground	Plant #1	Well House	Permanent	0319432	62	Siler City	NC	27344
Non-Transient, Non-Community	Ground	Plant #1	Treatment Plant	Permanent	0319431	68	Siler City	NC	27344
Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392444	200	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392435	50	Raleigh	NC	27602
Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392421	30	Willow Springs	NC	27592
Transient, Non-Community	Ground	Plant #1	Treatment Plant	Permanent	4392418	25	Durham	NC	27713
Transient, Non-Community	Ground	Well #2	Well House	Permanent	4392402	100	Raleigh	NC	27607
Transient, Non-Community	Ground	Well #1	Well House	Permanent	4392402	100	Raleigh	NC	27607
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392927	200	Garner	NC	27529
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392759	90	Raleigh	NC	27606
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392587	300	Willow Springs	NC	27592
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392448	25	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0392429	150	Raleigh	NC	27606
Transient, Non-Community	Ground	Plant #1	Well House	Permanent	0392427	25	Durham	NC	27713
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0368471	50	Chapel Hill	NC	27516
Transient, Non-Community	Ground	Plant #1	Treatment Plant	Permanent	0368463	70	Chapel Hill	NC	27516
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0353415	100	Sanford	NC	27330
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319492	70	Siler City	NC	27344
Transient, Non-Community	Ground	Plant #1	Well House	Deactivate	0319454	154	Bennett	NC	27208
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319440	25	Apex	NC	27520
Transient, Non-Community	Ground	Well #1	Pumping Facility	Permanent	0319429	30	Apex	NC	27502
Transient, Non-Community	Ground	Plant #1	Well House	Permanent	0319428	75	Apex	NC	27502
Transient, Non-Community	Ground	Well	Well House	Permanent	0319424	100	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319420	100	Apex	NC	27502
Transient, Non-Community	Ground	Well #1	Well House	Permanent	0319404	80	Bear Creek	NC	27207

Source: Reference 2.4-208

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Table 2.4.1-210 (Sheet 1 of 2)
USGS County Water Use Data – North Carolina 2000

		All Counties within 10 Miles of HAR Site				Additional Counties within 25 Miles of HAR Site					Additional Counties within 50 Miles of HAR Site																
	Units	Chatham	Harnett	Lee	Wake	Alamance	Durham	Johnston	Moore	Orange	Caswell	Cumberland	Franklin	Granville	Guilford	Hoke	Montgomery	Nash	Person	Randolph	Richmond	Robeson	Sampson	Scotland	Vance	Wayne	Wilson
Federal Information Processing Standards (FIPS)		37037	37085	37105	37183	37001	37063	37101	37125	37135	37033	37051	37069	37077	37081	37093	37123	37127	37145	37151	37153	37155	37163	37165	37181	37191	37195
State		NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
State FIPS Code		37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37	37
County FIPS Code		037	085	105	183	001	063	101	125	135	033	051	069	077	081	093	123	127	145	151	153	155	163	165	181	191	195
Year		2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000	2000
Total Population of County	Thousands	49.33	91.03	49.04	627.85	130.80	223.31	121.97	74.77	118.23	23.50	302.96	47.26	48.50	421.05	33.65	26.82	87.42	35.62	130.45	46.56	123.34	60.16	36.00	42.95	113.33	73.81
		Public Supply				Public Supply					Public Supply																
Total Population Served	Thousands	22.59	85.91	29.44	538.83	68.12	166.66	48.63	42.71	90.45	2.76	258.41	10.53	17.38	265.74	24.44	17.50	68.05	8.75	63.35	34.21	81.08	21.80	23.25	20.40	89.68	48.08
Ground-water Withdrawals, Fresh Coded	Mgal/day	0.16	0.31	0.33	4.93	0.26	0.34	1.35	3.09	0.52	0.07	4.85	0.28	0.13	0.79	1.50	0.09	0.81	0.02	0.67	0.00	11.58	2.63	4.48	0.07	5.76	0.57
Surface-water Withdrawals, Fresh Coded	Mgal/day	4.57	9.69	6.89	67.83	17.91	30.13	5.54	3.88	12.44	0.24	34.88	2.06	2.60	55.86	0.00	2.77	14.90	3.62	6.87	7.35	8.82	0.00	0.00	5.26	6.26	8.31
Total Withdrawals, Fresh	Mgal/day	4.73	10.00	7.22	72.76	18.17	30.47	6.89	6.97	12.96	0.31	39.73	2.34	2.73	56.65	1.50	2.86	15.71	3.64	7.54	7.35	20.40	2.63	4.48	5.33	12.02	8.88
		Domestic Water Use				Domestic Water Use					Domestic Water Use																
Self-Supplied Population	Thousands	26.74	5.12	19.60	89.02	62.68	56.65	73.34	32.06	27.78	20.74	44.55	36.73	31.12	155.31	9.21	9.32	19.37	26.87	67.10	12.35	42.26	38.36	12.75	22.55	23.65	25.73
Ground-water Withdrawals, Fresh Coded	Mgal/day	1.87	0.36	1.37	6.23	4.39	3.97	5.13	2.24	1.94	1.45	3.12	2.57	2.18	10.87	0.64	0.65	1.36	1.88	4.70	0.86	2.96	2.69	0.89	1.58	1.66	1.80
Surface-water Withdrawals, Fresh Coded	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total Withdrawals, Fresh	Mgal/day	1.87	0.36	1.37	6.23	4.39	3.97	5.13	2.24	1.94	1.45	3.12	2.57	2.18	10.87	0.64	0.65	1.36	1.88	4.70	0.86	2.96	2.69	0.89	1.58	1.66	1.80
		Industrial Water Use				Industrial Water Use					Industrial Water Use																
Ground-water Withdrawals, Fresh Coded	Mgal/day	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.00	0.00	0.00	0.02	0.00	1.35	1.06	0.15	0.00	0.00	0.00
Total Withdrawals, Ground-water	Mgal/day	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.00	0.00	0.00	0.02	0.00	1.35	1.06	0.15	0.00	0.00	0.00
Surface-water Withdrawals, Fresh Coded	Mgal/day	0.47	1.07	0.00	0.00	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.74	0.00	0.00	0.00
Total Withdrawals, Surface-water	Mgal/day	0.47	1.07	0.00	0.00	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.74	0.00	0.00	0.00
Total Withdrawals, Fresh	Mgal/day	0.49	1.07	0.00	0.00	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.45	0.00	0.00	0.00	0.02	0.00	1.35	1.06	3.89	0.00	0.00	0.00
Total Withdrawals	Mgal/day	0.49	1.07	0.00	0.00	0.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.45	0.00	0.00	0.00	0.02	0.00	1.35	1.06	3.89	0.00	0.00	0.00
		Irrigation				Irrigation					Irrigation																
Irrigation, acres irrigated, sprinkler	Thousands	0.70	3.58	1.17	7.66	2.09	1.20	2.79	6.16	1.34	1.64	3.18	3.29	2.63	4.21	0.31	0.59	15.72	1.43	1.80	1.18	2.13	8.96	1.40	1.87	2.39	3.45
Irrigation, acres irrigated, microirrigation	Thousands	0.18	0.06	0.01	0.05	0.00	0.00	0.05	0.03	0.01	0.00	0.02	0.03	0.05	0.10	0.00	0.15	0.31	0.03	0.01	0.02	0.03	0.45	0.00	0.02	0.30	0.06
Irrigation, acres irrigated, total	Thousands	0.88	3.64	1.18	7.71	2.09	1.20	2.84	6.19	1.35	1.64	3.20	3.32	2.68	4.31	0.31	0.74	16.03	1.46	1.81	1.20	2.16	9.41	1.40	1.89	2.69	3.51
Irrigation, ground-water withdrawals, fresh	Mgal/day	0.15	0.82	0.31	3.54	0.00	0.47	1.13	3.42	0.52	1.47	0.99	0.07	0.23	1.00	0.23	0.27	1.38	0.14	0.46	0.21	0.92	1.65	0.52	0.00	1.20	1.59
Irrigation, surface-water withdrawals, fresh	Mgal/day	1.33	3.26	1.76	10.59	3.62	2.70	3.34	10.27	1.47	0.00	3.98	3.06	2.21	8.96	0.57	0.82	12.27	1.22	3.15	1.19	2.09	6.59	1.57	1.92	2.41	3.64
Irrigation, total withdrawals, fresh	Mgal/day	1.48	4.08	2.07	14.13	3.62	3.17	4.47	13.69	1.99	1.47	4.97	3.13	2.44	9.96	0.80	1.09	13.65	1.36	3.61	1.40	3.01	8.24	2.09	1.92	3.61	5.23

HAR COL 2.4-1

Table 2.4.1-210 (Sheet 2 of 2)
USGS County Water Use Data – North Carolina 2000

		All Counties within 10 Miles of HAR Site				Additional Counties within 25 Miles of HAR Site					Additional Counties within 50 Miles of HAR Site																
	Units	Chatham	Harnett	Lee	Wake	Alamance	Durham	Johnston	Moore	Orange	Caswell	Cumberland	Franklin	Granville	Guilford	Hoke	Montgomery	Nash	Person	Randolph	Richmond	Robeson	Sampson	Scotland	Vance	Wayne	Wilson
		Livestock Water Use				Livestock Water Use					Livestock Water Use																
Ground-water Withdrawals, Fresh Coded	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.04	0.00
Total Withdrawals, Fresh	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.00	0.04	0.00
		Livestock Water Use (Stock)				Livestock Water Use (Stock)					Livestock Water Use (Stock)																
Ground-water Withdrawals, Fresh Coded	Mgal/day	3.03	2.28	0.57	0.12	0.38	0.03	2.11	1.82	0.24	0.14	0.53	0.13	0.05	0.04	0.21	0.64	1.17	0.08	3.41	1.76	2.89	10.71	0.93	0.01	2.01	0.18
Surface-water Withdrawals, Fresh Coded	Mgal/day	0.98	0.24	0.14	0.00	0.35	0.01	0.11	1.79	0.06	0.01	0.06	0.37	0.15	0.30	0.05	1.52	0.14	0.06	2.28	0.96	0.34	0.00	0.17	0.02	1.99	0.02
Total Withdrawals, Fresh	Mgal/day	4.01	2.52	0.71	0.12	0.73	0.04	2.22	3.61	0.30	0.15	0.59	0.50	0.20	0.34	0.26	2.16	1.31	0.14	5.69	2.72	3.23	10.71	1.10	0.03	4.00	0.20
		Thermoelectric Power Water Use (All Fuel Types)				Thermoelectric Power Water Use (All Fuel Types)					Thermoelectric Power Water Use (All Fuel Types)																
Surface-water Withdrawals, Fresh Coded	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.80	0.00	0.00	0.00	20.59	0.00
Total Withdrawals, Surface-water	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.80	0.00	0.00	0.00	20.59	0.00
Total Withdrawals, Fresh	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.82	0.00	0.00	0.00	20.59	0.00
Total Withdrawals	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.82	0.00	0.00	0.00	20.59	0.00
		Thermoelectric Power Once-Through				Thermoelectric Power Once-Through					Thermoelectric Power Once-Through																
Surface-water Withdrawals, Fresh Coded	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.80	0.00	0.00	0.00	20.59	0.00
Total Withdrawals, Surface-water	Mgal/day	352.00	0.00	0.00	34.58	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1052.20	0.00	0.00	6.80	0.00	0.00	0.00	20.59	0.00
		Thermoelectric Power Closed-Loop				Thermoelectric Power Closed-Loop																					
Ground-water Withdrawals, Fresh Coded	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00
Total Withdrawals, Fresh	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00
Total Withdrawals	Mgal/day	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00
		Totals				Totals					Totals																
Total Ground-water Withdrawals, Fresh Coded	Mgal/day	5.23	3.77	2.58	14.82	5.03	4.81	9.72	10.57	3.22	3.13	9.49	3.05	2.59	12.70	2.78	1.65	4.72	2.12	9.26	2.83	19.72	18.81	6.97	1.66	10.67	4.14
Total Withdrawals, Ground-water	Mgal/day	5.23	3.77	2.58	14.82	5.03	4.81	9.72	10.57	3.22	3.13	9.49	3.05	2.59	12.70	2.78	1.65	4.72	2.12	9.26	2.83	19.72	18.81	6.97	1.66	10.67	4.14
Total Surface-water Withdrawals, Fresh Coded	Mgal/day	359.35	14.26	8.79	113.00	22.11	32.84	8.99	15.94	13.97	0.25	38.92	5.49	4.96	66.37	0.87	5.11	27.31	1057.10	12.30	9.50	18.05	6.59	5.48	7.20	31.25	11.97
Total Withdrawals, Surface-water	Mgal/day	359.35	14.26	8.79	113.00	22.11	32.84	8.99	15.94	13.97	0.25	38.92	5.49	4.96	66.37	0.87	5.11	27.31	1057.10	12.30	9.50	18.05	6.59	5.48	7.20	31.25	11.97
Total Withdrawals, Fresh	Mgal/day	364.58	18.03	11.37	127.82	27.14	37.65	18.71	26.51	17.19	3.38	48.41	8.54	7.55	79.07	3.65	6.76	32.03	1059.22	21.56	12.33	37.77	25.40	12.45	8.86	41.92	16.11
Total Withdrawals	Mgal/day	364.58	18.03	11.37	127.82	27.14	37.65	18.71	26.51	17.19	3.38	48.41	8.54	7.55	79.07	3.65	6.76	32.03	1059.22	21.56	12.33	37.77	25.40	12.45	8.86	41.92	16.11

Source: Reference 2.4-251

HAR COL 2.4-2

Table 2.4.2-201
Calculated Peak Flood Magnitudes and Frequencies at the Buckhorn Creek and Lillington Monitoring Stations

Monitoring Station	Recurrence Interval (year)	Streamflow (cfs)
Buckhorn Creek	2.33	764
	10	1907
	25	2695
	50	3326
	100	3985
Lillington	2.33	34,624
	10	57,900
	25	73,104
	50	85,389
	100	98,510

HAR COL 2.4-2

Table 2.4.2-202
Probable Maximum Precipitation Estimate for a 2.6-km^2 (1-mi.^{2}) Area

Duration		Precipitation (inches)
Minutes	**Hours**	
5	0.083	6.2
15	0.250	9.7
30	0.500	14.0
60	1	18.9
360	6	36.7
720	12	41.9
1440	24	47.2

HAR COL 2.4-2

Table 2.4.2-203 (Sheet 1 of 2)
Yearly Peak Streamflow Measurements for Buckhorn Creek, North Carolina
Buckhorn Creek Monitoring Station
Chatham County (Near Corinth)
USGS Station Identification #: 02102192
Hydrologic Unit Code: 3030004
Latitude: 35°33'35"
Longitude: -78°58'25"
Drainage Area: 76.3 mi.^{2}

Year	Date	Gage Height (ft.)	Streamflow (cfs)
1973	February 2, 1973	20.02	6920
1974	August 7, 1974	9.84	1410
1975	July 16, 1975	12.00	2300
1976	January 28, 1976	7.54	1060
1977	March 14, 1977	12.73	2520
1978	April 26, 1978	17.48	4660
1979	April 4, 1979	12.68	2440
1980	November 3, 1979	9.27	1420
1981	February 19, 1981	3.93	284
1982	July 29, 1982	6.20	703
1983	April 15, 1983	7.45	968
1984	March 28, 1984	9.58	1500
1985	Feburary 2, 1985	4.50	381
1986	August 20, 1986	11.65	2110
1987	January 22, 1987	8.27	1160
1988	March 10, 1988	2.33	46
1989	July 16, 1989	9.57	1500
1990	March 31, 1990	4.39	363
1991	January 11, 1991	4.80	429
1992	June 26, 1992	6.71	800
1993	April 6, 1993	7.29	931
1994	March 2, 1994	6.56	770
1995	June 29, 1995	5.48	526
1996	September 6, 1996	16.79	4300

HAR COL 2.4-2

Table 2.4.2-203 (Sheet 2 of 2)
Yearly Peak Streamflow Measurements for Buckhorn Creek, North Carolina

Year	Date	Gage Height (ft.)	Streamflow (cfs)
1997	July 24, 1997	7.60	1000
1998	March 19, 1998	10.44	1740
1999	September 16, 1999	9.51	1480
2000	June 19, 2000	7.33	941
2001	August 12, 2001	9.25	1410
2002	January 23, 2002	ND	513
2003	April 10, 2003	9.55	1490
2004	May 2, 2004	4.40	335
2005	March 17, 2005	4.18	303

Notes:
ND = no data recorded
The gage height for 1985 is an estimate.
The 1981 to 2005 discharge is affected by regulation or diversion.

Source: Reference 2.4-201

HAR COL 2.4-2

Table 2.4.2-204 (Sheet 1 of 2)
Yearly Peak Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

Cape Fear River Monitoring Station
Harnett County
USGS Station Identification #: 02102500
Hydrologic Unit Code: 3030004
Latitude: 35°24'22"
Longitude: -78°48'48"
Drainage Area: 3,464.0 mi.^{2}

Year	Date	Gage Height (ft.)	Streamflow (cfs)
1924	September 30, 1924	18.80	52,400
1925	January 12, 1925	17.60	46,200
1926	January 19, 1926	13.30	27,300
1927	March 7, 1927	14.80	33,200
1928	September 20, 1928	24.80	84,000
1929	March 1, 1929	21.90	67,700
1930	October 2, 1929	27.55	107,000
1931	August 21, 1931	13.75	29,200
1932	March 7, 1932	18.74	50,900
1933	October 18, 1932	13.75	29,200
1934	April 10, 1934	16.45	40,000
1935	December 2, 1934	16.55	41,000
1936	April 7, 1936	22.88	73,200
1937	January 29, 1937	15.21	34,800
1938	July 27, 1938	17.87	47,000
1939	February 10, 1939	18.00	47,500
1940	February 8, 1940	14.54	32,000
1941	November 15, 1940	14.40	31,600
1942	February 18, 1942	14.16	30,800
1943	July 14, 1943	16.55	40,900
1944	March 21, 1944	16.90	42,300
1945	September 19, 1945	33.19	150,000
1946	February 11, 1946	19.45	54,400
1947	January 14, 1947	16.30	39,600
1948	February 15, 1948	18.50	49,900
1949	November 29, 1948	19.15	53,400
1950	May 15, 1950	15.55	36,500
1951	April 9, 1951	15.18	34,800
1952	March 5, 1952	23.55	77,100
1953	February 16, 1953	17.34	44,100
1954	January 23, 1954	19.83	56,500
1955	October 17, 1954	18.50	49,900
1956	March 17, 1956	17.84	46,500
1957	February 2, 1957	16.80	41,800
1958	November 26, 1957	17.80	46,500
1959	April 20, 1959	16.42	40,000

HAR COL 2.4-2

Table 2.4.2-204 (Sheet 2 of 2)
Yearly Peak Streamflow Measurements for Cape Fear River, North Carolina, at Lillington

Year	Date	Gage Height (ft.)	Streamflow (cfs)
1960	April 6, 1960	18.04	47,500
1961	March 22, 1961	15.63	36,500
1962	January 7, 1962	19.76	56,500
1963	March 7, 1963	16.92	42,300
1964	April 9, 1964	16.94	42,300
1965	July 28, 1965	20.00	57,500
1966	March 1, 1966	18.43	49,400
1967	August 25, 1967	13.06	26,400
1968	January 15, 1968	15.59	36,500
1969	February 3, 1969	14.32	31,300
1970	February 18, 1970	15.74	37,200
1971	March 4, 1971	16.66	41,300
1972	October 25, 1971	17.24	43,900
1973	February 3, 1973	19.27	53,800
1974	January 29, 1974	12.12	22,900
1975	July 16, 1975	18.00	47,500
1976	January 28, 1976	10.87	19,100
1977	January 10, 1977	12.70	25,000
1978	April 27, 1978	15.66	36,800
1979	February 26, 1979	18.23	48,700
1980	March 21, 1980	13.33	27,500
1981	February 12, 1981	11.48	21,000
1982	June 11, 1982	15.26	35,100
1983	March 19, 1983	14.54	32,200
1984	January 11, 1984	14.90	33,600
1985	August 19, 1985	12.40	24,100
1986	November 22, 1985	13.42	27,800
1987	March 1, 1987	15.59	36,500
1988	January 21, 1988	7.60	9950
1989	March 24, 1989	13.73	29,000
1990	October 4, 1989	11.43	20,800
1991	January 12, 1991	13.76	29,100
1992	April 24, 1992	10.56	18,000
1993	April 6, 1993	15.81	37,500
1994	March 3, 1994	14.00	30,000
1995	June 29, 1995	14.01	30,000
1996	September 7, 1996	18.97	51,800
1997	July 24, 1997	14.26	29,000
1998	March 19, 1998	18.37	47,500
1999	September 16, 1999	14.46	29,800
2000	January 31, 2000	9.70	14,600
2001	March 30, 2001	11.41	19,500
2002	January 24, 2002	9.39	13,800
2003	April 11, 2003	17.07	41,200
2004	September 9, 2004	10.83	18,900
2005	January 15, 2005	11.35	20,500

Notes:
The 1973 to 1981 discharge is affected to an unknown degree by regulation or diversion.
The 1982 to 2005 discharge is affected by regulation or diversion.

Source: Reference 2.4-271

HAR COL 2.4-2

Table 2.4.2-205
Zones A and B – Water Levels and Flow Velocities

ZONE A				ZONE B			
Cross-Section (Downstream to Upstream)	Velocity (ft/sec)	Froude Number	Water Level (ft. NGVD29)	Cross-Section (Downstream to Upstream)	Velocity (ft/sec)	Froude Number	Water Level (ft. NGVD 29)
A 000	3.71	1.00	258.47	B 000	3.77	1.00	260.44
A 120	1.37	0.20	258.90	B 160	0.55	0.07	260.71
A 220	1.58	0.22	258.96	B 340	0.33	0.04	260.72
A 320	1.47	0.21	259.04	B 475	0.32	0.04	260.73
A 420	1.25	0.18	259.10	B 650	0.31	0.04	260.73
A 535	1.07	0.15	259.16	B 750	0.34	0.04	260.73
A 685	0.88	0.12	259.21	B 870	0.72	0.15	260.73
A 840	1.17	0.20	259.25	B 955	0.41	0.08	260.75
A 926	3.88	1.01	259.86	B 1070	0.28	0.06	260.76
A 987	1.18	0.34	260.33	B 1185	0.19	0.04	260.76
A 1000	2.06	0.59	260.33	B 1295	0.00	0.00	260.76
A 1050	1.30	0.33	260.48	-	-	-	-
A 1140	0.67	0.16	260.56	-	-	-	-
A 1300	0.60	0.17	260.61	-	-	-	-
A 1502	0.0	0.00	260.62	-	-	-	-

HAR COL 2.4-2

Table 2.4.3-201
6-Hour Incremental PMP Depths (HMR 51)

Area ($mi.^2$)	PMP Depths (inches) for Various Durations				
	6-Hr.	12-Hr.	24-Hr.	48-Hr.	72-Hr.
10	29.8	35.5	41	45	47.4
200	21.8	26	31	35.5	37.0
1000	15.8	20.8	26	30	31.5
5000	9.25	13.25	18	22	23.6

HAR COL 2.4-2

Table 2.4.3-202
6-Hour Incremental PMP Depths (after Smoothing)

Area ($mi.^2$)	PMP Depths (inches) for Various Durations				
	6-Hr.	12-Hr.	24-Hr.	48-Hr.	72-Hr.
10	29.9	35.5	41.0	45.0	47.1
200	21.8	26.1	31.2	35.5	37.1
1000	15.8	20.5	25.7	30.0	31.7
5000	9.3	13.4	18.1	22.0	23.6

HAR COL 2.4-2

Table 2.4.3-203
Depth-Area-Duration Values for the Selected Standard Areas at 35°38'00" N, 78°57'22'' W

Duration (hr.)	PMP Values (inches) for Selected Standard Areas						
	10 mi.^{2}	25 mi.^{2}	50 mi.^{2}	100 mi.^{2}	175 mi.^{2}	300 mi.^{2}	450 mi.^{2}
6	29.86	27.20	25.57	23.84	22.25	20.49	19.02
12	35.49	31.65	29.75	28.01	26.52	24.92	23.58
24	40.99	36.71	34.70	32.96	31.52	29.99	28.71
48	45.00	40.90	38.94	37.24	35.83	34.33	33.04
72	47.05	42.66	40.61	38.87	37.45	35.95	34.69

HAR COL 2.4-2

Table 2.4.3-204
Interpolated PMP Values for 18-Hour Duration

Area (mi.^{2})	PMP Depth (inch)
10	38.24
25	34.18
50	32.23
100	30.48
175	29.02
300	27.46
450	26.14

HAR COL 2.4-2

Table 2.4.3-205
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
10	29.9	5.63	2.75
25	27.2	4.46	2.53
50	25.6	4.19	2.48
100	23.8	4.17	2.47
175	22.2	4.28	2.50
300	20.5	4.43	2.54
450	19.0	4.56	2.57

HAR COL 2.4-2

Table 2.4.3-206
Incremental Differences for the First Three 6-Hour Periods Based on Smooth Curves of Figure 2.4.3-204

	Incremental Difference (inch)		
Area (mi.^{2})	1st 6-hr. period	2nd 6-hr. period	3rd 6-hr. period
10	29.9	5.63	2.75
25	27.2	5.31	2.69
50	25.6	5.08	2.65
100	23.8	4.84	2.60
175	22.2	4.64	2.56
300	20.5	4.46	2.53
450	19.0	4.32	2.50

HAR COL 2.4-2

Table 2.4.3-207
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. 27.2**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 22.2**	**Avg. Depth**	**Delta A**	**Delta V**
25	A	102	27.74	27.74	10	277.40	175	A	119	26.47	26.47	10	264.73
	B	95	25.84	26.79	15	401.82		B	111	24.69	25.58	15	383.75
	C	67	18.22	22.03	25	550.72		C	103	22.91	23.80	25	595.08
	D	52	14.14	16.59	20.3	336.77		D	96	21.36	22.29	20.3	452.50
		316			Sum =	1566.71						Sum =	1696.06
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt. 25.6**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 20.5**	**Avg. Depth**	**Delta A**	**Delta V**
50	A	106	27.10	27.10	10	270.99	300	A	126	25.81	25.81	10	258.15
	B	99	25.31	26.20	15	393.07		B	118	24.18	25.00	15	374.93
	C	92	23.52	24.41	25	610.37		C	110	22.54	23.36	25	583.91
	D	66	16.87	20.86	20.3	423.48		D	103	21.10	21.96	20.3	445.85
					Sum =	1697.91						Sum =	1662.84
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt. 23.8**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 19.0**	**Avg. Depth**	**Delta A**	**Delta V**
100	25.10	112	26.70	26.70	10	267.01	450	A	132	25.10	25.10	10	251.03
	23.58	105	25.03	25.87	15	388.00		B	124	23.58	24.34	15	365.14
	22.06	98	23.36	24.20	25	604.94		C	116	22.06	22.82	25	570.53
	20.54	90	21.46	22.60	20.3	458.79		D	108	20.54	21.45	20.3	435.47
					Sum =	1718.74						Sum =	1622.16

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-208
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. 5.31**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 4.64**	**Avg. Depth**	**Delta A**	**Delta V**
25	A	103	5.47	5.47	10	54.73	175	A	110	5.11	5.11	10	51.09
	B	98	5.21	5.34	15	80.10		B	105	4.88	4.99	15	74.90
	C	72	3.83	4.52	25	112.91		C	101.5	4.71	4.80	25	119.89
	D	59	3.13	3.55	20.3	72.05		D	97.5	4.53	4.64	20.3	94.20
					Sum =	319.78						Sum =	340.08
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt. 5.08**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 4.46**	**Avg. Depth**	**Delta A**	**Delta V**
50	A	105.5	5.35	5.35	10	53.54	300	A	111.5	4.97	4.97	10	49.72
	B	100.5	5.10	5.23	15	78.41		B	107	4.77	4.87	15	73.08
	C	96.5	4.90	5.00	25	124.98		C	103.5	4.62	4.69	25	117.34
	D	76	3.86	4.48	20.3	90.97		D	100	4.46	4.55	20.3	92.43
					Sum =	347.90						Sum =	332.58
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt. 4.84**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt. 4.32**	**Avg. Depth**	**Delta A**	**Delta V**
100	A	108	5.22	5.22	10	52.24	450	A	113	4.88	4.88	10	48.82
	B	103	4.98	5.10	15	76.55		B	109	4.71	4.80	15	71.93
	C	99	4.79	4.89	25	122.14		C	105	4.54	4.62	25	115.57
	D	95	4.60	4.71	20.3	95.64		D	102	4.41	4.48	20.3	91.04
					Sum =	346.56						Sum =	327.36

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-209
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. 27.2**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt. 22.2**	**Avg. Depth**	**Delta A**	**Delta V**
25	A	101	2.72	2.72	10	27.17	175	A	102.8	2.64	2.64	10	26.36
	B	99	2.66	2.69	15	40.35		B	101.3	2.60	2.62	15	39.25
	C	74.5	2.00	2.33	25	58.34		C	100	2.56	2.58	25	64.52
	D	60.5	1.63	1.85	20.3	37.62		D	99.2	2.54	2.56	20.3	51.88
					Sum =	163.48						Sum =	182.01
Area Size ($mi.^2$)	**Iso**	**Nomo**	**Amt. 2.65**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt. 2.53**	**Avg. Depth**	**Delta A**	**Delta V**
50	A	101.6	2.69	2.69	10	26.87	300	A	103.4	2.62	2.62	10	26.15
	B	99.8	2.64	2.66	15	39.95		B	101.9	2.58	2.60	15	38.94
	C	98.5	2.61	2.62	25	65.56		C	100.7	2.55	2.56	25	64.05
	D	78.5	2.08	2.39	20.3	48.59		D	99.8	2.52	2.54	20.3	51.52
					Sum =	180.99						Sum =	180.66
Area Size ($mi.^2$)	**Iso**	**Nomo**	**Amt. 2.60**	**Avg. Depth**	**Delta A**	**Delta V**	**Area Size ($mi.^2$)**	**Iso**	**Nomo**	**Amt. 2.50**	**Avg. Depth**	**Delta A**	**Delta V**
100	A	102.3	2.66	2.66	10	26.60	450	A	103.8	2.60	2.60	10	25.98
	B	100.7	2.62	2.64	15	39.59		B	102.4	2.56	2.58	15	38.71
	C	99.3	2.58	2.60	25	65.01		C	101.2	2.53	2.55	25	63.70
	D	98.6	2.56	2.57	20.3	52.27		D	100.3	2.51	2.52	20.3	51.24
					Sum =	183.46						Sum =	179.63

Note:
* = Weighting factor (HMR 52 Section 7.1 Step C6)

HAR COL 2.4-2

Table 2.4.3-210
Incremental Average Depths for Each 6-Hour Period for 100-mi.^{2} Drainage Area

Increment	Duration (hr.)	Cumulative PMP (inches)	Incremental PMP (inches)
1	6	23.90	23.90
2	12	27.86	3.97
3	18	30.70	2.84
4	24	32.80	2.10
5	30	34.39	1.58
6	36	35.60	1.21
7	42	36.54	0.94
8	48	37.27	0.73
9	54	37.84	0.57
10	60	38.28	0.44
11	66	38.61	0.33
12	72	38.86	0.25

HAR 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	26.70	5.22	2.66	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
B	25.03	4.98	2.62	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
C	23.36	4.79	2.58	2.10	1.58	1.21	0.94	0.73	0.57	0.44	0.33	0.25
D	21.46	4.60	2.56	1.71	1.29	0.99	0.77	0.59	0.46	0.36	0.27	0.20

HAR COL 2.4-2

Table 2.4.3-212
Computation of Drainage Average Depths (Increments 1 to 6)

Increment #1							Increment #4						
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	112	26.76	26.76	10	267.65	100	A	100	2.10	2.10	10	21.00
	B	105	25.09	25.93	15	388.93		B	100	2.10	2.10	15	31.49
	C	98	23.42	24.26	25	606.39		C	100	2.10	2.10	25	52.49
	D	90	21.51	22.46	20.3	456.01		D	78.5	1.65	1.87	20.3	38.04
					Sum =	1718.98						Sum =	143.02
				Average Depth =		24.45					Average Depth =		2.03

Increment #2							Increment #5						
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	108	4.28	4.28	10	42.83	100	A	100	1.58	1.58	10	15.85
	B	103	4.08	4.18	15	62.75		B	100	1.58	1.58	15	23.77
	C	99	3.93	4.01	25	100.13		C	100	1.58	1.58	25	39.62
	D	95	3.77	3.85	20.3	78.08		D	78.5	1.24	1.41	20.3	28.71
					Sum =	283.79						Sum =	107.96
				Average Depth =		4.04					Average Depth =		1.54

Increment #3							Increment #6						
Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV	Area Size (mi.^{2})	Iso	Nomo	Amt.	Avg. Depth	ΔA	ΔV
100	A	102.3	2.91	2.91	10	29.07	100	A	100	1.21	1.21	10	12.14
	B	100.7	2.86	2.88	15	43.26		B	100	1.21	1.21	15	18.21
	C	99.3	2.82	2.84	25	71.04		C	100	1.21	1.21	25	30.35
	D	98.6	2.80	2.81	20.3	57.08		D	78.5	0.95	1.08	20.3	22.00
					Sum =	200.45						Sum =	82.70
				Average Depth =		2.85					Average Depth =		1.18

HAR COL 2.4-2

Table 2.4.3-213
Computation of Drainage Average Depths (Increments 7 to 12)

Increment #7							Increment #10						
	I	II	III	IV	V	VI		I	II	III	IV	V	VI
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**
100	A	100	0.94	0.94	10	9.39	100	A	100	0.44	0.44	10	4.38
	B	100	0.94	0.94	15	14.08		B	100	0.44	0.44	15	6.57
	C	100	0.94	0.94	25	23.47		C	100	0.44	0.44	25	10.95
	D	78.5	0.74	0.84	20.3	17.01		D	78.5	0.34	0.39	20.3	7.93
					Sum =	63.95						Sum =	29.83
					Average Depth =	0.91						Average Depth =	0.42
Increment #8							**Increment #11**						
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**
100	A	100	0.73	0.73	10	7.29	100	A	100	0.33	0.33	10	3.35
	B	100	0.73	0.73	15	10.94		B	100	0.33	0.33	15	5.02
	C	100	0.73	0.73	25	18.23		C	100	0.33	0.33	25	8.37
	D	78.5	0.57	0.65	20.3	13.21		D	78.5	0.26	0.30	20.3	6.07
					Sum =	49.67						Sum =	22.81
					Average Depth =	0.71						Average Depth =	0.32
Increment #9							**Increment #12**						
Area Size (mi.^{2})	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**	**Area Size (mi.^{2})**	**Iso**	**Nomo**	**Amt.**	**Avg. Depth**	**ΔA**	**ΔV**
100	A	100	0.57	0.57	10	5.66	100	A	100	0.25	0.25	10	2.51
	B	100	0.57	0.57	15	8.50		B	100	0.25	0.25	15	3.77
	C	100	0.57	0.57	25	14.16		C	100	0.25	0.25	25	6.28
	D	78.5	0.44	0.51	20.3	10.26		D	78.5	0.20	0.22	20.3	4.55
					Sum =	38.59						Sum =	17.11
					Average Depth =	0.55						Average Depth =	0.24

HAR 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	24.45	24.45
2	12	28.49	4.04
3	18	31.34	2.85
4	24	33.37	2.03
5	30	34.91	1.54
6	36	36.09	1.18
7	42	37.00	0.91
8	48	37.70	0.71
9	54	38.25	0.55
10	60	38.68	0.42
11	66	39.00	0.32
12	72	39.24	0.24

HAR COL 2.4-2

Table 2.4.3-215
Distribution of PMP According to ANSI/ANS-2.8-1992

6-hour Period	Time (hours)	Incremental Average PMP	ANSI Sequence	Sequence No.	ANSI Storm Distribution	Storm Pattern	Cumulative PMP
1	6	24.45	4	1st-day	2.03	0.71	0.71
2	12	4.04	2		4.04	0.91	1.62
3	18	2.85	1		24.45	1.18	2.79
4	24	2.03	3		2.85	1.54	4.33
5	30	1.54	8	2nd-day	0.71	2.03	6.36
6	36	1.18	6		1.18	4.04	10.40
7	42	0.91	5		1.54	24.45	34.85
8	48	0.71	7		0.91	2.85	37.70
9	54	0.55	12	3rd-day	0.24	0.55	38.25
10	60	0.42	10		0.42	0.42	38.68
11	66	0.32	9		0.55	0.32	39.00
12	72	0.24	11		0.32	0.24	39.24

Note:
PMP depths are in inches.

HAR COL 2.4-2

Table 2.4.3-216
Incremental Probable Maximum Precipitation for the Basins above the Main Dam and the Auxiliary Dam

Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)	Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)
1	0.12	0.15	37	4.08	5.18
2	0.12	0.15	38	4.08	5.18
3	0.12	0.15	39	4.08	5.18
4	0.12	0.15	40	4.08	5.18
5	0.12	0.15	41	4.08	5.18
6	0.12	0.15	42	4.08	5.18
7	0.15	0.19	43	0.48	0.6
8	0.15	0.19	44	0.48	0.6
9	0.15	0.19	45	0.48	0.6
10	0.15	0.19	46	0.48	0.6
11	0.15	0.19	47	0.48	0.6
12	0.15	0.19	48	0.48	0.6
13	0.2	0.25	49	0.09	0.12
14	0.2	0.25	50	0.09	0.12
15	0.2	0.25	51	0.09	0.12
16	0.2	0.25	52	0.09	0.12
17	0.2	0.25	53	0.09	0.12
18	0.2	0.25	54	0.09	0.12
19	0.26	0.33	55	0.07	0.09
20	0.26	0.33	56	0.07	0.09
21	0.26	0.33	57	0.07	0.09
22	0.26	0.33	58	0.07	0.09
23	0.26	0.33	59	0.07	0.09
24	0.26	0.33	60	0.07	0.09
25	0.34	0.43	61	0.05	0.07
26	0.34	0.43	62	0.05	0.07
27	0.34	0.43	63	0.05	0.07
28	0.34	0.43	64	0.05	0.07
29	0.34	0.43	65	0.05	0.07
30	0.34	0.43	66	0.05	0.07
31	0.67	0.85	67	0.04	0.05
32	0.67	0.85	68	0.04	0.05
33	0.67	0.85	69	0.04	0.05
34	0.67	0.85	70	0.04	0.05
35	0.67	0.85	71	0.04	0.05
36	0.67	0.85	72	0.04	0.05

HAR COL 2.4-2

Table 2.4.3-217
Incremental Effective Probable Maximum Precipitation for the Main Dam and the Auxiliary Dam

Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)	Time (hr.)	Incremental PMP for the Main Dam (inches)	Incremental PMP for the Auxiliary Dam (inches)
1	0.00	0.00	37	1.56	2.00
2	0.00	0.00	38	1.56	2.00
3	0.00	0.00	39	1.56	2.00
4	0.00	0.00	40	1.56	2.00
5	0.00	0.00	41	1.57	2.01
6	0.00	0.00	42	1.57	2.01
7	0.00	0.00	43	0.13	0.18
8	0.00	0.00	44	0.13	0.18
9	0.00	0.00	45	0.13	0.19
10	0.00	0.00	46	0.14	0.19
11	0.00	0.00	47	0.14	0.19
12	0.00	0.00	48	0.14	0.19
13	0.00	0.00	49	0.00	0.00
14	0.00	0.00	50	0.00	0.00
15	0.00	0.00	51	0.00	0.00
16	0.00	0.00	52	0.00	0.00
17	0.00	0.00	53	0.00	0.01
18	0.00	0.00	54	0.00	0.01
19	0.00	0.02	55	0.00	0.00
20	0.00	0.02	56	0.00	0.00
21	0.00	0.02	57	0.00	0.00
22	0.00	0.03	58	0.00	0.01
23	0.00	0.03	59	0.00	0.01
24	0.00	0.03	60	0.00	0.01
25	0.04	0.07	61	0.00	0.00
26	0.04	0.08	62	0.00	0.01
27	0.04	0.08	63	0.00	0.01
28	0.04	0.08	64	0.00	0.01
29	0.05	0.08	65	0.01	0.01
30	0.05	0.08	66	0.01	0.01
31	0.18	0.26	67	0.01	0.01
32	0.19	0.26	68	0.01	0.01
33	0.19	0.26	69	0.01	0.01
34	0.19	0.26	70	0.01	0.02
35	0.19	0.26	71	0.01	0.02
36	0.19	0.27	72	0.02	0.02

HAR COL 2.4-2

Table 2.4.3-218
Sub-Basin Loss Parameters

Sub-Basin	Initial Loss I_a (inch)	Constant rate (in/hr)	% Impervious
Auxiliary Reservoir Surface	0	0	100
Main Reservoir Surface	0	0	100
Residual Land Surface	0	0	30
Sub-basin IV	0	0	30
Sub-basin V	0	0	30
Sub-basin VI	0	0	30
Sub-basin VII	0	0	30
Sub-basin VIII	0	0	30
Sub-basin IX	0	0	30

Note:

It was assumed that the watershed was in a saturated condition, (i.e., Ia = 0). Further, the hydrologic soil is classified as "HSG-C" based on soil group in the watershed as indicated by Appendix-A of TR-55 corresponding to watershed soil names Creedmoor, Mayodan, and White Store (Table 2.4.12-201) (Reference 2.4-227). Based on TR-55, the loss rates for soil group C range between 0.05 and 0.15 inches per hour (in/hr). To be on the conservative side, 0.0 in/hr was selected as the loss rate for all sub-basins.

HAR COL 2.4-2

Table 2.4.3-219
Sub-Basin Areas

Basin ID (Figure 2.4.3-201)	Area ($mi.^2$)	Notes
Sub-Basins above the Main Dam		
Sub-basin IV	12.46	Land area
Sub-basin V	3.60	Land area
Sub-basin VI	3.38	Land area
Sub-basin VII	13.16	Land area
Sub-basin VIII	4.02	Land area
Sub-basin IX	1.14	Land area
Residual Land Surface	17.60	Land area around the Main Reservoir
Main Reservoir Surface	11.94	Water surface area
Auxiliary Reservoir		
Sub-basin X	2.47	Land area
Auxiliary Reservoir Surface	0.53	Water surface area
Total	**70.29**	

HAR COL 2.4-2

Table 2.4.3-220
Sub-Basin Unit Hydrograph Characteristics

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
A (mi.^{2})	12.46	3.60	3.38	13.16	4.02	1.14	2.47	17.60
L (mi.)	5.61	3.22	2.93	5.25	2.98	1.14	2.45	9.07 (0.5)
L_c (mi.)	2.28	2.02	1.64	1.92	1.02	0.41	1.37	3.08 (0.2)
C_t	3.91	3.91	3.91	3.91	3.91	3.91	3.91	3.91
C_p	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
t_L	8.39	6.86	6.27	7.82	5.46	3.12	5.63	10.6 (2.0)
Q_p (cfs)	712	252	259	808	353	175	211	796 (4311)

Notes:
Residual Area = residual land surface
(x) = alternate conservative parameters were used for the residual area

HAR COL 2.4-2

Table 2.4.3-221
1-Hour Unit Hydrograph Parameters

Parameter	Sub-basin - IV	Sub-basin - V	Sub-basin - VI	Sub-basin - VII	Sub-basin - VIII	Sub-basin - IX	Sub-basin - X	Residual Land Main Reservoir
Q_p (cfs)	750	264	271	847	369	181	220	4992
t_p (hr)	8.5	7.1	6.5	8	5.7	3.5	5.9	2.2
D/2 (hr)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
t_L (hr)	8	6.6	6	7.5	5.2	3	5.4	1.7

HAR COL 2.4-2

Table 2.4.3-222
Comparison of Peak Flows determined using the USGS Equations and the HEC-HMS Model

Storm Return Period (year) (Col-1)	USGS Equation Based Peak Flow (cfs) (Col-2)	USGS Equation Prediction Error (%) (Col-3)	USGS Equation Based Peak Flow Corrected for Prediction Error (cfs) (Col-4)	FSAR HEC-HMS Model Based Peak Flow (cfs) (Col-5)	Peak Flow Over-prediction by FSAR HEC-HMS Model (%) (Col-6)
100	10,628	±47.00	15,624	22,488	44%
200	12,467	±48.90	18,564	24,271	31%
500	15,199	±51.60	23,042	31,329	36%

HAR COL 2.4-2

Table 2.4.3-223
1-Hour Unit Hydrograph Parameters with Peaking

Item	Sub-Basin IV	Sub-Basin V	Sub-Basin VI	Sub-Basin VII	Sub-Basin VIII	Sub-Basin IX	Sub-Basin X	Residual Area
Time to Peak, t_p (hr)	6.80	5.68	5.20	6.40	4.56	2.80	4.72	1.76
Peak Flow, Q_p (cfs)	937	330	339	1059	462	226	275	6240
Volume Check (in)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Lag time, t_L (hr)	6.30	5.18	4.70	5.90	4.06	2.30	4.22	1.26

HAR COL 2.4-2

Table 2.4.3-224
Summary of PMF Inflow to Auxiliary and Main Reservoirs, With Peaking vs. Without Peaking

Metric	Auxiliary Reservoir		Main Reservoir	
	Without Peaking	With Peaking	Without Peaking	With Peaking
Peak Inflow (cfs)	6241	6962	110,752	125,191
Peak Outflow (cfs)	5599	6261	46,742	49,807
Total Inflow (inches)	66.0	66.0	52.2	52.2
Total Outflow (inches)	64.7	64.7	45.2	45.3
Peak Storage (ac-ft.)	6650	6781	280,458	284,384
Peak Elevation (ft. NGVD29)	255.43	255.73	251.34	251.71

HAR COL 2.4-2

Table 2.4.3-225
Selection of Critical Storm

Case	PMP Storm	Peak Inflow (cfs)	Total Inflow (ac-ft.)
Case-1	Using a single PMP storm that corresponds to the Main Reservoir drainage basin (25% Peaking)	124,380	193,664
Case-2	Using two different PMP storms: PMP storm for the Main Reservoir drainage basin and a more severe PMP storm for the drainage basin above the Auxiliary Dam (25% Peaking)	125,065	195,572

HAR COL 2.4-2

Table 2.4.3-226 (Sheet 1 of 2)
HEC-RAS Computed Maximum Water Surface Profile in the Main Reservoirs

Reach	River Station	Profile	Q Total (cfs)	Min Ch El (ft. NGVD29)	W.S. Elev (ft. NGVD29)	E.G. Elev (ft. NGVD29)
Tom Jack Ck	10,000	Max WS	4514.61	218.27	252.76	252.76
Tom Jack Ck	8800	Max WS	4485.91	203.00	252.76	252.76
Tom Jack Ck	7700	Max WS	4459.73	198.00	252.76	252.76
Tom Jack Ck	6600	Max WS	4436.75	194.00	252.76	252.76
Tom Jack Ck	4500	Max WS	4395.02	192.41	252.76	252.76
Tom Jack Ck	2661.272	Max WS	4354.43	189.00	252.76	252.76
LittleWhiteOak-C	17,000	Max WS	3151.57	208.00	252.76	252.76
LittleWhiteOak-C	14,800	Max WS	3130.86	198.00	252.76	252.76
LittleWhiteOak-C	12,600	Max WS	3108.33	200.81	252.76	252.76
LittleWhiteOak-C	9800	Max WS	3081.34	202.00	252.76	252.76
LittleWhiteOak-C	7200	Max WS	3059.05	199.49	252.76	252.76
Thomas Ck	11,242.48	Max WS	420.50	225.71	252.76	252.76
Thomas Ck	9991.82	Max WS	407.44	222.27	252.76	252.76
Thomas Ck	8539.703	Max WS	394.36	220.09	252.76	252.76
Thomas Ck	6905.036	Max WS	382.44	220.00	252.76	252.76
Thomas Ck	5660.595	Max WS	370.94	208.00	252.76	252.76
Thomas Ck	4400	Max WS	358.88	198.00	252.76	252.76
Thomas Ck	3500	Max WS	349.13	200.81	252.76	252.76
Thomas Ck	2600	Max WS	339.99	202.00	252.76	252.76
Thomas Ck	2205.884	Max WS	336.04	199.49	252.76	252.76
Thomas -White CK	5118.695	Max WS	3395.09	194.00	252.76	252.76
Thomas -White CK	4600	Max WS	3386.25	195.72	252.76	252.76
Thomas -White CK	3578.48	Max WS	3366.10	196.00	252.76	252.76
Thomas -White CK	2732.033	Max WS	3351.33	192.24	252.76	252.76
White Oak	41,200	Max WS	20,878.11	232.00	252.81	252.92
White Oak	38,801.01	Max WS	20,130.56	224.00	252.76	252.80
White Oak	35,800	Max WS	19,769.86	222.00	252.76	252.76
White Oak	32,600	Max WS	19,769.37	215.75	252.76	252.76
White Oak	29,600	Max WS	20,072.04	206.00	252.76	252.76
White Oak	26,400	Max WS	20,038.68	195.39	252.76	252.76
White Oak	22,600	Max WS	19,979.58	192.56	252.76	252.76
Main Res Reach-1	19,200	Max WS	23,330.91	185.00	252.76	252.76
Main Res Reach-1	16,800	Max WS	23,300.74	184.77	252.76	252.76
Main Res Reach-1	14,200	Max WS	28,853.13	179.26	252.76	252.76
Main Res Reach-2	12,800	Max WS	33,207.56	179.21	252.76	252.76
Main Res Reach-2	11,600	Max WS	33,195.46	176.05	252.76	252.76
A-3	5000	Max WS	2.00	198.72	252.76	252.76
A-3	3800	Max WS	-17.88	195.78	252.76	252.76

HAR COL 2.4-2

Table 2.4.3-226 (Sheet 2 of 2) HEC-RAS Computed Maximum Water Surface Profile in the Main Reservoirs

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft. NGVD29)	W.S. Elev (ft. NGVD29)	E.G. Elev (ft. NGVD29)
A-3	3289.593	Max WS	-27.98	186.35	252.76	252.76
A-3	2778.718	Max WS	-37.52	188.00	252.76	252.76
Main Res Reach-3	9800	Max WS	33,157.94	170.00	252.76	252.76
Main Res Reach-3	7800	Max WS	33,131.47	175.00	252.76	252.76
Buckhorn Ck-1	22,200	Max WS	16,758.25	198.00	252.76	252.76
Buckhorn Ck-1	20,600	Max WS	16,741.93	200.81	252.76	252.76
Buckhorn Ck-1	19,400	Max WS	16,729.94	202.00	252.76	252.76
Buckhorn Ck-1	17,600	Max WS	16,713.90	199.49	252.76	252.76
Buckhorn Ck-1	16,000	Max WS	16,698.95	194.00	252.76	252.76
Cary Ck	10,400	Max WS	4017.15	198.00	252.76	252.76
Cary Ck	8100	Max WS	3993.76	200.81	252.76	252.76
Cary Ck	5800	Max WS	3971.38	202.00	252.76	252.76
Cary Ck	4100	Max WS	3956.09	199.49	252.76	252.76
Cary Ck	2600	Max WS	3942.01	194.00	252.76	252.76
Buckhorn Ck -2	13,200	Max WS	20,640.96	195.72	252.76	252.76
Buckhorn Ck -2	9015.458	Max WS	20,561.16	196.00	252.76	252.76
Buckhorn Ck -2	7400	Max WS	20,533.03	192.24	252.76	252.76
Buckhorn Ck -2	5800	Max WS	20,512.96	179.26	252.76	252.76
Buckhorn Ck -2	4200	Max WS	20,497.21	179.21	252.76	252.76
Buckhorn Ck -2	3070.306	Max WS	20,485.39	176.05	252.76	252.76
A-2	5200	Max WS	2.00	207.00	252.76	252.76
A-2	4400	Max WS	-9.85	200.00	252.76	252.76
A-2	3200	Max WS	-26.14	190.49	252.76	252.76
A-2	2600	Max WS	-33.04	183.00	252.76	252.76
A-2	2089.779	Max WS	-37.68	179.00	252.76	252.76
Main Res Reach-4	5900	Max WS	33,093.79	170.00	252.76	252.76
Main Res Reach-4	5400	Max WS	33,083.50	170.00	252.76	252.76
Main Res Reach-5	3800	Max WS	53,568.89	169.00	252.76	252.76
Main Res Reach-5	2200	Max WS	53,551.84	167.67	252.75	252.76
Main Res Reach-5	1000	Max WS	53,543.32	213.34	252.74	252.76
Main Res Reach-5	900			Inline Structure		
Main Res Reach-5	800	Max WS	53,518.79	165.00	176.28	176.58
Main Res Reach-5	600	Max WS	35,946.50	162.50	164.67	169.02
Main Res Reach-5	400	Max WS	53,540.51	160.00	163.49	167.16

HAR COL 2.4-2

Table 2.4.3-227
Maximum PMF Stillwater Elevation in the Auxiliary and Main Reservoirs

HEC-HMS Results		HEC-RAS Results	Max of HEC-HMS and -RAS
PMF Elevation Aux. Reservoir (ft. NGVD29)	**PMF Elevation Main Reservoir (ft. NGVD29)**	**PMF Elevation Main Reservoir (ft. NGVD29)**	**Selected PMF Elevation Main Reservoir (ft. NGVD29)**
256.50	252.48	252.76	252.76

HAR COL 2.4-2

Table 2.4.3-228
Reservoir Bottom Elevation

Location	Line ID	Lake Bottom Elevation (ft. NGVD29)
HAR 2	1	220
	2	220
	3	240
	4	240
	5	240
HAR 3	1	220
	2	220
	3	220
	4	240
	5	240
	6	240
	7	240
HNP	1	240
	2	220
	3	220
Dams	1	220
	2	220
	3	240
	4	240

HAR COL 2.4-2

Table 2.4.3-229
Fetch Distances for HAR 2

Line ID	Fetch Dist (mi.)
1	0.93*
2	0.88
3	0.87
4	0.40
5	0.50
6	0.50
7	0.50

*Critical Fetch Distance = 0.93 mi.

HAR COL 2.4-2

Table 2.4.3-230
Fetch Distances for HAR 3

Line ID	Fetch Dist (mi.)
1	0.85*
2	0.72
3	0.61
4	0.64
5	0.55

*Critical Fetch Distance = 0.85 mi.

HAR COL 2.4-2

Table 2.4.3-231
Fetch Distances for HNP

Line ID	Fetch Dist (mi.)
1	0.76
2	4.33*
3	2.73

*Critical Fetch Distance = 4.33 mi.

HAR COL 2.4-2

Table 2.4.3-232
Fetch Distances for Auxiliary and Main Dams

Line ID	Fetch Dist (mi.)
1	4.29*
2	4.29*
3	1.17
4	1.08

*Critical Fetch Distance = 4.29 mi.

HAR COL 2.4-2

Table 2.4.3-233
Correction for Wind Averaging Interval

Location	Line ID	St. Line Fetch, X (mi.)	St. Line Fetch, X (km)	T(X,U) (s)	Correction Factor	Wind Speed Ut (m/s)
HAR 2	1	0.85	1.36	1592	1.02	22.56
	2	0.72	1.16	1431	1.02	22.63
	3	0.61	0.98	1280	1.02	22.70
	4	0.64	1.02	1314	1.02	22.68
	5	0.55	0.88	1192	1.02	22.74
HAR 3	1	0.93	1.49	1693	1.01	22.53
	2	0.88	1.41	1632	1.01	22.55
	3	0.87	1.39	1616	1.01	22.55
	4	0.40	0.80	1116	1.03	22.79
	5	0.50	0.80	1118	1.03	22.79
	6	0.50	0.80	1119	1.03	22.79
	7	0.50	0.80	1116	1.03	22.79
HNP	1	0.76	1.22	1482	1.02	22.60
	2	4.33	6.92	4738	0.98	21.82
	3	2.73	4.36	3476	1.00	22.23
Dams	1	4.29	6.87	4713	0.98	21.83
	2	4.29	6.87	4713	0.98	21.83
	3	1.17	1.88	1975	1.01	22.45
	4	1.08	1.73	1871	1.01	22.48

HAR COL 2.4-2

Table 2.4.3-234
Coefficients in Equations (11a, 11b, and 11c) for Runup of Irregular Head-On Waves in Impermeable and Permeable Rock Armored Slopes

Percent	A	B	C	D
0.1	1.12	1.34	0.55	2.58
2	0.96	1.17	0.46	1.97
1	1.04	1.26	0.51	2.29
Significant	0.72	0.88	0.41	1.35

HAR COL 2.4-2

Table 2.4.3-235
Wave Runup Computation at HAR 2, HAR 3, HNP, and Auxiliary and Main Dams

Reservoir	PMF Elevation (ft. NGVD29)	Bottom Elevation (ft. NGVD29)	Water Depth (m)	Limiting Wave Period (sec)
Auxiliary	256.50	240.00	5.03	7.00
Main	252.76	220.00	9.98	9.87

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan (alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft.)	Max Runup (0.1%), Ru (ft.)
HAR 2	1	22.56	1.36	0.48	1.80	0.09	0.09	0.28	0.32	0.50
	2	22.63	1.16	0.44	1.71	0.09	0.10	0.28	0.29	0.45
	3	22.70	0.98	0.41	1.62	0.13	0.10	0.40	0.39	0.60
	4	22.68	1.02	0.42	1.64	0.13	0.10	0.40	0.39	0.61
	5	22.74	0.88	0.39	1.57	0.13	0.10	0.40	0.36	0.56
HAR 3	1	22.53	1.49	0.50	1.86	0.09	0.09	0.29	0.34	0.52
	2	22.55	1.41	0.49	1.83	0.09	0.09	0.28	0.33	0.51
	3	22.55	1.39	0.48	1.82	0.09	0.09	0.28	0.32	0.50
	4	22.91	0.63	0.33	1.41	0.13	0.11	0.38	0.30	0.47
	5	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	6	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.54
	7	22.79	0.80	0.37	1.52	0.13	0.10	0.39	0.34	0.53
HNP	1	22.60	1.22	0.45	1.74	0.20	0.10	0.65	0.69	1.08
	2	21.82	6.92	1.03	3.06	0.10	0.07	0.38	0.92	1.43
	3	22.23	4.36	0.84	2.64	0.10	0.08	0.36	0.71	1.11
Dams	1	21.83	6.87	1.03	3.05	0.50	0.07	1.88	3.85	6.41
	2	21.83	6.87	1.03	3.05	0.40	0.07	1.50	3.52	5.67
	3	22.45	1.88	0.56	2.00	0.40	0.09	1.34	1.77	2.75
	4	22.48	1.73	0.54	1.95	0.40	0.09	1.33	1.69	2.62

HAR COL 2.4-2

Table 2.4.3-236
Wave Setup Computation at HAR 2, HAR 3, HNP, and Auxiliary and Main Dams

Location	Line ID	Wind Velocity (mph)	St. Line Fetch, X (mi.)	Depth (ft.)	Setup (ft.)
HAR 2	1	50.77	0.85	32.76	0.05
	2	50.91	0.72	32.76	0.04
	3	51.06	0.61	16.50	0.07
	4	51.03	0.64	16.50	0.07
	5	51.17	0.55	16.50	0.06
HAR 3	1	50.69	0.93	32.76	0.05
	2	50.74	0.88	32.76	0.05
	3	50.75	0.87	32.76	0.05
	4	51.54	0.40	16.50	0.05
	5	51.27	0.50	16.50	0.06
	6	51.27	0.50	16.50	0.06
	7	51.27	0.50	16.50	0.06
HNP	1	50.86	0.76	16.50	0.09
	2	49.10	4.33	32.76	0.23
	3	50.01	2.73	32.76	0.15
Dams	1	49.12	4.29	32.76	0.23
	2	49.12	4.29	32.76	0.23
	3	50.51	1.17	16.50	0.13
	4	50.57	1.08	16.50	0.12

HAR COL 2.4-2

Table 2.4.3-237
Overall PMF Elevation at HAR 2, HAR 3, HNP, and Auxiliary and Main Dams

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMF (ft. NGVD29)
HAR 2	1	252.76	0.50	0.05	253.31
	2	252.76	0.45	0.04	253.26
	3	256.50	0.60	0.07	257.17
	4	256.50	0.61	0.07	257.18
	5	256.50	0.56	0.06	257.13
HAR 3	1	252.76	0.52	0.05	253.34
	2	252.76	0.51	0.05	253.32
	3	252.76	0.50	0.05	253.31
	4	256.50	0.47	0.05	257.01
	5	256.50	0.54	0.06	257.09
	6	256.50	0.54	0.06	257.09
	7	256.50	0.53	0.06	257.09
HNP	1	256.50	1.08	0.09	257.66
	2	252.76	1.43	0.23	254.42
	3	252.76	1.11	0.15	254.02
Dams	1	252.76	6.41	0.23	259.39
	2	252.76	5.67	0.23	258.65
	3	256.50	2.75	0.13	259.38
	4	256.50	2.62	0.12	259.24

Note:
252.76 ft. and 256.50 ft. are the maximum stillwater PMF elevations for the Main and Auxiliary reservoirs.

HAR COL 2.4-2

Table 2.4.3-238
Maximum PMF Elevation at HAR 2, HAR 3, HNP, and Auxiliary and Main Dams

Location	Maximum PMF Elevation (ft. NGVD29)
HAR 2	257.18
HAR 3	257.09
HNP	257.66
Dams	259.39

HAR COL 2.4-2

Table 2.4.3-239
Water Levels and Velocities at Different Locations in the Emergency Spillway Channel

Location	Water Level (ft. NGVD29)	Velocity (ft./sec)
Downstream of the Emergency Spillway	241.4	8.4
Upstream of the Drop Structure	237.9	15.3
Downstream of the Drop Structure	233.1	6.4
Upstream of the Railroad Crossing	231.8	10.9
Downstream of the Railroad Crossing	230.8	11.8

HAR COL 2.4-2

Table 2.4.5-201
Correction for PMH Wind Averaging Duration

Location	Line ID	St. Line Fetch, X (mi)	St. Line Fetch, X (km)	T(X,U) (s)	Correction Factor	Wind Speed Ut (m/s)
HAR 2	1	0.85	1.37	1290	1.02	42.52
	2	0.72	1.17	1160	1.02	42.66
	3	0.61	0.99	1038	1.03	42.81
	4	0.64	1.03	1065	1.03	42.77
	5	0.55	0.89	966	1.03	42.91
HAR 3	1	0.93	1.50	1372	1.02	42.45
	2	0.88	1.42	1323	1.02	42.49
	3	0.87	1.40	1311	1.02	42.50
	4	0.40	0.64	774	1.04	43.26
	5	0.50	0.81	907	1.03	43.01
	6	0.50	0.81	907	1.03	43.00
	7	0.50	0.80	905	1.03	43.01
HNP	1	0.76	1.23	1201	1.02	42.61
	2	4.33	6.96	3841	1.00	41.47
	3	2.73	4.38	2818	1.00	41.79
Dams	1	4.29	6.91	3821	1.00	41.48
	2	4.29	6.91	3821	1.00	41.48
	3	1.17	1.89	1601	1.02	42.28
	4	1.08	1.74	1517	1.02	42.34

HAR COL 2.4-2

Table 2.4.5-202
Wave Runup Computation Due to PMH Wind at HAR 2, HAR 3, HNP, and the Auxiliary and Main Dams

Reservoir	Stillwater Elevation (ft. NGVD29)	Bottom Elevation (ft. NGVD29)	Water Depth (m)	Limiting Wave Period (sec)
Auxiliary	252.0	240.0	3.66	5.97
Main	240.0	220.0	6.10	7.71

Location	Line ID	Wind Velocity (m/s)	St. Line Fetch, X (km)	Hm0 (m)	Predicted Peak Wave Period, Tp (sec)	Slope = tan (alpha)	Deepwater Wave Steepness, s0	Iribarren Number	Significant Runup, Rus (ft.)	Max Runup (0.1%), Ru (ft.)
HAR 2	1	42.52	1.37	1.05	2.35	0.09	0.12	0.25	0.62	0.96
	2	42.66	1.17	0.98	2.23	0.09	0.13	0.25	0.57	0.88
	3	42.81	0.99	0.90	2.12	0.13	0.13	0.35	0.75	1.16
	4	42.77	1.03	0.92	2.14	0.13	0.13	0.35	0.77	1.19
	5	42.91	0.89	0.86	2.04	0.13	0.13	0.35	0.71	1.10
HAR 3	1	42.45	1.50	1.10	2.42	0.09	0.12	0.25	0.65	1.02
	2	42.49	1.42	1.07	2.38	0.09	0.12	0.25	0.63	0.99
	3	42.50	1.40	1.07	2.37	0.09	0.12	0.25	0.63	0.98
	4	43.26	0.64	0.74	1.84	0.13	0.14	0.34	0.59	0.91
	5	43.01	0.81	0.82	1.98	0.13	0.13	0.34	0.67	1.04
	6	43.00	0.81	0.82	1.98	0.13	0.13	0.34	0.67	1.04
	7	43.01	0.80	0.82	1.98	0.13	0.13	0.34	0.67	1.04
HNP	1	42.61	1.23	1.00	2.27	0.20	0.12	0.57	1.34	2.09
	2	41.47	6.96	2.30	4.00	0.10	0.09	0.33	1.79	2.79
	3	41.79	4.38	1.85	3.44	0.10	0.10	0.32	1.38	2.15
Dams	1	41.48	6.91	2.30	3.99	0.50	0.09	1.65	8.13	13.28
	2	41.48	6.91	2.30	3.99	0.40	0.09	1.32	7.15	11.11
	3	42.28	1.89	1.23	2.61	0.40	0.12	1.18	3.42	5.32
	4	42.34	1.74	1.18	2.54	0.40	0.12	1.17	3.27	5.08

HAR COL 2.4-2

Table 2.4.5-203
Wave Setup Computation Due to PMH Winds at HAR 2, HAR 3, HNP, and the Auxiliary and Main Dams

Location	Line ID	Wind Velocity (mph)	St. Line Fetch, X (mi.)	Depth (ft.)	Setup (ft.)
HAR 2	1	95.67	0.85	20.00	0.28
	2	95.98	0.72	20.00	0.24
	3	96.32	0.61	12.00	0.34
	4	96.23	0.64	12.00	0.35
	5	96.55	0.55	12.00	0.31
HAR 3	1	95.51	0.93	20.00	0.30
	2	95.61	0.88	20.00	0.29
	3	95.63	0.87	20.00	0.28
	4	97.34	0.40	12.00	0.22
	5	96.76	0.50	12.00	0.28
	6	96.76	0.50	12.00	0.28
	7	96.77	0.50	12.00	0.28
HNP	1	95.88	0.76	12.00	0.42
	2	93.30	4.33	20.00	1.34
	3	94.02	2.73	20.00	0.86
Dams	1	93.34	4.29	20.00	1.34
	2	93.34	4.29	20.00	1.34
	3	95.13	1.17	12.00	0.63
	4	95.26	1.08	12.00	0.58

HAR COL 2.4-2

Table 2.4.5-204
Overall PMH Elevation at HAR 2, HAR 3, HNP, and the Auxiliary and Main Dams

Location	Line ID	Stillwater EL. (ft. NGVD29)	Max Runup (0.1%), Ru (ft.)	Setup (ft.)	Overall PMH (ft. NGVD29)
HAR 2	1	240	0.96	0.28	241.24
	2	240	0.88	0.24	241.12
	3	252	1.16	0.34	253.50
	4	252	1.19	0.35	253.54
	5	252	1.10	0.31	253.40
HAR 3	1	240	1.02	0.30	241.32
	2	240	0.99	0.29	241.27
	3	240	0.98	0.28	241.26
	4	252	0.91	0.22	253.14
	5	252	1.04	0.28	253.32
	6	252	1.04	0.28	253.32
	7	252	1.04	0.28	253.32
HNP	1	252	2.09	0.42	254.51
	2	240	2.79	1.34	244.14
	3	240	2.15	0.86	243.01
Dams	1	240	13.28	1.34	254.62
	2	240	11.11	1.34	252.45
	3	252	5.32	0.63	257.95
	4	252	5.08	0.58	257.67

Note:
240.0 ft. and 252.0 ft. NGVD29 are the stillwater water elevations in the Main and Auxiliary Reservoirs, respectively.

HAR COL 2.4-2

Table 2.4.5-205
Maximum PMH Elevation at HAR 2, HAR 3, HNP, and the Auxiliary and Main Dams

Location	Maximum PMH Elevation (ft. NGVD29)
HAR 2	253.54
HAR 3	253.32
HNP	254.51
Dams	257.95

HAR COL 2.4-2

Table 2.4.5-206
Oscillation Resonance Modes in the Main and Auxiliary Reservoirs

Location	Fetch Line ID	Straight Line Fetch, L (km)	Wave Period, Tp (sec)	Depth (ft.)	Velocity (m/s)	Wavelength (m)	Mode, n
HAR 2	1	1.36	1.80	32.76	9.9	17.8	153
	2	1.16	1.71	32.76	9.9	16.9	137
	3	0.98	1.62	16.56	7.0	11.4	172
	4	1.02	1.64	16.56	7.0	11.5	177
	5	0.88	1.57	16.56	7.0	11.0	159
HAR 3	1	1.49	1.86	32.76	9.9	18.4	162
	2	1.41	1.83	32.76	9.9	18.1	156
	3	1.39	1.82	32.76	9.9	18.0	154
	4	0.63	1.41	16.56	7.0	9.9	127
	5	0.80	1.52	16.56	7.0	10.7	150
	6	0.80	1.52	16.56	7.0	10.7	150
	7	0.80	1.52	16.56	7.0	10.7	150
HNP	1	1.22	1.74	16.56	7.0	12.2	199
	2	6.92	3.06	32.76	9.9	30.3	457
	3	4.36	2.64	32.76	9.9	26.1	334
Dams	1	6.87	3.05	32.76	9.9	30.2	455
	2	6.87	3.05	32.76	9.9	30.2	455
	3	1.88	2.00	16.56	7.0	14.1	267
	4	1.73	1.95	16.56	7.0	13.7	252

HAR COL 2.4-2

Table 2.4.5-207
Wave Amplification due to Resonance for the Main and Auxiliary Reservoirs

Location	Line ID	Mode, n	Setup (ft.)	A_n (ft.)
HAR 2	1	153	0.05	0
	2	137	0.04	0
	3	172	0.07	0
	4	177	0.07	0
	5	159	0.06	0
HAR 3	1	162	0.05	0
	2	156	0.05	0
	3	154	0.05	0
	4	127	0.05	0
	5	150	0.06	0
	6	150	0.06	0
	7	150	0.06	0
HNP	1	199	0.09	0
	2	457	0.23	0
	3	334	0.15	0
Dams	1	455	0.23	0
	2	455	0.23	0
	3	267	0.13	0
	4	252	0.12	0

HAR 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	Fragments > 10 Inches (%)	Fragments 3 - 10 Inches (%)	Sieve No. 200 (%)	Organic Matter[a] (%)	Available Water Capacity[b] (in/in)	Moist Bulk Density[c] (g/cm^3)	Porosity[d] (cm^3/cm^3)	Saturated Hydraulic Conductivity[e] (cm/sec)
Creedmoor	0-8	Fine sandy loam	SC-SM, SM	0	0-3	30-49	0.5-2.0	0.10-0.14	1.55-1.70	0.36-0.42	1.4E-03 to 4.2E-03
	8-14	Clay loam, Sandy clay loam, Silty clay loam	CL	0	0-3	60-80	0.0-0.5	0.13-0.15	1.45-1.65	0.38-0.45	1.4E-04 to 4.0E-04
	14-35	Clay, Sandy clay, Silty clay	CH	0	0-3	70-95	0.0-0.5	0.13-0.15	1.30-1.50	0.43-0.51	1.0E-06 to 4.2E-05
	35-83	Sandy clay loam, Silty clay loam, Sandy loam	CL-ML, ML, SC, SM	0	0-5	45-90	0.0-0.5	0.10-0.14	1.60-1.95	0.26-0.40	1.0E-06 to 4.2E-05
	83-99	Unweathered bedrock	---	---	---	---	---	0.00-0.01	---	---	0 to 4.2E-05
The parent material consists of residuum weathered from shale and siltstone and/or mudstone and/or sandstone. The natural drainage class is moderately well drained. Water movement in the most restrictive layer is very low. Shrink-swell potential is moderate.											
Mayodan	0-9	Sandy Loam	ML, SM	0	0-5	30-70	0.5-2.0	0.11-0.17	1.40-1.65	0.38-0.47	1.4E-03 to 4.2E-03
	9-35	Clay, Clay loam, Sandy clay, Sandy clay loam, Silty clay, Silty clay loam	CL	0	0-2	50-98	0.5-1.0	0.12-0.22	1.30-1.40	0.47-0.51	4.0E-04 to 1.4E-03
	35-44	Clay, Clay loam, Sandy clay, Sandy clay loam, Silty clay, Silty clay loam	CH, CL, MH, ML	0	0-2	50-98	0.0-0.5	0.12-0.18	1.25-1.55	0.42-0.53	4.0E-04 to 1.4E-03
	44-65	Clay loam, Sandy clay loam, Silty clay loam	CL	0	0-2	50-98	0.0-0.2	0.12-0.22	1.30-1.40	0.47-0.51	4.0E-04 to 1.4E-03
	65-75	Weathered bedrock	---	---	---	---	---	---	---	---	---
The parent material consists of residuum weathered from mudstone and/or shale and siltstone and/or sandstone. The natural drainage class is well drained. Water movement in the most restrictive layer is moderately high. Shrink-swell potential is low.											
White Store	0-6	Sandy loam	CL-ML, ML	0	0-3	56-76	0.5-2.0	0.14-0.16	1.30-1.65	0.38-0.51	4.0E-04 to 1.4E-03
	6-35	Clay	CH	0	0-3	80-98	0.0-0.5	0.15-0.17	1.15-1.35	0.49-0.57	1.0E-06 to 4.2E-05
	35-53	Clay loam, Loam, Sandy loam	CL, ML	0	0-3	55-85	0.0-0.5	0.13-0.17	1.15-1.35	0.49-0.57	4.2E-05 to 1.4E-04
	53-60	Weathered bedrock	---	---	---	---	---	0.00-0.01	---	---	0 to 1.4E-04
The parent material consists of residuum weathered from mica schist and/or metamorphic rock. The natural drainage class is moderately well drained. Water movement in the most restrictive layer is low. Shrink-swell potential is very high.											

Notes:
a) Organic matter increases the available water capacity. Each 1 percent of organic matter adds about 1.5 percent to available water capacity.

b) 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 is given in inches of water per inch of soil for each soil layer.

c) Moist bulk density is the weight of soil (oven dry) per unit volume. The moist bulk density of a soil indicated 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.

d) 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) (Reference 2.4-252).

e) Saturated hydraulic conductivity (Ksat) refers to the ease with which pores in a saturated coil transmit water.

--- = no data available
in/in = inches per inches

Source: National Resources Conservation Service (NRCS), soildatamart.nrcs.usda.gov/Report.aspx?Survey=NC183&UseState=NC, 2006; Reference 2.4-243, except where noted.

HAR COL 2.4-4

Table 2.4.12-202
Nearest Residences Relative to the HAR Site[a]

Sector	Distance from HAR Site[b] (miles)	Private Water Well (Yes/No)	Number of Wells	Total Depth (feet)	Screened Lithology of Well	In Use? (Yes/No)	Usage for Well Water
North	1.9	Yes	1	105	Bedrock	Yes	Drinking
North-Northeast	1.8	Yes	1	75	Bedrock	Yes	Drinking
Northeast	2.3	Yes	---	---	---	---	---
East-Northeast	1.8	Yes	3	(Two) 250 / (One) 150	---	Yes	Drinking and Farm Use
East	2	Yes	1	300	Bedrock	Yes	Drinking
	2.1	Yes	2	(Two) 360	Bedrock	Yes	Drinking
East-Southeast	3	Yes	1	---	Bedrock	Yes	Drinking
	5	Yes	---	---	---	---	---
Southeast	3	Yes	---	---	---	---	---
	4.5	Yes	1	310	Bedrock	Yes	Drinking
South-Southeast	4.6	Yes	---	---	---	---	---
	4.6	Yes	---	---	---	---	---
South	5.6	Yes	---	---	---	---	---
South-Southwest	4	Yes	---	---	---	---	---
Southwest	2.9	Yes	---	---	---	---	---
West-Southwest	4.3	Yes	---	---	---	---	---
West	2.7	Yes	1	---	Bedrock	Yes	Drinking
	2.8	Yes	---	---	---	---	---
West-Northwest	2.5	Yes	---	---	---	---	---
Northwest	2	Yes	1	160	Bedrock	Yes	Drinking
North-Northwest	1.2	Yes	---	---	---	---	---
	1.5	Yes	---	---	---	---	---

Notes:
a) Information was collected during the 2006 HNP Land Use Census Survey for HNP.
b) Original distances and sectors were measured using HNP as the centerpoint. New distances were calculated from the center of the HAR site.
--- = no data available

HAR COL 2.4-4

Table 2.4.12-203
Public Water Supply Users within 5 Miles of the HAR Site

Groundwater Users (G): 6

Public Water Supply	Source Type	Public Water Supply ID	Source Code	Source Name	City	State	Zip	County	Direction from HAR Site	Linear Distance (miles)
Transient, Non-Community	G	4392458	W01	Well #1	New Hill	NC	27562	Wake	East-Northeast	0.6
Transient, Non-Community	G	4392521	W01	Well #2	Raleigh	NC	27602	Wake	East-Southeast	1.1
Transient, Non-Community	G	4392520	W01	Well #1	Raleigh	NC	27602	Wake	East	1.6
Community	G	392271	W01	Well #1	New Hill	NC	27562	Wake	North-Northeast	2.3
Community	G	392271	W02	Well #2	New Hill	NC	27562	Wake	North-Northeast	2.3
Transient, Non-Community	G	392669	W01	Well #1	New Hill	NC	27562	Wake	North-Northeast	3.2
Transient, Non-Community	G	392660	W01	Well #1	New Hill	NC	27562	Wake	North-Northeast	3.7
Community	G	392078	LS1	Well #1	Fuquay-Varina	NC	27519	Wake	East-Southeast	4.9
Community	G	392078	LS2	Well #2	Fuquay-Varina	NC	27519	Wake	East-Southeast	4.9

Source: Reference 2.4-207

HAR COL 2.4-4

Table 2.4.12-204 (Sheet 1 of 4)
1997 and 2010 Cape Fear River Population and Water Use as Reported by Local Water Supply Plan (LWSP) Systems

Water Systems by County	Water Source or Supplier	Year-round Service Population 1997	Year-round Service Population 2010	Average Daily Demand (mgd) 1997	Average Daily Demand (mgd) 2010	Available Supply (mgd) 1997	Available Supply (mgd) 2010	Demand as % of Supply[b] 1997	Demand as % of Supply[b] 2010
ALAMANCE									
GREEN LEVEL[a]	Graham Mebane	1536	1705	0.107	0.119	0.132	0.132	81%	90%
OSSIPEE SD[a]	Bedrock Wells	300	425	0.024	0.034	0.03	0.53	80%	6%
ALAMANCE	Burlington	257	313	0.033	0.04	0.5	0.5	7%	8%
BURLINGTON	Lake Mackintosh / Stoney Creek	43,200	51,967	14.517	18.227	48	48	30%	38%
ELON COLLEGE	Bedrock Wells / Burlington	5045	5710	0.47	0.562	1.123	1.123	42%	50%
GRAHAM	Graham-Mebane Lake / Burlington	11,725	14,250	7.158	8.211	12	12	60%	68%
HAW RIVER	Burlington / Graham	2183	3345	0.695	0.761	1.8	2.4	39%	32%
MEBANE	Graham Mebane	5100	11,359	1.749	2.83	4	4	44%	71%
BLADEN									
BLADEN CO WD - 701 NORTH	Upper Cape Fear Aquifer	1240	2136	0.067	0.116	0.144	0.144	47%	81%
BLADEN CO WD - EAST ARCADIA	Upper Cape Fear Aquifer	496	1368	0.05	0.139	0.198	0.198	25%	70%
BLADEN CO WD - WHITE OAK	Black Creek Aquifer	1400	2860	0.063	0.129	0.31	0.31	20%	37%
ELIZABETHTOWN	Lower Cape Fear and Upper Cape Fear Aquifers	4181	4602	0.901	0.933	1.368	1.368	66%	68%
WHITE LAKE (s)	Black Creek and Upper Cape Fear Aquifers	1010	1085	0.411	0.575	0.95	0.95	43%	61%
BRUNSWICK									
NORTH BRUNSWICK WSA (LELAND SD)[a]	Brunswick Co	3464	5000	0.494	0.561	1	1	49%	56%
BRUNSWICK CO (s)	LCFWSA	61,959	83,175	17.3	23.9	27.418	27.418	63%	87%
CASWELL BEACH (s)	Brunswick Co	220	400	0.121	0.389	0.26	0.26	47%	150%
HOLDEN BEACH (s)	Brunswick Co	910	2060	0.353	1.121	0.822	0.822	43%	136%
LONG BEACH WATER (s)	Brunswick Co	4789	6797	1.044	1.514	1.32	1.32	79%	115%
NAVASSA	N Brunswick Sd	520	590	0.047	0.122	0.133	0.133	35%	92%
OCEAN ISLE BEACH (s)	Brunswick Co	689	1057	0.386	1.171	0.92	0.92	42%	128%
SHALLOTTE	Brunswick Co	1250	1380	0	0	0	0	65%	70%
SOUTHPORT	Brunswick Co / Peedee Aquifer	5124	6756	0.607	0.801	0.771	1.116	79%	72%
SUNSET BEACH (s)	Brunswick Co	1908	2350	0.501	1.358	1.085	1.085	46%	125%
YAUPON BEACH (s)	Brunswick Co / Peedee Aquifer	891	1048	0.186	0.26	0.425	0.425	44%	61%
CHATHAM									
CHATHAM CO E	Sanford	680	1218	0.069	0.116	0.3	1.8	23%	6%
CHATHAM CO N	Jordan Lake	5860	13,163	0.759	3.149	6	12	13%	26%
CHATHAM CO SW	Siler City / Goldston Gulf Sd	1793	4218	0.279	0.668	0.55	2.05	51%	33%
GOLDSTON-GULF SD	Deep River	1000	1257	0.387	0.458	2.2	2.2	18%	21%
PITTSBORO	Haw River	2022	3350	0.707	1.042	7.6	7.6	9%	14%
SILER CITY	Rocky River	5541	6929	2.8	3.4	3.8	5.8	72%	59%
COLUMBUS									
RIEGELWOOD SD	Cape Fear River	323	400	0.593	0.564	1	1	59%	56%
CUMBERLAND									
FALCON	Dunn	695	797	0.474	0.489	0.2	0.2	11%	13%
FAYETTEVILLE	Big Cross Cr./ Glenville Lake / Cape Fear River	159,225	286,500	27.809	47.936	92	92	30%	52%
FT BRAGG WTP	Little River	65,000	65,000	7.56	7.56	20	20	38%	38%
GODWIN	Falcon	203	237	0.012	0.0141	0.04	0.04	30%	35%

HAR COL 2.4-4

Table 2.4.12-204 (Sheet 2 of 4) 1997 and 2010 Cape Fear River Population and Water Use as Reported by Local Water Supply Plan (LWSP) Systems

Water Systems by County	Water Source or Supplier	Year-round Service Population 1997	Year-round Service Population 2010	Average Daily Demand (mgd) 1997	Average Daily Demand (mgd) 2010	Available Supply (mgd) 1997	Available Supply (mgd) 2010	Demand as % of Supply[b] 1997	Demand as % of Supply[b] 2010
HOPE MILLS	Fayetteville	10,433	14,750	0.838	1.2	1.33	1.33	63%	90%
LINDEN	Harnett Co	800	950	0.058	0.073	0.1	0.1	58%	73%
SPRING LAKE	Surficial Aquifer / Fayetteville	12,050	15,375	0.99	1.27	0.757	1.4	131%	90%
STEDMAN	Surficial and Upper Cape Fear Aquifers	668	887	0.108	0.089	0.157	0.157	69%	57%
WADE	Surficial Aquifer / Bedrock Wells	457	532	0.035	0.0611	0.11	0.204	32%	30%
DUPLIN (in proposed Central Coastal Plain Capacity Use Area)									
ALBERTSON WSD	Black Creek Aquifer / Duplin Co	1047	1259	0.141	0.1653	0.287	0.287	49%	57%
BEULAVILLE	Peedee and Black Creek Aquifers	1210	1263	0.136	0.151	0.396	0.396	34%	38%
CALYPSO	Upper Cape Fear Aquifer	487	460	0.105	0.106	0.317	0.317	33%	33%
CHINQUAPIN WA	Black Creek and Peedee Aquifers	3800	4500	0.233	0.4	0.648	0.648	36%	62%
DUPLIN CO COMBINED	Black Creek Aquifer / Dublin	3976	15079	0.4707	1.682	1.66	2.862	0%	0%
FAISON	Black Crk and U C Fear Aquifers / Duplin Co	752	712	0.576	0.594	0.702	0.702	82%	85%
GREENEVERS	Peedee and Black Creek Aquifers	981	1054	0.088	0.095	0.36	0.36	24%	26%
KENANSVILLE	Black Creek Aquifer	1026	1050	0.199	0.233	0.423	0.423	47%	55%
MAGNOLIA	Black Creek Aquifer	815	874	0.092	0.097	0.45	0.45	20%	22%
ROSE HILL	Black Creek Aquifer	1510	1708	0.316	0.36	0.792	0.792	40%	45%
TEACHEY	Wallace	484	360	0.03	0.034	0.035	0.035	85%	96%
WALLACE	Peedee and Black Creek Aquifers	3386	3642	2.529	0.455	2.531	2.531	100%	18%
WARSAW	Black Creek and Upper Cape Fear Aquifers	3292	3643	0.444	0.463	0.396	0.58	112%	80%
GUILFORD									
GIBSONVILLE[a]	Bedrock Wells/Burlington	3799	5815	0.399	0.576	1.131	1.381	35%	42%
GREENSBORO	Lake Higgins, Lake Brandt, Lake Townsend	199,000	214,000	40.3	50.482	36	71	112%	71%
HIGH POINT	City Lake, Oak Hollow Lake	71,160	80,063	15.519	22.277	21.44	31.44	72%	71%
JAMESTOWN	Greensboro / High Point	4329	6000	0.409	0.547	1.1	2.2	37%	25%
HARNETT									
ANGIER	Harnett Co	3010	4114	0.349	0.508	2.02	2.02	17%	25%
COATS	Harnett Co	1800	1900	0.13	0.184	0.72	0.72	22%	26%
DUNN	Cape Fear River	9731	12,561	4.643	5.56	8	8	58%	70%
ERWIN	Swift Textiles Reservoir	4265	5373	0.619	0.739	1.5	1.5	41%	49%
HARNETT CO	Cape Fear River / Dunn/Johnston Co	65,000	101,970	10.05	18.23	13.3	13.3	76%	137%
LILLINGTON	Harnett Co	3003	4341	0.478	0.742	1.3	1.3	37%	57%
JOHNSTON									
BENSON	Dunn / Johnston Co	4000	5175	1.77	1.98	1.72	1.72	103%	115%
LEE									
BROADWAY	Bedrock Wells / Sanford	1070	1246	0.093	0.111	0.096	0.162	97%	68%
LEE CO	Deep River	145	213	0.756	0.854	1.5	1.5	50%	57%
LEE CO WSD I	Sanford	1870	7166	0.179	0.574	2	2	9%	29%
SANFORD	Cape Fear River	21,608	33,000	8.18	10.3	12.6	12.6	65%	82%
MOORE									
CAMERON	Bedrock Wells	391	524	0.049	0.064	0.109	0.134	45%	48%
CARTHAGE	WTP Pond /Nick's Creek	2175	2400	0.3	0.49	0.5	0.5	60%	98%

HAR COL 2.4-4

Table 2.4.12-204 (Sheet 3 of 4)
1997 and 2010 Cape Fear River Population and Water Use as Reported by Local Water Supply Plan (LWSP) Systems

		Year-round Service Population		Average Daily Demand (mgd)		Available Supply (mgd)		Demand as % of Supply[b]	
Water Systems by County	**Water Source or Supplier**	**1997**	**2010**	**1997**	**2010**	**1997**	**2010**	**1997**	**2010**
MOORE CO (HYLAND HILLS - NIAGRA)	Bedrock Wells	267	277	0.021	0.0222	0.032	0.032	57%	69%
MOORE CO (PINEHURST)	Bedrock Wells/Southern Pines	7746	13,019	1.61	3.492	2.417	4.999	67%	70%
MOORE CO (SEVEN LAKES)	Bedrock Wells	2685	4163	0.314	0.479	0.341	0.773	92%	62%
MOORE CO (VASS)	Little River	736	1000	0.094	0.1255	1.45	1.45	6%	9%
ROBBINS	Bear Cr./Cabin Cr./Brooks Res.	1950	2074	0.822	0.826	1.5	1.5	55%	55%
NEW HANOVER									
APPLE VALLEY	Peedee, Castle Hayne, and Surficial Aquifers	199	254	0.122	0.158	0.166	0.166	73%	95%
BRICKSTONE - MARSH OAKS	Peedee, Castle Hayne, and Surficial Aquifers	535	683	0.059	0.076	0.216	0.216	27%	36%
CAROLINA BEACH	Castle Hayne and Surficial Aquifers	4643	5468	0.841	0.99	0.89	1.322	94%	75%
FIGURE EIGHT ISLAND	Peedee Aquifer	125	169	0.4	0.532	0.564	0.564	71%	94%
KURE BEACH	Surficial & Peedee Aquifers	1251	1518	0.493	0.598	0.396	0.396	124%	151%
LOWER CAPE FEAR WSA	Cape Fear River	0	0	41.15	51.15	50	50	82%	102%
MONTEREY HEIGHTS	Peedee, Castle Hayne, and Surficial Aquifers	1095	1325	0.101	0.117	0.242	0.242	42%	48%
MURRAYVILLE	Peedee, Castle Hayne, and Surficial Aquifers	7671	10548	1.212	1.67	2.916	2.916	42%	57%
NEW HANOVER CO AIRPORT	Wilmington	0	0	0.019	0.024	0.025	0.025	75%	95%
NEW HANOVER CO FLEMINGTON	Surficial Aquifer	187	239	0.283	0.32	0.432	0.432	66%	74%
PRINCE GEORGE	Peedee, Castle Hayne, and Surficial Aquifers	596	760	0.052	0.068	0.18	0.18	29%	38%
RUNNYMEADE	Peedee, Castle Hayne, and Surficial Aquifers	728	929	0.052	0.068	0.144	0.144	36%	47%
WALNUT HILLS	Peedee, Castle Hayne, and Surficial Aquifers	781	997	0.072	0.094	0.148	0.148	48%	63%
WESTBAY	Peedee, Castle Hayne, and Surficial Aquifers	644	822	0.039	0.051	0.648	0.648	6%	8%
WILMINGTON	LCFWSA / Cape Fear River	66,686	73,200	12.336	19.853	40.5	45.85	30%	43%
WRIGHTSVILLE BEACH	Surficial Aquifer	3146	3580	1.374	1.554	1.222	1.222	112%	127%
ONSLOW (in proposed Central Coastal Plain Capacity Use Area)									
HOLLY RIDGE[a]	ONSLOW CO	723	870	0.09	0.108	0.09	0.09	100%	120%
CAMP LEJEUNE – Combined	Castle Hayne and Surficial Aquifers / ONSLOW CO	68,700	68,700	6.547	6.547	15.582	15.582	42%	42%
JACKSONVILLE	Peedee & Black Creek Aquifers	32,489	38,175	4.01	4.503	3.448	3.448	117%	132%
NW ONSLOW WATER	Peedee Aquifer	1000	1137	0.085	0.108	0.216	0.216	39%	50%
ONSLOW CO	Black Creek, Peedee, Castle Hayne, and Surficial Aquifers	81,041	115,000	6.07	9.455	9.286	13.286	64%	70%
RICHLANDS	Black Creek Aquifer	1250	2048	0.174	0.212	0.324	0.324	54%	65%
ORANGE									
OWASA	University Lake / Cane Creek	65,000	80,300	8.978	11.693	10.4	20.4	86%	57%
PENDER									
BURGAW	Peedee and Black Creek Aquifers	3519	4682	0.449	0.65	0.81	0.81	55%	80%
SURF CITY	Peedee Aquifer	910	1162	0.407	0.488	0.63	0.936	65%	52%
TOPSAIL BEACH	Peedee Aquifer	450	650	0.324	0.459	0.497	0.497	65%	92%
RANDOLPH									
ARCHDALE	High Point / Davidson Ws	8500	15,000	0.564	1.359	1	2.75	56%	49%
FRANKLINVILLE	Ramseur	831	1200	0.047	0.065	0.09	0.09	52%	73%
LIBERTY	Bedrock Wells	2200	2598	0.297	0.3452	0.365	0.581	82%	59%
RAMSEUR	Sandy Creek	2524	2970	0.628	0.904	6.6	6.6	10%	14%
RANDLEMAN	Polecat Creek / ASHEBORO	3526	4398	1.226	1.51	2.5	3.5	49%	43%

HAR COL 2.4-4

Table 2.4.12-204 (Sheet 4 of 4)
1997 and 2010 Cape Fear River Population and Water Use as Reported by Local Water Supply Plan (LWSP) Systems

Water Systems by County	Water Source or Supplier	Year-round Service Population 1997	Year-round Service Population 2010	Average Daily Demand (mgd) 1997	Average Daily Demand (mgd) 2010	Available Supply (mgd) 1997	Available Supply (mgd) 2010	Demand as % of Supply[b] 1997	Demand as % of Supply[b] 2010
ROCKINGHAM									
REIDSVILLE	Troublesome Cr./Lake Reidsville	14,085	15,200	3.36	8.058	19	19	18%	42%
ROCKINGHAM CO	Reidsville	0	2082	0	0.176	0	0.55	0%	32%
SAMPSON									
AUTRYVILLE	Stedman	400	457	0.037	0.042	0.04	0.04	94%	104%
CLINTON	Black Creek, U Cape Fear, and L Cape Fear Aquifers	9211	11,461	1.836	2.962	2.221	3.034	83%	98%
GARLAND	Black Creek and Upper Cape Fear Aquifers	766	950	0.094	0.614	0.173	0.569	55%	108%
HARRELLS WC	Black Creek Aquifer	1134	1306	0.097	0.113	0.306	0.306	32%	37%
NEWTON GROVE	Black Creek Aquifer	614	820	0.091	0.146	0.244	0.244	37%	60%
ROSEBORO	Black Creek and Upper Cape Fear Aquifers	1617	1842	0.297	0.329	0.54	0.54	55%	62%
SALEMBURG	Surficial Aquifer	660	763	0.12	0.14	0.24	0.24	50%	58%
SAMPSON CO WSD I	Clinton / Roseboro / Turkey / Autryville	2988	3416	0.134	0.194	0.22	0.22	61%	88%
SAMPSON CO WSD II	Clinton / Dunn / Garland	0	7425	0	0.919	0	1.6	0%	57%
TURKEY	Upper Cape Fear Aquifer	290	340	0.071	0.071	0.432	0.432	16%	16%
WAKE									
APEX	Jordan Lake	12,000	58,398	1.795	5.58	3.68	9.2	49%	61%
CARY	Jordan Lake / Raleigh	82,700	120,900	11.99	11.77	15.82	18.57	76%	63%
FUQUAY-VARINA	Harnett Co / Garner	6249	18268	0.719	2.192	1.75	1.75	41%	125%
HOLLY SPRINGS	Apex	5492	35,000	0.518	4.3	0.75	5	69%	86%
MORRISVILLE	Cary	2200	18,700	0.46	3.8	1	3	46%	126%
WAYNE (in proposed Central Coastal Plain Capacity Use Area)									
MOUNT OLIVE	Upper Cape Fear Aquifer	6200	6755	1.183	1.432	1.872	1.872	63%	76%
WEST MOUNT OLIVE	Mount Olive	875	937	0.078	0.086	0.081	0.113	96%	76%

Notes:
a) 1997 LWSP not submitted -1992 data used in analysis.

b) "Demand as % of supply" is based on seasonal demands.

mgd = million gallons per day

HAR COL 2.4-4

Table 2.4.12-205 (Sheet 1 of 2)
Summary of Piezometer and Monitoring Well Construction Details

Well ID	Surficial or Bedrock Aquifer	Northing (NAD27)	Easting (NAD27)	Ground Elevation (feet NGVD29)	Top of Casing (TOC) Elevation (feet NGVD29)	Flush / Stick-up	Height from TOC to Ground Surface (feet)	Depth, Top of Screen (feet BTOC)	Depth, Bottom of Screen (feet BTOC)	Measured Total Depth[a] (feet BTOC)	Riser Material	Riser Diameter (inch)	Screen Length (feet)	Borehole Log/ Completion Form Available?	Date Installed
HAR Units 2 and 3 Monitoring Wells															
MWA-1S	Surficial	686565.2	2012706.8	263.70	266.31	Stick-up	2.6	12.3	17.3	17.5	Sch. 40 PVC	2	5	Y / Y	08/02/2006
MWA-1D	Bedrock	686572.8	2012703.0	263.88	266.22	Stick-up	2.3	38.6	68.6	68.8	Sch. 40 PVC	2	30	Y / Y	08/03/2006
MWA-2S	Surficial	686443.8	2011686.0	261.09	263.05	Stick-up	2.0	15.6	30.6	30.8	Sch. 40 PVC	2	15	Y / Y	07/31/2006
MWA-2D	Bedrock	686452.2	2011682.3	260.59	262.88	Stick-up	2.3	49.7	79.7	80.0	Sch. 40 PVC	2	30	Y / Y	08/01/2006
MWA-3S	Surficial	686910.2	2012316.3	266.30	268.67	Stick-up	2.4	9.9	14.9	15.1	Sch. 40 PVC	2	5	Y / Y	08/15/2006
MWA-3D	Bedrock	686907.7	2012307.5	266.29	268.46	Stick-up	2.2	34.6	64.6	64.8	Sch. 40 PVC	2	30	Y / Y	08/15/2006
MWA-4S	Surficial	687126.4	2012812.9	260.64	263.30	Stick-up	2.7	12.0	17.0	17.3	Sch. 40 PVC	2	5	Y / Y	07/20/2006
MWA-4D	Bedrock	687118.2	2012815.1	260.64	263.03	Stick-up	2.4	27.1	67.1	67.3	Sch. 40 PVC	2	40	Y / Y	07/20/2006
MWA-5S	Surficial	687189.6	2013000.5	261.92	264.15	Stick-up	2.2	19.8	34.8	35.1	Sch. 40 PVC	2	15	Y / Y	08/08/2006
MWA-5D	Bedrock	687195.6	2012997.0	261.92	264.25	Stick-up	2.3	44.8	84.8	85.1	Sch. 40 PVC	2	40	Y / Y	08/07/2006
MWA-6S	Surficial	687568.1	2013443.0	263.14	265.52	Stick-up	2.4	20.6	35.6	35.9	Sch. 40 PVC	2	15	Y / Y	08/08/2006
MWA-7S	Surficial	687499.6	2011203.7	287.23	290.29	Stick-up	3.1	15.7	30.7	30.9	Sch. 40 PVC	2	15	Y / Y	08/09/2006
MWA-7D	Bedrock	687503.7	2011213.5	287.07	290.04	Stick-up	3.0	50.7	80.7	81.0	Sch. 40 PVC	2	30	Y / Y	08/09/2006
MWA-8S	Surficial	687763.3	2011928.4	268.28	271.21	Stick-up	2.9	12.9	17.9	18.2	Sch. 40 PVC	2	5	Y / Y	08/17/2006
MWA-8D	Bedrock	687757.7	2011918.6	268.21	271.18	Stick-up	3.0	38.4	68.4	68.7	Sch. 40 PVC	2	30	Y / Y	08/16/2006
MWA-9S	Surficial	687996.6	2012453.3	246.92	249.78	Stick-up	2.9	8.7	13.7	14.0	Sch. 40 PVC	2	5	Y / Y	08/11/2006
MWA-9D	Bedrock	687998.1	2012446.3	246.91	249.94	Stick-up	3.0	30.6	60.6	60.8	Sch. 40 PVC	2	30	Y / Y	08/11/2006
MWA-10S	Surficial	688247.9	2011681.6	266.68	269.37	Stick-up	2.7	12.1	22.1	22.3	Sch. 40 PVC	2	10	Y / Y	08/14/2006
MWA-10D	Bedrock	688250.1	2011672.9	267.28	270.19	Stick-up	2.9	38.0	68.0	68.2	Sch. 40 PVC	2	30	Y / Y	08/14/2006
MWA-11S	Surficial	688614.8	2012942.5	239.95	242.65	Stick-up	2.7	12.3	17.3	17.6	Sch. 40 PVC	2	5	Y / Y	07/28/2006
Plant Area Wells and Piezometers															
LP-2	Bedrock	687233.8	2012113.0	258.43	260.80	Stick-up	2.4	----	----	120.7	Sch. 40 PVC	2	----	N / N	----
LP-5	Bedrock	684993.0	2011066.7	260.34	263.70	Stick-up	3.4	----	----	88.4	Sch. 40 PVC	2	----	N / N	----
LP-6	Bedrock	685482.4	2012163.2	261.27	264.24	Stick-up	3.0	----	----	115.3	Sch. 40 PVC	2	----	N / N	----
LP-7	Bedrock	686282.8	2013330.6	261.33	263.10	Stick-up	1.8	----	----	119.3	Sch. 40 PVC	2	----	N / N	----
LP-9	Bedrock	685625.1	2014893.6	254.37	257.88	Stick-up	3.5	----	----	119.4	Sch. 40 PVC	2	----	N / N	----
LP-13 / GW-57	Bedrock	683769.0	2012029.1	259.39	262.11	Stick-up	2.7	----	----	122.1	Sch. 40 PVC	2	----	N / N	----
LP-16	Bedrock	684624.4	2014268.2	259.23	261.20	Stick-up	2.0	----	----	134.7	Sch. 40 PVC	2	----	N / N	----
W-5	Bedrock	688035.3	2012589.4	244.46	245.04	Stick-up	0.6	----	----	193.0	Steel	6	----	N / N	----
W-5A	Bedrock	684191.1	2011069.5	264.76	266.82	Stick-up	2.1	----	----	251.7	Steel	6	----	N / N	----
W-8A / GW-58	Bedrock	683947.1	2010951.7	259.40	260.86	Stick-up	1.5	----	----	200.0	Steel	6	----	N / N	----
W-9A / GW-60	Bedrock	684463.4	2015452.1	231.31	233.42	Stick-up	2.1	----	----	180.5	Steel	6	----	N / N	----
W-12	Bedrock	683780.7	2012076.6	258.66	260.51	Stick-up	1.8	----	----	253.3	Steel	6	----	N / N	----
W-13 / GW-59	Bedrock	688101.6	2013527.7	246.91	250.52	Stick-up	3.6	----	----	312.0	Steel	6	----	N / N	----
W-14	Bedrock	686188.5	2010477.0	270.63	271.44	Stick-up	0.8	----	----	173.0	Steel	6	----	N / N	----
W-15	Bedrock	688321.5	2013374.1	239.85	241.72	Stick-up	1.9	----	----	127.2	Steel	6	----	N / N	----
WAD-1 / GW-39	Bedrock	681636.4	2011487.9	263.56	264.11	Stick-up	0.6	----	----	189.4	Steel	6	----	----	----
MWA-12	Surficial	684465.5	2013884.5	260.07	262.50	Stick-up	2.4	32.8	52.8	53.04	Sch. 40 PVC	2	20	Y / Y	08/22/2006

HAR COL 2.4-4

Table 2.4.12-205 (Sheet 2 of 2)
Summary of Piezometer and Monitoring Well Construction Details

Well ID	Surficial or Bedrock Aquifer	Northing (NAD27)	Easting (NAD27)	Ground Elevation (feet NGVD29)	Top of Casing (TOC) Elevation (feet NGVD29)	Flush / Stick-up	Height from TOC to Ground Surface (feet)	Depth, Top of Screen (feet BTOC)	Depth, Bottom of Screen (feet BTOC)	Measured Total Depth[a] (feet BTOC)	Riser Material	Riser Diameter (inch)	Screen Length (feet)	Borehole Log/ Completion Form Available?	Date Installed
							Landfill Area Monitoring Wells								
MW-1 (LF)	Bedrock	689656.5	2011798.3	276.96	278.75	Stick-up	1.8	87.0	97.0	96.8	Sch. 40 PVC	2	10.0	N / Y	11/14/1986
MW-2 (LF)	Bedrock	689005.2	2011447.7	270.10	272.59	Stick-up	2.5	30.0	40.0	41.6	Sch. 40 PVC	2	10.0	N / Y	11/18/1986
MW-3 (LF)	Bedrock	688886.7	2010621.5	278.43	280.63	Stick-up	2.2	47.0	57.0	58.7	Sch. 40 PVC	2	10.0	N / Y	11/24/1986
MW-6 (LF)	Bedrock	689887.7	2011953.3	254.62	256.82	Stick-up	2.2	53.5	68.5	70.9	Sch. 40 PVC	2	15.0	Y / Y	12/19/2003
MW-7 (LF)	Bedrock	689707.8	2011435.7	273.48	275.21	Stick-up	1.7	38.0	53.0	55.1	Sch. 40 PVC	2	15.0	Y / Y	12/19/2003
							Auxiliary Dam Area Piezometers								
ADP-1	Surficial	----	----	----	----	Stick-up	----	10.0	20	----	Sch. 40 PVC	1.5	10	N / Y	06/25/1981
ADP-2	Surficial	----	----	----	----	Stick-up	----	20.0	45	----	Sch. 40 PVC	1.5	25	N / Y	06/23/1981
ADP-3	Surficial	----	----	----	----	Stick-up	----	30.0	60	----	Sch. 40 PVC	1.5	30	N / Y	06/17/1981
ADP-4	Surficial	----	----	----	----	Stick-up	----	30.0	60	----	Sch. 40 PVC	1.5	30	N / N	----
ADP-5	Surficial	683844.5	2008733.8	260.17	263.41	Stick-up	3.2	22.0	45	47.1	Sch. 40 PVC	1.5	23	N / Y	06/04/1981
ADP-6	Surficial	683844.1	2009133.5	260.22	263.29	Stick-up	3.1	22.0	40	42.2	Sch. 40 PVC	1.5	18	N / Y	06/03/1981
ADP-7	Surficial	683844.8	2009533.6	260.12	263.66	Stick-up	3.5	10.0	20	23.2	Sch. 40 PVC	1.5	10	N / Y	06/17/1981
ADP-8	Surficial	----	----	----	----	Stick-up	----	45.0	65	----	Sch. 40 PVC	1.5	20	N / Y	06/25/1981
ADP-9	Surficial	----	----	----	----	Stick-up	----	45.0	65	----	Sch. 40 PVC	1.5	20	N / Y	06/23/1981
ADP-10	Bedrock	----	----	----	----	Stick-up	----	75.0	95	----	Sch. 40 PVC	1.5	20	N / Y	06/16/1981
ADP-11	Bedrock	----	----	----	----	Stick-up	----	75.0	95	----	Sch. 40 PVC	1.5	20	N / Y	06/11/1981
ADP-12	Bedrock	683843.9	2008743.7	259.95	263.27	Stick-up	3.3	70.0	90	91.6	Sch. 40 PVC	1.5	20	N / N	----
ADP-13	Bedrock	683844.3	2009543.8	260.34	263.59	Stick-up	3.2	40.0	60	51.9	Sch. 40 PVC	1.5	20	N / N	----
ADP-14	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/14/1980
ADP-15	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/13/1980
ADP-16	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/20/1980
ADP-17	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/20/1980
ADP-18	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/21/1980
ADP-19	Bedrock	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	11/26/1980
ADP-20	Both	----	----	----	----	Stick-up	----	20.0	40	----	Sch. 40 PVC	1.5	20	N / Y	06/30/1981
ADP-21	----	----	----	----	----	Stick-up	----	15.0	35	----	Sch. 40 PVC	1.5	20	N / N	----
ADP-23	Bedrock	683753.7	2009696.2	254.15	257.51	Stick-up	3.4	15.0	35	36.6	Sch. 40 PVC	1.5	20	N / Y	11/07/1980
ADP-21A	Both	683610.0	2009086.4	238.12	241.27	Stick-up	3.1	----	----	47.5	Sch. 40 PVC	1.5	----	N / Y	06/30/1981
Sludge Application Wells															
MW-1 (SA)	----	685928.1	2011307.4	260.02	261.51	Stick-up	1.5	8.5	23.5	23.8	Sch. 40 PVC	2	15	N / Y	05/31/1995
MW-2 (SA)	----	685234.8	2011112.9	260.55	263.01	Stick-up	2.5	20.0	35	37.5	Sch. 40 PVC	2	15	Y / Y	05/26/1995
MW-3 (SA)	----	684375.1	2011566.1	259.45	262.23	Stick-up	2.8	20	35	37.3	Sch. 40 PVC	2	15	N / Y	05/26/1995

Notes:
a) Measured in the field on August 28, 2006, by CH2M HILL personnel.

--- = no data available
BTOC = below top of casing

HAR COL 2.4-4

Table 2.4.12-206 (Sheet 1 of 3)
Summary of Groundwater Levels within the Plant Site

Well Identification	Ground Elevation (feet NGVD29)	Top of Casing (TOC) Elevation (feet NGVD29)	Groundwater Surface Elevation				
			June 6/7, 2006	August 28, 2006	November 27, 2006	February 28, 2007	May 28, 2007
HAR Units 2 and 3 Monitoring Wells							
MWA-1S	263.70	266.31	NA	253.31	253.08	254.03	255.09
MWA-1D	263.88	266.22	NA	222.05[(a)]	250.38	252.43	253.40
MWA-2S	261.09	263.05	NA	253.06	258.17	258.65	254.11
MWA-2D	260.59	262.88	NA	252.12	251.68	251.78	251.89
MWA-3S	266.30	268.67	NA	257.12	256.60	260.19	258.03
MWA-3D	266.29	268.46	NA	253.86	256.21	256.67	254.53
MWA-4S	260.64	263.30	NA	253.76	254.07	256.45	254.85
MWA-4D	260.64	263.03	NA	255.62	256.93	258.62	256.42
MWA-5S	261.92	264.15	NA	250.06	253.37	254.81	252.11
MWA-5D	261.92	264.25	NA	249.80	252.47	253.14	251.56
MWA-6S	263.14	265.52	NA	238.61	238.62	238.29	237.95
MWA-7S	287.23	290.29	NA	270.41	268.99	269.95	271.13
MWA-7D	287.07	290.04	NA	268.29	268.09	269.66	269.74
MWA-8S	268.28	271.21	NA	257.08	256.06	255.45	257.06
MWA-8D	268.21	271.18	NA	248.04	249.28	251.50	249.28
MWA-9S	246.92	249.78	NA	241.57	244.09	244.65	243.51
MWA-9D	246.91	249.94	NA	247.98	247.70	249.38	248.59
MWA-10S	266.68	269.37	NA	251.36	251.18	260.00	258.99
MWA-10D	267.28	270.19	NA	252.28	252.09	254.43	252.73
MWA-11S	239.95	242.65	NA	227.11	225.43	232.69	231.46
Plant Area Wells and Piezometers							
LP-2	258.43	260.80	257.40	254.65	258.88	258.70	255.46
LP-5	260.34	263.70	248.94	246.18	246.88	252.39	249.03
LP-6	261.27	264.24	245.24	246.31	246.70	246.42	246.12

HAR COL 2.4-4

Table 2.4.12-206 (Sheet 2 of 3)
Summary of Groundwater Levels within the Plant Site

Well Identification	Ground Elevation (feet NGVD29)	Top of Casing (TOC) Elevation (feet NGVD29)	Groundwater Surface Elevation				
			June 6/7, 2006	August 28, 2006	November 27, 2006	February 28, 2007	May 28, 2007
LP-7	261.33	263.10	244.20	246.44	245.74	245.52	245.91
LP-9	254.37	257.88	223.76	223.18	224.81	223.87	223.44
LP-13	259.39	262.11	238.61	234.30	237.39	239.41	238.30
LP-16	259.23	261.20	226.20	227.07	227.64	227.33	226.96
W-5	244.46	245.04	243.41	242.07	243.33	243.94	242.71
W-5A	264.76	266.82	240.92	239.57	239.63	241.97	241.01
W-8A	259.40	260.86	239.91	238.42	238.47	240.89	239.87
W-9A	231.31	233.42	220.17	219.32	220.91	220.43	219.79
W-12	258.66	260.51	238.51	237.17	237.30	239.27	238.18
W-13	246.91	250.52	226.92	226.00	226.40	227.78	227.44
W-14	270.63	271.44	252.61	251.44	251.82	253.04	252.16
W-15	239.85	241.72	227.92	Not Located	227.46	229.80	228.88
WAD-1	263.56	264.11	235.46	234.04	234.45	236.41	235.42
MWA-12	260.07	262.50	Not Installed	235.94	236.50	235.62	234.80
Landfill Area Monitoring Wells							
MW-1	276.96	278.75	250.80	249.11	250.02	254.28	252.58
MW-2	270.10	272.59	260.62	259.75	260.12	260.90	259.99
MW-3	278.43	280.63	260.95	260.07	259.81	261.66	260.70
MW-6	254.62	256.82	250.01	248.41	249.26	253.55	251.80
MW-7	273.48	275.21	>275.21[b]	275.16	274.71	>275.21[b]	N/A
Auxiliary Dam Area Piezometers							
ADP-5	260.17	263.41	236.79	236.73	237.39	237.45	236.53
ADP-6	260.22	263.29	235.26	235.24	236.09	236.16	235.04
ADP-7	260.12	263.66	242.86	246.39	246.36	243.58	242.36
ADP-12	259.95	263.27	224.71	224.24	225.21	224.73	224.42

HAR COL 2.4-4

Table 2.4.12-206 (Sheet 3 of 3)
Summary of Groundwater Levels within the Plant Site

Well Identification	Ground Elevation (feet NGVD29)	Top of Casing (TOC) Elevation (feet NGVD29)	Groundwater Surface Elevation				
			June 6/7, 2006	August 28, 2006	November 27, 2006	February 28, 2007	May 28, 2007
ADP-13	260.34	263.59	238.42	238.66	237.58	237.62	237.88
ADP-23	254.15	257.51	236.81	236.89	235.60	235.91	236.51
ADP-21A	238.12	241.27	218.12	218.81	219.25	219.44	218.48
Sludge Application Wells							
MW-1 (SA)	260.02	261.51	248.96	246.49	247.01	250.93	248.61
MW-2 (SA)	260.55	263.01	249.46	247.15	246.21	250.99	249.28
MW-3 (SA)	259.45	262.23	240.38	240.34	240.31	242.67	241.65

Notes:
Elevation units are feet NGVD29.

a) MWA-1D water level elevation as measured on August 28, 2006 is incorrect due to influences of prior development activities.

b) Water level in MW-7 exceeded the top of casing and therefore created artesian conditions.

HAR COL 2.4-4

Table 2.4.12-207 (Sheet 1 of 3)
Summary of Groundwater Vertical Gradients within the HAR Site

Well Identification	Top of Casing (TOC) Elevation (feet NGVD29)	Depth to Well Screen (feet BTOC)	Screen Length (feet)	Depth to Water (feet BTOC)	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)	
					(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)	(feet/feet)	(up/down)
August 28, 2006														
MWA-1S	266.31	12.26	5	13.00	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
MWA-1D	266.22	38.55	30	--- [a]										
MWA-2S	263.05	15.55	15	9.99	0.01	Down	0.03	Down	0.02	Down	0.02	Down	0.05	Down
MWA-2D	262.88	49.70	30	10.76										
MWA-3S	268.67	9.86	5	11.55	0.06	Down	0.14	Down	0.09	Down	0.07	Down	0.16	Down
MWA-3D	268.46	34.59	30	14.60										
MWA-4S	263.30	12.01	5	9.54	0.03	Up	0.12	Up	0.06	Up	0.04	Up	0.18	Up
MWA-4D	263.03	27.08	40	7.41										
MWA-5S	264.15	19.80	15	14.09	0.004	Down	0.01	Down	0.01	Down	0.01	Down	0.03	Down
MWA-5D	264.25	44.81	40	14.45										
MWA-7S	290.29	15.65	15	19.88	0.03	Down	0.07	Down	0.05	Down	0.04	Down	0.10	Down
MWA-7D	290.04	50.71	30	21.75										
MWA-8S	271.21	12.92	5	14.13	0.17	Down	0.37	Down	0.24	Down	0.18	Down	0.44	Down
MWA-8D	271.18	38.40	30	23.14										
MWA-9S	249.78	8.74	5	8.21	0.12	Up	0.30	Up	0.19	Up	0.14	Up	0.38	Up
MWA-9D	249.94	30.56	30	1.96										
MWA-10S	269.37	12.08	10	18.01	0.02	Up	0.05	Up	0.03	Up	0.02	Up	0.06	Up
MWA-10D	270.19	37.95	30	17.91										
November 27, 2006														
MWA-1S	266.31	12.26	5	13.23	0.05	Down	0.11	Down	0.07	Down	0.05	Down	0.13	Down
MWA-1D	266.22	38.55	30	15.84										
MWA-2S	263.05	15.55	15	4.88	0.10	Down	0.19	Down	0.16	Down	0.13	Down	0.34	Down
MWA-2D	262.88	49.70	30	11.20										
MWA-3S	268.67	9.86	5	12.07	0.01	Down	0.02	Down	0.01	Down	0.01	Down	0.02	Down
MWA-3D	268.46	34.59	30	12.25										
MWA-4S	263.30	12.01	5	9.23	0.05	Up	0.19	Up	0.09	Up	0.06	Up	0.28	Up
MWA-4D	263.03	27.08	40	6.10										
MWA-5S	264.15	19.80	15	10.78	0.01	Down	0.04	Down	0.02	Down	0.02	Down	0.09	Down
MWA-5D	264.25	44.81	40	11.78										
MWA-7S	290.29	15.65	15	21.30	0.02	Down	0.03	Down	0.02	Down	0.02	Down	0.04	Down
MWA-7D	290.04	50.71	30	21.95										
MWA-8S	271.21	12.92	5	15.15	0.13	Down	0.29	Down	0.18	Down	0.13	Down	0.33	Down
MWA-8D	271.18	38.40	30	21.90										
MWA-9S	249.78	8.74	5	5.69	0.07	Up	0.17	Up	0.11	Up	0.08	Up	0.22	Up
MWA-9D	249.94	30.56	30	2.24										
MWA-10S	269.37	12.08	10	18.19	0.02	Up	0.05	Up	0.03	Up	0.02	Up	0.06	Up
MWA-10D	270.19	37.95	30	18.10										

HAR COL 2.4-4

Table 2.4.12-207 (Sheet 2 of 3)
Summary of Groundwater Vertical Gradients within the HAR Site

Well Identification	Top of Casing (TOC) Elevation (feet NGVD29)	Depth to Well Screen (feet BTOC)	Screen Length (feet)	Depth to Water (feet BTOC)	Bottom of Screen to Top of Screen (L:H) (feet/feet)	Bottom of Screen to Top of Screen (L:H) (up/down)	Top of Screen to Top of Screen (H:H) (feet/feet)	Top of Screen to Top of Screen (H:H) (up/down)	Mid-point of Screen to Mid-point of Screen (M:M) (feet/feet)	Mid-point of Screen to Mid-point of Screen (M:M) (up/down)	Bottom of Screen to Bottom of Screen (L:L) (feet/feet)	Bottom of Screen to Bottom of Screen (L:L) (up/down)	Top of Screen to Bottom of Screen (H:L) (feet/feet)	Top of Screen to Bottom of Screen (H:L) (up/down)
February 28, 2007														
MWA-1S	266.31	12.26	5	12.28	0.03	Down	0.06	Down	0.04	Down	0.03	Down	0.07	Down
MWA-1D	266.22	38.55	30	13.79										
MWA-2S	263.05	15.55	15	4.40	0.11	Down	0.20	Down	0.16	Down	0.14	Down	0.36	Down
MWA-2D	262.88	49.70	30	11.10										
MWA-3S	268.67	9.86	5	8.48	0.06	Down	0.14	Down	0.09	Down	0.07	Down	0.18	Down
MWA-3D	268.46	34.59	30	11.79										
MWA-4S	263.30	12.01	5	6.85	0.04	Up	0.14	Up	0.07	Up	0.04	Up	0.21	Up
MWA-4D	263.03	27.08	40	4.41										
MWA-5S	264.15	19.80	15	9.34	0.03	Down	0.07	Down	0.04	Down	0.03	Down	0.17	Down
MWA-5D	264.25	44.81	40	11.11										
MWA-7S	290.29	15.65	15	20.34	0.005	Down	0.009	Down	0.007	Down	0.006	Down	0.01	Down
MWA-7D	290.04	50.71	30	20.38										
MWA-8S	271.21	12.92	5	15.76	0.08	Down	0.17	Down	0.11	Down	0.08	Down	0.19	Down
MWA-8D	271.18	38.40	30	19.68										
MWA-9S	249.78	8.74	5	5.13	0.09	Up	0.22	Up	0.14	Up	0.10	Up	0.28	Up
MWA-9D	249.94	30.56	30	0.56										
MWA-10S	269.37	12.08	10	9.37	0.10	Down	0.22	Down	0.16	Down	0.12	Down	0.37	Down
MWA-10D	270.19	37.95	30	15.76										

HAR COL 2.4-4

Table 2.4.12-207 (Sheet 3 of 3)
Summary of Groundwater Vertical Gradients within the HAR Site

Well Identification	Top of Casing (TOC) Elevation (feet NGVD29)	Depth to Well Screen (feet BTOC)	Screen Length (feet)	Depth to Water (feet BTOC)	Bottom of Screen to Top of Screen (L:H) (feet/feet)	Bottom of Screen to Top of Screen (L:H) (up/down)	Top of Screen to Top of Screen (H:H) (feet/feet)	Top of Screen to Top of Screen (H:H) (up/down)	Mid-point of Screen to Mid-point of Screen (M:M) (feet/feet)	Mid-point of Screen to Mid-point of Screen (M:M) (up/down)	Bottom of Screen to Bottom of Screen (L:L) (feet/feet)	Bottom of Screen to Bottom of Screen (L:L) (up/down)	Top of Screen to Bottom of Screen (H:L) (feet/feet)	Top of Screen to Bottom of Screen (H:L) (up/down)
May 27, 2007														
MWA-1S	266.31	12.26	5	11.2	0.03	Down	0.06	Down	0.04	Down	0.03	Down	0.08	Down
MWA-1D	266.22	38.55	30	12.82										
MWA-2S	263.05	15.55	15	8.94	0.03	Down	0.06	Down	0.05	Down	0.05	Down	0.11	Down
MWA-2D	262.88	49.70	30	10.99										
MWA-3S	268.67	9.86	5	10.64	0.06	Down	0.14	Down	0.09	Down	0.07	Down	0.18	Down
MWA-3D	268.46	34.59	30	13.93										
MWA-4S	263.30	12.01	5	8.45	0.03	Up	0.10	Up	0.05	Up	0.03	Up	0.15	Up
MWA-4D	263.03	27.08	40	6.61										
MWA-5S	264.15	19.80	15	12.04	0.01	Down	0.02	Down	0.01	Down	0.01	Down	0.06	Down
MWA-5D	264.25	44.81	40	12.69										
MWA-7S	290.29	15.65	15	19.16	0.02	Down	0.04	Down	0.03	Down	0.03	Down	0.07	Down
MWA-7D	290.04	50.71	30	20.30										
MWA-8S	271.21	12.92	5	14.15	0.14	Down	0.32	Down	0.21	Down	0.15	Down	0.38	Down
MWA-8D	271.18	38.40	30	21.90										
MWA-9S	249.78	8.74	5	6.27	0.10	Up	0.23	Up	0.15	Up	0.11	Up	0.30	Up
MWA-9D	249.94	30.56	30	1.35										
MWA-10S	269.37	12.08	10	10.38	0.11	Down	0.25	Down	0.18	Down	0.14	Down	0.42	Down
MWA-10D	270.19	37.95	30	17.46										

Note:
a) MWA-1D water level elevation as measured on August 28, 2006 is incorrect due to influences of prior development activities.

Source: Reference 2.4-253

HAR COL 2.4-4

Table 2.4.12-208
Slug Test Results Data Reduction

	Hydraulic Conductivity (ft/day) Min.	Max.	Average
Shallow Monitoring Wells:	**0.1**	**5.4**	**2.1**
Bedrock Monitoring Wells:	**0.002**	**0.8**	**0.1**

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)	Calculated Aquifer Thickness[d] (ft.)	Is water level in the well screen?	Hydraulic Conductivity [e],[f] (cm/sec)	Hydraulic Conductivity (ft/day)
MWA-1S	Out	*Fully*	0.17	0.50	12.3	17.26	17.51	12.60	*4.7*	Yes	8.0E-04	2.3
MWA-1D[g]	Out	*Partially*	0.17	0.50	38.6	68.55	68.80	34.79	*33.8*	No	3.8E-06	0.01
MWA-2S	Out	*Partially*	0.17	0.50	15.6	30.55	30.80	9.85	*20.7*	No	1.9E-03	5.4
MWA-2D	Out	*Partially*	0.17	0.50	49.7	79.70	79.95	11.08	*68.6*	No	8.6E-07	0.002
MWA-3S	Out	*Fully*	0.17	0.50	9.9	14.86	15.11	11.94	*2.9*	Yes	4.1E-04	1.2
MWA-3D	Out	*Partially*	0.17	0.50	34.6	64.59	64.84	14.23	*50.4*	No	3.9E-06	0.01
MWA-4S	Out	*Partially*	0.17	0.44	12.0	17.01	17.26	9.44	*7.6*	No	2.0E-04	0.6
MWA-4D	Out	*Partially*	0.17	0.44	27.1	67.08	67.33	8.50	*58.6*	No	7.7E-06	0.02
MWA-5S	Out	*Partially*	0.17	0.50	19.8	34.80	35.05	14.08	*20.7*	No	5.1E-05	0.1
MWA-5D	Out	*Partially*	0.17	0.50	44.8	84.81	85.06	13.93	*70.9*	No	3.0E-04	0.8
MWA-6S	Out	*Fully*	0.17	0.50	20.6	35.63	35.88	27.00	*8.6*	Yes	8.1E-04	2.3
MWA-7S	Out	*Fully*	0.17	0.50	15.7	30.65	30.90	21.80	*8.9*	Yes	5.2E-04	1.5
MWA-8D	Out	*Partially*	0.17	0.50	38.4	68.40	68.65	23.25	*45.2*	No	6.1E-05	0.2
MWA-9D	Out	*Partially*	0.17	0.50	30.6	60.56	60.81	0.00	*60.6*	No	2.8E-05	0.08
MWA-10S	Out	*Fully*	0.17	0.50	12.1	22.08	22.33	17.59	*4.5*	Yes	1.3E-03	3.6
MWA-10D	Out	*Partially*	0.17	0.50	38.0	67.95	68.20	18.35	*49.6*	No	7.4E-06	0.02

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 September 11, 2006, through September 13, 2006.
d) Software uses value of Aquifer Thickness = depth to bottom of screen - depth to static water level.
e) Pressure heads were measured using a MiniTroll Pro, manufactured by In-Situ Inc.
f) AquiferWin32 software (developed by Environmental Simulations, Inc., Version 3, 1999) and the Bouwer & Rice, 1976 method were used.
g) MWA-1D was not fully recharged since completion of development activities.

HAR COL 2.4-4

Table 2.4.12-209
Groundwater Linear Flow Velocity

*Seepage Velocity[a] [v_x] = (Hydraulic Conductivity [K] * Hydraulic Gradient [dH/dL])/Effective Porosity [n_e])*

*Darcy Flux[b] [Q] = Hydraulic Conductivity [K] * Hydraulic Gradient [dH/dL] * Cross-sectional Area [A]*

Monitoring Wells			Hydraulic Conduct-ivity[c] [*K*] (ft/day)	Water Level Gauging Date	Water Level - Up Gradient Well (feet NGVD29)	Water Level - Down Gradient Well (feet NGVD29)	Water Level Change [*dH*] (ft.)	Distance Between Wells [*dL*] (ft.)	Hydraulic Gradient [*dH/dL*] (feet/feet)	Effective Porosity[d] [n_e]	Seepage Velocity [v_x] (ft/day)	Cross-Sectional Area (ft.^{2})	Darcy Flux (ft^3/ft^2/day)
Surficial Aquifer													
MWA-3S	to	MWA-6S	5.4	28-Aug-06	257.12	238.61	18.51	1305	0.0142	0.1	**0.8**	1	**7.7E-02**
MWA-3S	to	MWA-5S	5.4	28-Aug-06	257.12	250.06	7.06	740	0.0095	0.1	**0.5**	1	**5.2E-02**
MWA-7S	to	MWA-9S	5.4	28-Aug-06	270.41	241.57	28.84	1346	0.0214	0.1	**1.2**	1	**1.2E-01**
MWA-8S	to	MWA-9S	5.4	28-Aug-06	257.08	241.57	15.51	574	0.0270	0.1	**1.5**	1	**1.5E-01**
Bedrock Aquifer													
MWA-3D	to	MWA-5D	0.8	28-Aug-06	253.86	249.80	4.06	748	0.0054	0.05	**0.09**	1	**4.6E-03**
MWA-7D	to	MWA-9D	0.8	28-Aug-06	268.29	247.98	20.31	1329	0.0153	0.05	**0.3**	1	**1.3E-02**
MWA-8D	to	MWA-9D	0.8	28-Aug-06	248.04	247.98	0.06	581	0.0001	0.05	**0.002**	1	**8.7E-05**

Notes:

a) Equation from: Reference 2.4-254, Page 145.

b) Equation from: Reference 2.4-255, Pages 16 and 17.

c) Hydraulic conductivity estimates are maximum values derived from Table 2.4.12-208, Slug Test Results Data Reduction.

d) Effective porosity representative values from: Reference 2.4-252, Page 16.4 and Reference 2.4-238, Table 4-4.

HAR COL 2.4-5

Table 2.4.13-201 (Sheet 1 of 2)
Effluent Tank Inventory and 10 CFR 20 Effective Concentration Limits

Nuclide	RC Activity, µCi/g	Hold-Up 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
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

HAR COL 2.4-5

Table 2.4.13-201 (Sheet 2 of 2)
Effluent Tank Inventory and 10 CFR 20 Effective Concentration Limits

Nuclide	RC Activity, µCi/g	Hold-Up Tank Activity, $\mu Ci/cm^3$	Half Life, Days	Decay Constant, $Days^{-1}$	ECL, $\mu Ci/cm^3$
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
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:
Half lives from Kennedy and Strange, 1992; Reference 2.4-256 Table E.1, except Br-85 and Sr-92 from Kocher, 1981; Reference 2.4-257.

ECLs from 10 CFR 20, Appendix B, Table 2, Column 2.

Equilibrium daughters Ba-137m, Rh-106 are included in ECLs for Cs-137, Ru-106.

$\mu Ci/cm^3$ = microcuries per cubic centimeter.
NA = not available

HAR COL 2.4-5

Table 2.4.13-202
Groundwater Parameters

Parameter	Surficial Aquifer	Bedrock Aquifer
Hydraulic conductivity, K	5.4 ft/day	0.8 ft/day
Effective Porosity, n_e	0.1	0.05
Head gradient, dh/dl	0.027 ft/ft	0.015 ft/ft
Seepage velocity, U	1.5 ft/day	0.3 ft/day
Bulk density	1.5 g/cm^3	2.6 g/cm^3
Dispersivity	1 m	1 m
Cs distribution coefficient, Kd	30 ml/g	10 ml/g
Sr distribution coefficient, Kd	10 ml/g	2 ml/g
Other nuclides distribution coefficient, Kd	0 ml/g	0 ml/g
Cs transport time	1000 yr	57,000 yr
Sr transport time	340 yr	12,000 yr
Other nuclides transport time	2.3 yr	110 yr

Notes:
Data from Table 2.4.12-201 and Table 2.4.12-208.

Bulk density for bedrock from HNP FSAR Table 2.5.4-1.

Transport time included retardation effect.

HAR COL 2.4-5

Table 2.4.13-203
Main Reservoir Groundwater Transport of Surficial Aquifer 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	7.4E-06	7.5E-06	7.5E-03	Y-90	2.4E-101	1.5E-106	2.2E-101
Br-83	0.0E+00	0.0E+00	0.0E+00	Y-91m	0.0E+00	0.0E+00	0.0E+00
Br-84	0.0E+00	0.0E+00	0.0E+00	Y-91	1.5E-11	1.1E-15	1.3E-10
Br-85	0.0E+00	0.0E+00	0.0E+00	Y-92	0.0E+00	0.0E+00	0.0E+00
I-129	1.4E-05	1.0E-13	5.2E-07	Y-93	0.0E+00	0.0E+00	0.0E+00
I-130	0.0E+00	0.0E+00	0.0E+00	Zr-95	3.9E-11	3.0E-15	1.5E-10
I-131	6.0E-39	2.1E-39	2.1E-33	Nb-95	1.5E-14	1.2E-18	3.8E-14
I-132	0.0E+00	0.0E+00	0.0E+00	Mo-99	1.2E-98	1.3E-99	6.3E-95
I-133	1.3E-294	8.2E-295	1.2E-289	Tc-99m	0.0E+00	0.0E+00	0.0E+00
I-134	0.0E+00	0.0E+00	0.0E+00	Ru-103	9.0E-14	6.1E-18	2.0E-13
I-135	0.0E+00	0.0E+00	0.0E+00	Rh-103m	0.0E+00	0.0E+00	0.0E+00
Cs-134	3.5E-157	1.2E-157	1.3E-151	Rh-106	0.0E+00	0.0E+00	0.0E+00
Cs-136	0.0E+00	0.0E+00	0.0E+00	Ag-110m	1.1E-07	2.1E-11	3.5E-06
Cs-137	1.8E-18	4.3E-19	4.3E-13	Te-127m	2.6E-09	9.5E-13	1.1E-07
Cs-138	0.0E+00	0.0E+00	0.0E+00	Te-129m	6.6E-15	8.3E-18	1.2E-12
Cr-51	1.5E-16	1.9E-19	3.8E-16	Te-129	0.0E+00	0.0E+00	0.0E+00
Mn-54	2.1E-07	1.4E-10	4.8E-06	Te-131m	5.6E-207	1.8E-209	2.3E-204
Mn-56	0.0E+00	0.0E+00	0.0E+00	Te-131	0.0E+00	0.0E+00	0.0E+00
Fe-55	1.9E-06	9.7E-10	9.7E-06	Te-132	1.8E-84	6.8E-86	7.6E-81
Fe-59	5.5E-13	7.2E-17	7.2E-12	Te-134	0.0E+00	0.0E+00	0.0E+00
Co-58	1.0E-10	1.9E-13	9.7E-09	Ba-137m	0.0E+00	0.0E+00	0.0E+00
Co-60	4.1E-06	9.0E-10	3.0E-04	Ba-140	1.9E-27	9.1E-31	1.1E-25
Rb-88	0.0E+00	0.0E+00	0.0E+00	La-140	3.6E-156	5.4E-160	6.0E-155
Rb-89	0.0E+00	0.0E+00	0.0E+00	Ce-141	3.6E-15	2.8E-19	9.2E-15
Sr-89	0.0E+00	0.0E+00	0.0E+00	Ce-143	2.8E-188	1.9E-192	9.4E-188
Sr-90	2.2E-11	5.2E-16	1.0E-09	Pr-143	3.9E-26	2.9E-30	1.4E-25
Sr-91	0.0E+00	0.0E+00	0.0E+00	Ce-144	1.6E-07	9.3E-12	3.1E-06
Sr-92	0.0E+00	0.0E+00	0.0E+00	Pr-144	0.0E+00	0.0E+00	0.0E+00

Notes:
∑(Max conc_i / ECL_i) at spillway = 0.8%

∑(Max conc_i / ECL_i) at Lillington = 0.005%

Note: Lillington effective dose equivalent is additionally diluted by Cape Fear River flow.

HAR COL 2.4-5

Table 2.4.13-204
Main Reservoir at Thomas Creek
Groundwater Transport Analysis of Surficial Aquifer
with Comparisons to 10 CFR 20 Effective Concentration Limits

Nuclide	Dilution Factor	Conc uCi/cm^3	Conc / ECL_i	Nuclide	Dilution Factor	Conc uCi/cm^3	Conc / ECL_i
H-3	1.3E-04	1.3E-04	1.3E-01	Y-90	2.8E-97	1.8E-102	2.6E-97
Br-83	0.0E+00	0.0E+00	0.0E+00	Y-91m	0.0E+00	0.0E+00	0.0E+00
Br-84	0.0E+00	0.0E+00	0.0E+00	Y-91	8.4E-09	5.7E-13	7.1E-08
Br-85	0.0E+00	0.0E+00	0.0E+00	Y-92	0.0E+00	0.0E+00	0.0E+00
I-129	1.4E-04	1.0E-12	5.2E-06	Y-93	0.0E+00	0.0E+00	0.0E+00
I-130	0.0E+00	0.0E+00	0.0E+00	Zr-95	1.9E-08	1.5E-12	7.5E-08
I-131	2.3E-35	8.1E-36	8.1E-30	Nb-95	1.3E-11	1.0E-15	3.4E-11
I-132	0.0E+00	0.0E+00	0.0E+00	Mo-99	1.4E-94	1.4E-95	7.2E-91
I-133	4.7E-290	3.0E-290	4.2E-285	Tc-99m	0.0E+00	0.0E+00	0.0E+00
I-134	0.0E+00	0.0E+00	0.0E+00	Ru-103	7.2E-11	4.9E-15	1.6E-10
I-135	0.0E+00	0.0E+00	0.0E+00	Rh-103m	0.0E+00	0.0E+00	0.0E+00
Cs-134	1.8E-155	6.0E-156	6.7E-150	Rh-106	0.0E+00	0.0E+00	0.0E+00
Cs-136	0.0E+00	0.0E+00	0.0E+00	Ag-110m	1.5E-05	2.8E-09	4.7E-04
Cs-137	2.3E-17	5.5E-18	5.5E-12	Te-127m	7.7E-07	2.8E-10	3.1E-05
Cs-138	0.0E+00	0.0E+00	0.0E+00	Te-129m	6.1E-12	7.7E-15	1.1E-09
Cr-51	1.7E-13	2.2E-16	4.4E-13	Te-129	0.0E+00	0.0E+00	0.0E+00
Mn-54	2.3E-05	1.6E-08	5.2E-04	Te-131m	1.4E-202	4.5E-205	5.7E-200
Mn-56	0.0E+00	0.0E+00	0.0E+00	Te-131	0.0E+00	0.0E+00	0.0E+00
Fe-55	8.0E-05	4.0E-08	4.0E-04	Te-132	1.7E-80	6.5E-82	7.2E-77
Fe-59	3.9E-10	5.1E-14	5.1E-09	Te-134	0.0E+00	0.0E+00	0.0E+00
Co-58	4.6E-08	8.7E-11	4.4E-06	Ba-137m	0.0E+00	0.0E+00	0.0E+00
Co-60	1.1E-04	2.4E-08	7.9E-03	Ba-140	4.6E-24	2.2E-27	2.8E-22
Rb-88	0.0E+00	0.0E+00	0.0E+00	La-140	6.7E-152	1.0E-155	1.1E-150
Rb-89	0.0E+00	0.0E+00	0.0E+00	Ce-141	3.5E-12	2.7E-16	8.9E-12
Sr-89	0.0E+00	0.0E+00	0.0E+00	Ce-143	6.3E-184	4.3E-188	2.1E-183
Sr-90	2.8E-10	6.7E-15	1.3E-08	Pr-143	9.0E-23	6.6E-27	3.3E-22
Sr-91	0.0E+00	0.0E+00	0.0E+00	Ce-144	1.9E-05	1.1E-09	3.7E-04
Sr-92	0.0E+00	0.0E+00	0.0E+00	Pr-144	0.0E+00	0.0E+00	0.0E+00

Note:
∑(Max conc_i / ECL_i) at Thomas Creek Branch = 14%

HAR COL 2.4-5

Table 2.4.13-205
Groundwater Transport from Bedrock Aquifer to Public Use Well with Comparisons to 10 CFR 20 Effective Concentration Limits

Nuclide	Dilution Factor	Conc./ $\mu Ci/cm^3$	Conc./ ECL	Nuclide	Dilution Factor	Conc / $\mu Ci/cm^3$	Conc / ECL
H-3	3.8E-06	3.80E-06	3.8E-03	Y-90	0.0E+00	0.0E+00	0.0E+00
Br-83	0.0E+00	0.0E+00	0.0E+00	Y-91m	0.0E+00	0.0E+00	0.0E+00
Br-84	0.0E+00	0.0E+00	0.0E+00	Y-91	2.6E-209	1.8E-213	2.2E-208
Br-85	0.0E+00	0.0E+00	0.0E+00	Y-92	0.0E+00	0.0E+00	0.0E+00
I-129	1.8E-03	1.3E-11	6.4E-05	Y-93	0.0E+00	0.0E+00	0.0E+00
I-130	0.0E+00	0.0E+00	0.0E+00	Zr-95	1.3E-191	9.8E-196	4.9E-191
I-131	0.0E+00	0.0E+00	0.0E+00	Nb-95	0.0E+00	0.0E+00	0.0E+00
I-132	0.0E+00	0.0E+00	0.0E+00	Mo-99	0.0E+00	0.0E+00	0.0E+00
I-133	0.0E+00	0.0E+00	0.0E+00	Tc-99m	0.0E+00	0.0E+00	0.0E+00
I-134	0.0E+00	0.0E+00	0.0E+00	Ru-103	0.0E+00	0.0E+00	0.0E+00
I-135	0.0E+00	0.0E+00	0.0E+00	Rh-103m	0.0E+00	0.0E+00	0.0E+00
Cs-134	0.0E+00	0.0E+00	0.0E+00	Rh-106	0.0E+00	0.0E+00	0.0E+00
Cs-136	0.0E+00	0.0E+00	0.0E+00	Ag-110m	1.2E-51	2.3E-55	3.9E-50
Cs-137	0.0E+00	0.0E+00	0.0E+00	Te-127m	6.0E-114	2.2E-117	2.4E-112
Cs-138	0.0E+00	0.0E+00	0.0E+00	Te-129m	0.0E+00	0.0E+00	0.0E+00
Cr-51	0.0E+00	0.0E+00	0.0E+00	Te-129	0.0E+00	0.0E+00	0.0E+00
Mn-54	6.0E-42	4.0E-45	1.3E-40	Te-131m	0.0E+00	0.0E+00	0.0E+00
Mn-56	0.0E+00	0.0E+00	0.0E+00	Te-131	0.0E+00	0.0E+00	0.0E+00
Fe-55	1.1E-15	5.4E-19	5.4E-15	Te-132	0.0E+00	0.0E+00	0.0E+00
Fe-59	4.5E-274	5.9E-278	5.9E-273	Te-134	0.0E+00	0.0E+00	0.0E+00
Co-58	1.5E-173	2.8E-176	1.4E-171	Ba-137m	0.0E+00	0.0E+00	0.0E+00
Co-60	1.0E-09	2.3E-13	7.5E-08	Ba-140	0.0E+00	0.0E+00	0.0E+00
Rb-88	0.0E+00	0.0E+00	0.0E+00	La-140	0.0E+00	0.0E+00	0.0E+00
Rb-89	0.0E+00	0.0E+00	0.0E+00	Ce-141	0.0E+00	0.0E+00	0.0E+00
Sr-89	0.0E+00	0.0E+00	0.0E+00	Ce-143	0.0E+00	0.0E+00	0.0E+00
Sr-90	1.8E-130	4.2E-135	8.4E-129	Pr-143	0.0E+00	0.0E+00	0.0E+00
Sr-91	0.0E+00	0.0E+00	0.0E+00	Ce-144	7.0E-46	4.1E-50	1.4E-44
Sr-92	0.0E+00	0.0E+00	0.0E+00	Pr-144	0.0E+00	0.0E+00	0.0E+00

Note:
∑(Max conc_i / ECL_i) at well = 0.4%

HAR COL 2.4-5

Table 2.4.13-206
Minimum Kd Values from Testing of HAR Site Samples [(a)]

Samples	Lithology[(a)]	Bore Hole	Sample Depth[(a)] (ft. bgs)	Minimum Kd Cs (ml/g)	Minimum Kd Sr (ml/g)
4	Sandstone	BPA-41	35.6 – 37.6 [(b)]	3072	52
		BPA-42	52.7 - 53.6		
		BPA-39	77 - 78		
		BCTA-2	48 - 48.7		
4	Siltstone	BPA-16	79.9 - 80.9	7584	48
		BPA-42	30.3 - 31.3		
		BPA-43	35.3 - 36.3		
		BCTA-2	76 – 78 [(b)]		
4	Shaley Siltstone	BPA-16	141.8 – 144 [(b)]	3286	71
		BPA-41	124.9 - 126.1		
		BPA-39	61.0 - 61.5		
		BPA-43	87.1 - 88.2		
2	Diabase Dike	BGA-17	39.6 - 40.6	422	18
		BGA-17	68.0 - 69.2		
6	Soils	BPA-16	0	4747	39
		BPA-41	2.5		
		BPA-42	2.5		
		BPA-39	15		
		BPA-43	7.5		
		BCTA-2	3.3		

Notes:

a) From Reference 2.4-269.

b) The sample depth range includes the duplicate QC sample.

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 Subsection 2.5.6.

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 proposed Shearon Harris Nuclear Power Plant Units 2 and 3 (HAR) site. This section was developed in accordance with requirements outlined in Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants." 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 Subsection 2.5.6, and references are in Subsection 2.5.7.

HAR SUP 2.5-1

2.5.0 SUMMARY

2.5.0.1 Basic Geologic and Seismic Information

2.5.0.1.1 Regional Geology

The HAR site is located in the Triassic Deep River basin near the eastern edge of the Piedmont Plateau in central North Carolina. The site is within the Piedmont physiographic province, which lies between the Blue Ridge and Coastal Plain provinces. The Deep River basin is a northeast-southwest-trending topographic trough, lying near the eastern edge of the Piedmont Plateau, which is occupied by a complex, wedge-shaped, down-faulted block of Triassic rocks. It is bounded on the west, north, and east by pre-Triassic metamorphic and igneous rocks of the Piedmont Plateau. On the south and southeast, along the inner edge of the Coastal Plain, is a cover of post-Triassic (Coastal Plain) deposits that overlap and cover parts of the Triassic and older rocks. The Deep River basin is subdivided into three smaller basins named, from north to south, the Durham, Sanford, and Wadesboro basins. The HAR site is located in the southern part of the Durham basin near the Colon cross structure, a basement high that separates the Durham from the Sanford basin.

The HAR site lies within a compressive midplate stress province characterized by reverse and strike-slip faulting. A relatively uniform east-northeast compressive

stress field has been identified that extends from the mid-continent east toward the Atlantic continental margin and possibly into the western Atlantic basin. 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. Analysis of well-constrained focal mechanisms of North American midplate earthquakes indicates that Central and Eastern United States (CEUS) earthquakes occur primarily in response to a strike-slip stress regime.

The Electric Power Research Institute and Seismic Owners Group (EPRI-SOG) seismic hazard analysis for the existing Shearon Harris Nuclear Power Plant Unit 1 (HNP) site identified the Charleston seismic zone, the source of a large, geologically recent earthquake, as a significant seismic source at a distance of close to or slightly more than 320 kilometers (km) (200 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 as described in Subsection 2.5.2.4.1.1.1.

2.5.0.1.2 Site Geology

The HAR site is located within the Deep River basin, a north- to northeast-trending half-graben, approximately 6.4 km (4 mi.) west of the eastern margin of the Deep River basin, where the Jonesboro fault separates Triassic rift-fill sediments from igneous and metamorphic rocks of the Piedmont Plateau. The sediments of the Deep River basin are composed largely of debris from nearby pre-Triassic metamorphic and igneous rocks; in places, the deposits contain much debris from nearby granite intrusive bodies. These sediments were deposited as alluvial fans, stream channel and flood plain deposits, and lake and swamp deposits. Bedrock within the site area consists predominantly of well-consolidated Triassic siltstones and sandstones interbedded with subordinate shale, claystone, and conglomerate. Diabase intrusive rocks also are present in the site area and site vicinity. Additionally, Proterozoic and Paleozoic igneous and metamorphic rocks; remnants of the flat-lying, poorly consolidated sediments of the Coastal Plain; terrace gravels; and Holocene alluvial deposits occur in the site vicinity.

Two main sets of faults cut the Deep River basin. The major set strikes northeast (paralleling the Jonesboro fault) and includes faults both synthetic and antithetic to the Jonesboro. These faults cut the basin into a series of largely post-depositional fault blocks that show differential subsidence. The other set of faults is roughly perpendicular to these, striking northwest. These faults are nearly vertical, generally strike N25W, and are characterized by much smaller vertical displacements. Many of these northwest-striking faults have been intruded by diabase dikes. Faults mapped in the site area (within an 8-km [5-mi.] radius) include the Jonesboro fault; the Harris fault; the South Borrow Pit fault; three smaller faults (faults A, B, and C) at the borrow pits; the W8 and W82 faults; and two small, unnamed faults. Several other more distant faults have been mapped within the site vicinity. These include the Bonsal-Morrisville, which

like the Jonesboro fault appears to be a primary southwest-dipping fault, bounding a half graben within the Deep River basin; and a number of postulated Cretaceous and Cenozoic faults that are identified at the boundary between the Piedmont and Inner Coastal Plain provinces to the east of the Deep River basin. Based on detailed studies, none of these faults are considered to be a capable tectonic source, as defined in the U.S. Nuclear Regulatory Commission's (NRC's) Regulatory Guide 1.208.

Based on the review and updating of the geological, seismological, geophysical, and geotechnical data for the HAR site, nothing was identified that would preclude the safe operation of the proposed facilities. There is no potential for occurrence of nontectonic geologic events (e.g., landsliding, collapse or subsidence, or differential settlement) that would pose a hazard to the facilities. The site is not located in an area where large-scale crustal stresses related to glacial or erosional/depositional loading or unloading have occurred in recent geological time. The absence of reported shallow stress-relief features and the limited seismicity in the region also suggest minimal unrelieved residual stresses in bedrock in the site vicinity. Previous investigations performed for the HNP site and for this study identified no human activities that would adversely affect the site. Reservoir-triggered seismicity has not occurred since filling of the Harris Reservoir, and there is no reason to expect it will occur in the future. There are no known mineral resources of economic significance at the HAR site. Oil and gas exploration in the 1980s within the Durham basin identified no oil or gas prospects. There is limited withdrawal of groundwater in the site area, and adverse effects from groundwater withdrawal and construction groundwater control are not expected to occur at the HAR site.

2.5.0.2 Vibratory Ground Motion

The selected starting point for developing the site-specific ground motion assessments for the HAR site was the Probabilistic Seismic Hazard Analysis (PSHA) conducted by the EPRI-SOG in the 1980s. Following guidance in 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 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. The earthquakes in the updated catalog that have occurred between 1774 and January 1, 2007, within 320 km (200 mi.) of the HAR site include 225 events of body-wave magnitude (m_b) ≥ 3. The largest recorded earthquake in the region is the 1886 Charleston earthquake, with an estimated magnitude of m_b 6-¾ and a

maximum Modified Mercalli Scale (MM) intensity of X. This earthquake occurred approximately 315 km (195 mi.) from the HAR site. The remaining earthquakes within 320 km (200 mi.) of the HAR site all have estimated or recorded m_b values of 5.7 or less and maximum intensities of MMI VII½ or less. The closest event to the HAR site in this catalog is the 1896 MMI IV earthquake at an approximate distance of 80 km (50 mi.). Sensitivity analysis showed that updates to the earthquake catalog for the site region did not produce a significant change in earthquake occurrence rates from those defined by the EPRI-SOG earthquake catalog.

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, two additional specific seismic sources were identified and evaluated: repeated large-magnitude earthquakes at Charleston and the postulated East Coast fault system.

The EPRI-SOG seismic source models in the vicinity of Charleston, South Carolina, were updated in 2006 by the Southern Nuclear Company 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). For the HAR site, the seismic source model for repeated large-magnitude, Charleston-type earthquakes is taken directly from the UCSS presented by Southern Nuclear Company.

The postulated East Coast fault system (ECFS) was not recognized as a specific tectonic feature at the time of the EPRI-SOG study in the late 1980s, and therefore was not considered as a distinct source by the EPRI-SOG teams. There is considerable uncertainty about the existence, activity, and seismogenic potential of the postulated ECFS. The postulated ECFS is classified as a Class C feature, or one for which “geologic evidence is insufficient to demonstrate (1) the existence of tectonic faulting, or (2) Quaternary slip or deformation associated with the feature.” Only the southern portion of the southern segment is included in the National Seismic Hazard Mapping Project as an alternative seismic source for repeated large-magnitude events in the Charleston region. The central and northern segments of the postulated ECFS (ECFS-C and ECFS-N) were not included as Quaternary active faults in the 2002 U.S. Geological Survey (USGS) hazard model, and there are no plans to include them in the 2007 national seismic hazard maps.

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 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 are applied to the postulated ECFS-C segment, which lies closest to the HAR site, resulting in a very low probability that the source exists and is active. The southern segment of the postulated fault zone (ECFS-S) is included as a possible alternative fault source for the UCSS that was developed by Southern Nuclear Company.

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:

- The updated earthquake catalog does not show a pattern of seismicity different from that exhibited by earthquakes in the EPRI-SOG catalog that would suggest a new seismic source, or that a significant revision to the geometry of any of the seismic sources defined in the EPRI-SOG characterization is required.

- The updated catalog does not show any earthquakes within the site region that can be associated with a known geologic structure.

- The largest earthquake within the site region, 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.

2.5.0.2.4 Probabilistic Seismic Hazard Analysis (PSHA) and Controlling Earthquakes

Sensitivity studies were carried out during this study to address any need for changes in the EPRI-SOG PSHA. These included the following:

- A review of the source sets that should be included to capture at least 99 percent of the total hazard using the updated ground motion models developed by EPRI in 2004.

- An assessment of the weights assigned to source zone alternatives, relative to the occurrence of past earthquakes in the site region.

- A review of the maximum magnitude distributions for individual seismic sources relative to the size of recorded earthquakes in the updated seismicity catalog.

The review of the EPRI-SOG seismic sources and the sensitivity tests resulted in modifications to the EPRI-SOG seismic sources as implemented in the EPRI 1989 calculations for the HAR site. The resulting mean exceedance frequencies produced by the revised EPRI-SOG seismic sources are approximately 12 percent greater than that computed using the EPRI 1989 source implementation in the range of 10^{-4} to 10^{-5}.

Sensitivity analyses also showed that the updated characterization of repeated earthquakes near Charleston should be incorporated into updated PSHA for the HAR site. The assessment of the postulated ECFS is somewhat speculative. Evidence for the source's existence and potential activity is very weak. However, consistent with statements in the North Anna Final Safety Evaluation Report (FSER), the possible contribution to seismic hazard was evaluated. The ECFS does have a minor contribution (1 to 2 percent) to the HAR site hazard at exceedance frequencies in the range of 10^{-5} to 10^{-6}. Therefore, it was included in the updated PSHA for the HAR site.

The PSHA for the HAR site was conducted using the updated EPRI-SOG seismic sources combined with the UCSS and postulated ECFS sources. 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. The hazard was calculated 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 three equally weighted conversion relationships.

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 to that obtained from the updated EPRI-SOG sources for 10-Hz motions, and dominates the hazard for 1-Hz motions. The postulated ECFS produces a very minor contribution to the frequency of exceeding 1-Hz ground 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 moderate-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.

Smooth response spectra for the controlling earthquakes were developed using spectral shapes developed in NUREG/CR-6728.

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 HAR site. A combination of suspension logging, downhole velocity surveys, and spectral analysis of surface waves (SASW) surveys were used to measure shear wave velocities to a depth of approximately 61 meters (m) (200 feet [ft.]). Measurements were conducted in or near seven borings. Suspension logging interval V_S velocities were measured in all borings, downhole velocity measurements were conducted in two borings, and SASW surveys were conducted adjacent to four borings.

The reference location for the site ground motion response spectra (GMRS) was set at the top of competent rock. Because of the variable conditions across the site, three shallow velocity profiles are defined to represent the upper 30 m (100 ft.) of the subsurface materials: one for the proposed Shearon Harris Nuclear Power Plant Site 2 (HAR 2) site and two for the proposed Shearon Harris Nuclear Power Plant Site 3 (HAR 3) site. Similar velocity measurements were obtained in all borings below elevation 45.7 m (150 ft.) and a common velocity profile was used for depths below this point. Site response analyses were conducted for all three profiles in order to assess the range of ground motions representative of conditions at the site. The site response results for the two HAR 3 profiles were generally similar and enveloped in developing the ground motions for the HAR 3 site.

Sixty randomized sets of dynamic properties were developed for each site profile. Uncertainty in material damping and nonlinear behavior of the shallowest weathered rock were incorporated into the analysis by using alternative sets of modulus reduction and damping relationships for weathered rock and alternative values of the total site kappa value. Thirty rock time histories were developed for

each DE magnitude-distance pair by weak spectral matching recordings from NUREG/CR-6728 to the target DE spectra. Each of the 30 time histories was used to compute the response of two profile-soil property curves sets, providing a total of 60 site amplification functions for each DE. The arithmetic mean of the 60 individual response spectral ratios was then computed to define the amplification function for each DE. Following Approach 2B, a weighted combination of the individual DE amplification functions was computed to define the HF and LF amplification of the site at each hazard level. These amplification functions were smoothed and then used to scale the rock UHRS and RE spectra for the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels, to develop free-field surface spectra on a hypothetical outcrop of competent material treated as a free-field outcropping surface for the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. These surface spectra are based on PSHA calculations that use cumulative absolute velocity (CAV) filtering in place of a fixed minimum magnitude.

2.5.0.2.6 Ground Motion Response Spectra

The horizontal GMRS for the HAR site were developed using the performance-based approach defined in NRC Regulatory Guide 1.208. The vertical GMRS were developed by multiplying the horizontal GMRS by vertical/horizontal spectral ratios recommended for rock in NUREG/CR-6728.

2.5.0.3 Surface Faulting

Investigations performed to evaluate the potential for surface fault rupture at the HAR 2 and HAR 3 sites, as well as the surrounding HAR 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 or nontectonic surface faulting or fold deformation at the HAR site, and no capable tectonic sources have been identified within 40 km (25 mi.) of the site.

The HAR site area (an 8-km [5-mi.] radius) sits largely within the Deep River basin, a north- to northeast-trending half-graben. The HAR site is located approximately 6.4 km (4 mi.) north of the Jonesboro fault and sits in the hanging wall of the fault, which is disrupted by distributed synthetic and antithetic faults. Smaller-scale intrabasin and cross-basin faults in the site area also have been identified based on surface geological investigations, subsurface trenching and drilling, and interpretation of seismic reflection data. In addition to the Jonesboro fault, the closest well-documented faults to the HAR site include the Harris fault, the South Borrow Pit fault, the W8 and W82 faults, and minor faults exposed within igneous and metamorphic rocks in the foundations of the Main Dam structures approximately 8 km (5 mi.) south of the HAR 2 site. Comprehensive studies of the Harris fault and Main Dam faults were completed as part of the HNP Fault Investigations Program and the Final Report on Foundation Conditions for Power Plant, Dams, and Related Structures, respectively. The South Borrow Pit fault and the W8 and W82 faults were identified and studied as

part of investigations for a proposed low-level radioactive waste (LLRW) disposal facility site in Wake County, approximately 2.4 km (1.5 mi.) west of the HNP site.

No evidence of Quaternary deformation associated with these structures is reported in the literature or was observed during field and aerial reconnaissance conducted for this study. There are no associated historical earthquakes or alignments of seismicity to suggest the presence of a capable tectonic source in the site area.

Light detection and ranging (LIDAR) data that provide detailed images of the surface topography were used to evaluate the previously identified faults in the site vicinity and to identify lineaments in the site area. The Cretaceous and Cenozoic faults previously identified in the site vicinity and adjoining region are not expressed as topographic features in hillslope maps derived from the LIDAR data. Several trends of lineaments are expressed in the LIDAR data within the site area. The identified lineaments can be explained by nontectonic mechanisms, such as differential erosion along less resistant bedrock units or along pre-existing ancient bedrock structures.

The potential for tectonic deformation at the HAR site is assessed to be negligible. This conclusion is based on the following:

- The results of comprehensive fault investigations have demonstrated that mapped faults within the site area are not capable faults.
- Evidence observed in trenches shows that the faults identified at the LLRW disposal facility site, approximately 2.4 km (1.5 mi.) west of the HAR site, predate significant weathering and soil formation, or exhibit no evidence of recent faulting.
- Bedrock geologic mapping in the site vicinity identified no evidence for surface faulting or deformation that would suggest capable tectonic sources in the HAR site area.
- The absence of geomorphic features indicative of Quaternary deformation, as reported in the previous HNP reports and literature, was supported by observations made during the field reconnaissance conducted for this study.

2.5.0.4 Stability and Uniformity of Subsurface Materials and Foundations

Bedrock at the HAR sites consists predominantly of reddish-brown siltstone and reddish-brown to gray sandstone. Subordinate amounts of shale, conglomerate, and claystone were also occasionally encountered. Sound rock is typically present above the nuclear island subgrade elevation of 67.1 m (220 ft.) NGVD 29 at each HAR site. Isolated intervals of very weak rock and soil-like layers are present within otherwise sound rock in various locations. Such intervals under the nuclear islands are less than 1 m (3 ft.) thick. The depth to sound rock is shallower at HAR 2 than at HAR 3, in part because surficial soil and weathered rock at HAR 2 were

removed in the 1970s to create the existing site grade during construction of the HNP. The soil thickness over rock ranges from 1.5 to 4.6 m (5 to 15 ft.) at HAR 2, and 3.0 to 7.6 m (10 to 25 ft.) at HAR 3. Transitions from soil to weathered rock and to sound rock are gradual.

The continuity of individual rock strata varies both laterally and with depth as a result of the mechanisms of deposition of the Triassic basin. Fine-grained sandstone and siltstone strata are often traceable across multiple boreholes at HAR 2 and HAR 3. Coarse-grained sandstone deposits associated with stream deposition exhibit more lateral variability than the finer grained deposits. At HAR 2, bedrock strata dip between 6 and 9 degrees to the east. At HAR 3, strata dip approximately 20 degrees to the east-southeast based on V_S measurements, with local strata dip based on marker bed contacts marginally greater and less than 20 degrees in some locations.

A subsurface investigation program, consisting of geotechnical boreholes, geophysical surveys, in-situ testing, and laboratory testing, was performed from April through December 2006 in accordance with Regulatory Guide 1.132 and Regulatory Guide 1.138. Eighty-three boreholes were advanced, including 50 under or adjacent to the proposed Westinghouse Electric Company, Limited Liability Company's (LLC) (Westinghouse's) AP1000 Reactor (AP1000) structure sites (BPA-series boreholes). Geophysical survey methods included seismic refraction, magnetometer, SASW, suspension logging, and downhole logging. In addition, pressuremeter testing (PMT) was performed at various depths in two boreholes at each HAR site. A total of 80 special-care rock core samples were laboratory tested for unconfined compressive strength (UCS) and other index tests. Numerous soil samples were tested for index properties, strength properties, and consolidation properties.

Engineering properties of subsurface materials were characterized from the site investigation activities. Two of the key properties are summarized as follows:

- V_S is greater than 1372 meters per second (m/sec) (4500 feet per second [fps]) below the HAR 2 nuclear island basemat elevation (67.1 m [220 ft.]), and greater than 1067 m/sec (3500 fps) below the HAR 3 nuclear island, with a few exceptions at isolated very weak rock or clay seams less than 1 m (3 ft.) thick. V_S is typically greater than 1524 m/sec (5000 fps) below elevation 54.9 m (180 ft.) at each suspension logging borehole, except at a few isolated intervals noted previously. Comparison of V_S profiles at the three suspension logging boreholes at each HAR site indicates similarity in V_S along continuous strata under each nuclear island. Average Poisson's ratio of rock is approximately 0.33 at each HAR site.

- The UCS from laboratory tests on intact rock core samples from below nuclear island grade averages approximately 48.3 megaPascals (MPa) (7000 pounds per square inch [psi]) at HAR 2, and approximately 44.8 MPa (6500 psi) at HAR 3. Results range from 9.6 MPa (1388 psi) to 123.1 MPa (17,851 psi) among all samples tested from the HAR sites. These averages

and range of values are consistent with UCS calculated from rock point load tests conducted at numerous depths at two boreholes per HAR site.

Sloping excavations are planned for each nuclear island. Sound rock will be excavated at a planned slope of 0.25H:1V (horizontal:vertical) to 0.5H:1V, and soil will be excavated at a planned slope of 1.5H:1V to 2H:1V, which may be modified based on excavation conditions or future construction considerations. The excavation base and sidewalls will be geologically mapped and improved (as necessary) prior to construction of the nuclear island. Backfill against the nuclear island sidewalls will consist of concrete or compacted fill within sound rock excavations, and compacted fill within soil and weathered rock excavations.

Groundwater dewatering flow rates were calculated based on in-well groundwater elevations and hydraulic conductivity, which was calculated based on rapid drawdown tests summarized in Subsection 2.4.12. It is anticipated that groundwater inflow can be managed by pumping directly from the excavations. However, a system of perimeter dewatering wells may be installed during construction if groundwater flow rates are higher than anticipated.

The factors of safety (FS) for static and dynamic bearing capacity were analyzed for the nuclear islands using multiple methods, including the general bearing capacity equation (based on Hoek-Brown rock mass strength parameters) and an oriented rock discontinuity model. For the general bearing capacity equation method, conservative Hoek-Brown rock mass strength parameters were modeled based on the range of intact rock UCS test results and rock mass discontinuity data. For the oriented rock discontinuity method, rock discontinuity orientations and engineering properties were conservativey modeled based on the observed conditions of bedding planes and joint sets. The calculated static FS and dynamic FS were greater than 3.0 and 2.0, respectively, for each modeled condition.

The nuclear island foundations have no potential for liquefaction because these foundations consist of sound rock. Likewise, the Annex Buildings and the first bay of the Turbine Buildings (Seismic Category II structures) will be founded on sound rock or concrete fill over sound rock, which will have no potential for liquefaction. Field verification testing will be performed during construction to ensure compacted granular fill placed under other structures adjacent to the nuclear islands will not liquefy.

Total and differential settlements of the nuclear islands were estimated based on conservative estimates of elastic compression and consolidation of isolated clay seams within rock. Best estimates of total settlements are less than 1.3 centimeters (cm) (0.5 inch [in.]) at each HAR site, and differential settlement (distortion) slopes are estimated to be less than 0.0004, which are well within acceptable settlement criteria for the AP1000 nuclear islands.

2.5.0.5 Stability of Slopes

The site grade at the HAR site will be constructed at approximately 79.2 m (260 ft.), with minor variations to allow drainage, to a distance of at least 150 m (500 ft.) in each direction from the nuclear island structures. 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 HAR site safety-related facilities.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

HAR COL 2.5-1

The information in this subsection is organized in accordance with the NRC Regulatory Guide 1.208. Regulatory Guide 1.208 describes information and the level of investigation needed to provide an up-to-date, site-specific, earth science database that supports site characterization and a PSHA. The emphasis is on any new information that would suggest significant differences from the information used to develop the Electric Power Research Institute's (EPRI's) source model, which forms the starting point for the assessment of seismic hazard at sites in the CEUS (see discussion in Subsection 2.5.2) (Reference 2.5.1-201). The EPRI-SOG study represented an extensive evaluation of the scientific knowledge concerning earthquake hazards in the CEUS by multidisciplinary teams of experts in geology, seismology, geophysics, and earthquake ground motions. Regulatory Guide 1.208 specifies that the adequacy of the EPRI-SOG hazard results must be analyzed in light of new data and interpretations and evolving knowledge pertaining to the evaluation of seismic hazard in the CEUS.

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 HAR site. The EPRI-SOG seismic hazard analysis for this site identified significant seismic sources at distances greater than 320 km (200 mi.), specifically the Charleston seismic zone, which was the source of a large, geologically recent earthquake. Updated information regarding the location, magnitude, and recurrence of this more distant, but significant, seismic source is also presented. 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). Subsection 2.5.1.2 also addresses geologic conditions at the site (1-km [0.6-mi.] radius).

Several sources of information were used to develop the descriptions in this chapter. Initial compilation included review of reports and documents specific to HNP, including the "Shearon Harris Nuclear Power Plant Final Safety Analysis Report" (HNP FSAR) issued in 1983 by Carolina Power and Light Company (CP&L) (Reference 2.5.1-202), and the "Shearon Harris Nuclear Power Plant Final Safety Evaluation Report" (HNP FSER), NUREG-1038, issued in 1983 by

the NRC (Reference 2.5.1-358). Also reviewed were data sets and analyses compiled and interpreted for other site-specific and regional studies throughout the CEUS, since completion of the EPRI-SOG study in the late 1980s (Reference 2.5.1-203). These studies used a variety of techniques to characterize the location, extent, and activity of tectonic features; the location, magnitude, and rates of seismic activity; and the general characteristics of the continental crust throughout the CEUS. The information summarized in this subsection incorporates the information and findings of these studies, as well as recent reports, maps, and articles published by state and federal agencies and in professional/academic journals. Additional unpublished data and information were obtained through communications with individual researchers at universities and state agencies.

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 HAR site. Also presented is relevant new information on potential seismic sources for the large-magnitude 1886 earthquake in the Charleston, South Carolina, area, close to or just outside the 320-km (200-mi.) radius of the site.

The HAR site is located in the Triassic Deep River basin near the eastern edge of the Piedmont Plateau in central North Carolina. Physiographically, North Carolina is divided into three natural divisions: the Coastal Plain on the east, the Piedmont Uplands (or Piedmont Plateau) in the center, and the Appalachian Highlands on the west, as shown on the regional physiographic map (Figure 2.5.1-201). The Deep River basin is a northeast-southwest-trending topographic trough, lying near the eastern edge of the Piedmont Plateau, which is occupied by a complex, wedge-shaped, down-faulted block of Triassic rocks. It is bounded on the west, north, and east by pre-Triassic metamorphic and igneous rocks of the Piedmont Plateau. On the south and southeast, along the inner edge of the Coastal Plain, is a cover of post-Triassic (Coastal Plain) deposits that overlap and cover parts of the Triassic and older rocks. (Reference 2.5.1-202)

The Deep River basin is subdivided into three smaller basins named, from north to south, the Durham, Sanford, and Wadesboro basins (Figure 2.5.1-202). The HAR site is located in the southern part of the Durham basin near the Colon cross structure, a basement high that separates the Durham from the Sanford basin. The boundaries of these smaller, component basins are undefined. (Reference 2.5.1-204)

2.5.1.1.1 Regional Physiography and Topography

The HAR site is located in the Appalachian Highlands Division of North America (Reference 2.5.1-205, Figure 2.5.1-201). Seven physiographic provinces are recognized within the Appalachian Highlands Division, four of which are within a 320-km (200-mi.) radius of the HAR site: the Piedmont, Blue Ridge, Valley and

Ridge, and Appalachian Plateaus provinces. (The remaining three are the New England, Adirondacks, and St. Lawrence Valley provinces) (Reference 2.5.1-206). The HAR site is within the Piedmont Uplands section of the Piedmont province. The Atlantic Plain Division lies southeast of the Appalachian Highlands Division (Figure 2.5.1-201). The Coastal Plain province is the zone that parallels the Atlantic Coast shoreline within the study region. The major physiographic provinces within the study region are used as the basis for organizing many of the descriptions of geologic features in this subsection (although these provinces correspond imperfectly with lithotectonic subdivisions following Hatcher [Reference 2.5.1-207]). The following descriptions of the provinces are taken from Thornbury and the North Carolina Geological Survey (NCGS) (Reference 2.5.1-206, Reference 2.5.1-208).

2.5.1.1.1.1 Physiography and Topography of the Piedmont Province

The Piedmont physiographic province lies between the Coastal Plain and the Blue Ridge provinces (Figure 2.5.1-201). The Piedmont province, which is the least mountainous part of the Appalachian Highlands, is characterized by gently rolling, well-rounded hills and long, low ridges (Reference 2.5.1-206). In North Carolina, along the border between the Piedmont and Coastal Plain provinces, elevations range from 90 to 180 m (300 to 600 ft.) above sea level. To the west, elevations gradually rise to approximately 460 m (1500 ft.) above sea level at the foot of the Blue Ridge Mountains. (Reference 2.5.1-208)

The Piedmont province contains metamorphic and plutonic rocks that have highly complex structures. The maximum degree of metamorphism in the Appalachian Highlands is observed in about the middle of the Piedmont province; the metamorphic rank decreases to both the southeast and northwest (Reference 2.5.1-206).

The Triassic Deep River lowland surface, in which the HNP and HAR sites are located, generally is 15 – 60 m (50 – 200 ft.) lower than areas in the upland part of the province, which border the lowland on the southwest, west, and northeast, and areas of the Coastal Plain upland, which border it on the south and east. Consequently, the lowland is bordered locally by abrupt erosional escarpments. Elevations range from less than 50 m (160 ft.) near the Cape Fear River to more than 150 m (500 ft.) in some places in the northern part of the lowland. Except for near major streams and along some border escarpments, local relief generally is less than 30 m (100 ft.). The lowland surface is more intricately dissected than the Piedmont upland surface, and stream courses in the lowland are better adjusted to the structure of the bedrock. Stream valleys are wider in the lowland than in the upland, and narrow floodplains abut the rivers and their larger tributaries. Most of the smaller tributaries flow through valleys that have steep sides and narrow bottoms. The most extensive flat areas in the lowland are formed by terraces that border some of the major streams, such as the Haw, Deep, and Cape Fear rivers. (Reference 2.5.1-202)

2.5.1.1.1.2 Physiography and Topography of the Blue Ridge Province

The Blue Ridge physiographic province is bounded on the east by the Piedmont physiographic province and on the west by the Ridge and Valley physiographic province. The Blue Ridge province is a deeply dissected, mountainous area of numerous steep mountain ridges, intermontane basins, and trench valleys that intersect at all angles, giving the area its rugged mountain character. The Blue Ridge province contains the highest elevations and the most rugged topography in the Appalachian Mountain system. The North Carolina part of the Blue Ridge province is about 320 km (200 mi.) long and ranges from 25 to 90 km (15 to 55 mi.) wide. Within North Carolina, 43 peaks in the province exceed 1800 m (6000 ft.) in elevation; 82 peaks are between 1500 and 1800 m (5000 and 6000 ft.). (Reference 2.5.1-208)

The rocks of the Blue Ridge province are primarily Precambrian metamorphic and igneous rocks and early Paleozoic sedimentary (clastic) rocks that are folded and faulted, and that show varying degrees of metamorphism. Several great shear thrust faults are found along the western border of the province (Reference 2.5.1-206).

2.5.1.1.1.3 Physiography and Topography of the Valley and Ridge Province

The Valley and Ridge physiographic province, west of the Blue Ridge physiographic province, extends for 1900 km (1200 mi.), from eastern New York to central Alabama (Reference 2.5.1-206). Closed folding and thrust faulting characterize the province, which shows marked parallelism of ridges and valleys that are oriented northeast-southwest and that contain numerous water gaps and wind gaps that indicate past conditions of stream diversion (Reference 2.5.1-206).

Rocks of the Valley and Ridge province, which are predominantly sedimentary, range in age from lower to upper Paleozoic, with ages decreasing from east to west. Carbonate rocks are abundant within the province, although sandstone formations are prominent topographically. (Reference 2.5.1-206) Alternating strong and weak strata produce a conspicuous influence on topographic forms (Reference 2.5.1-206).

2.5.1.1.1.4 Physiography and Topography of the Appalachian Plateaus Province

The Appalachian Plateaus physiographic province, which is bounded on all sides by outfacing escarpments, rises higher in elevation than adjoining provinces (Reference 2.5.1-206). The scarps along the eastern margin of the province generally are higher and more clearly defined than those on the west. The Appalachian Plateaus province is essentially a broad syncline in predominantly clastic rocks of Paleozoic age (i.e., mainly Mississippian and Pennsylvanian, with some Devonian strata). In contrast to the adjoining provinces, the Appalachian

Plateaus have not been subjected to intense deformation and metamorphism, although most of the province has undergone considerable dissection. (Reference 2.5.1-206)

2.5.1.1.1.5 Physiography and Topography of the Coastal Plain Province

Along the eastern margin of the Appalachian Highlands Division, adjacent to the Atlantic Ocean, is the eastward-sloping Coastal Plain physiographic province within the Atlantic Plain Division, the part of the plain below sea level called the continental shelf (Reference 2.5.1-206). Located in the eastern third of North Carolina, the province is characterized by flat land to gently rolling hills and valleys (Reference 2.5.1-208). Elevations of the inner margin of the Coastal Plain range from 120 to 180 m (400 to 580 ft.) (Reference 2.5.1-202).

The Coastal Plain province is underlain by relatively unconsolidated sediments of both marine and terrestrial origin that range in age from Early Cretaceous to Holocene. In general, the younger sediments adjoin the continental shelf on the east, and the oldest sediments lie to the west and northwest. (Reference 2.5.1-206) 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.

Because there are significant differences in both geology and topography within the Coastal Plain province, it has been divided into six sections (Reference 2.5.1-206). The two sections that lie within a 320-km (200-mi.) radius of the HAR site are the embayed section on the north and the Sea Island section on the south. In the embayed section, estuarine embayments divide the Coastal Plain into broad peninsular tracts. The Sea Island section is characterized by youthful to mature terraced coastal plain. The Sea Island section shows less marked drowning of valleys than does the embayed section, and its terraces do not reach back to the inner border of the section. (Reference 2.5.1-209) The transition between the embayed and Sea Island sections, which is generally coincident with the northern flank of the Cape Fear arch, is typical of the basin and arch architecture of the U.S. Atlantic margin where broad regional flexural warping, in response to isostatic loading and unloading of the emerged Coastal Plain, is evidenced by depositional patterns during the Cretaceous and Cenozoic. (Reference 2.5.1-267)

2.5.1.1.2 Regional Geologic History

The HAR site is located within the Appalachian orogenic belt, a region that has a long and complex tectonic history of folding and faulting that led to the formation of the present Appalachian Mountain Range, which in the United States is exposed from Maine to Alabama. (Note: the process of mountain-building, particularly by folding and thrusting, is referred to as an orogen or orogeny; the area affected is an orogenic belt.) Information from stratigraphic sequences, the timing of major fault activity, and the timing of plutonic intrusions and

metamorphism have been used to reconstruct the plate tectonic history of the Appalachians. Details of these data sets and interpretations of the geologic history of the region are presented in a series of papers in a volume describing the Paleozoic Appalachian-Ouachita orogen in the United States and in papers describing the geologic history of the central Appalachians (Reference 2.5.1-210, Reference 2.5.1-211, Reference 2.5.1-212, Reference 2.5.1-213, and Reference 2.5.1-214). These references and other recent publications, as noted, provide the basis for the following summary of regional geologic history.

The Appalachian orogen was built on the Late Proterozoic – Early Paleozoic continental margin of North America by a series of compressional events that began in the Ordovician and continued episodically throughout most of the Paleozoic (Reference 2.5.1-215). Based on evidence distributed on both sides of the Atlantic Ocean, the Appalachian orogen in the United States is part of the much larger Appalachian – Caledonide orogen. The orogen ended with the opening of the present Atlantic Ocean during the Mesozoic (Reference 2.5.1-216). Although the accretionary history of the central Appalachian orogen is not understood fully, many of the major events have been reconstructed. These events are summarized in the following subsections and shown from the Mesoproterozoic (>1 billion years) to the present on Figures 2.5.1-242 and 2.5.1-243.

2.5.1.1.2.1 Late Proterozoic and Paleozoic Geologic History

In the Mesoproterozoic, several continental masses, including the proto-North American (Laurentia) and proto-African continents, were assembled into a supercontinent (Rodinia) as a result of the Grenville orogeny (Reference 2.5.1-212). Grenvillian crystalline rocks are found as basement underlying most of the Appalachians (Reference 2.5.1-215). These billion-year-old rocks crop out in northeast-southwest-trending bands in north-central North Carolina (in the Raleigh belt, described in Subsection 2.5.1.1.3) and in Virginia, where the rocks are referred to as the Goochland terrane (Reference 2.5.1-211, Reference 2.5.1-217). Grenvillian rocks are also exposed as an internal massif in the Sauratown Mountains window (Figure 2.5.1-213; Reference 2.5.1-348). Crustal extension and rifting began in the Proterozoic and continued into the Paleozoic (earliest Cambrian), separating the North American and African continents and forming the intervening proto-Atlantic Ocean (the Iapetus, Theic, and Rheic oceans) (Reference 2.5.1-212). During rifting, the newly formed passive continental margin began to subside and accumulate siliclastic sediments, which subsequently were overlain by a thick, wide carbonate shelf (Reference 2.5.1-212). Nearly 3 km (10,000 ft.) of sediments accumulated on the shallow platform edge as the North American continent drifted in tropical seas throughout the period of approximately 30 million years during the early Cambrian (Reference 2.5.1-211).

The subsequent closing of the proto-Atlantic to form a late Paleozoic supercontinent (Pangea) occurred during a period of about 250 million years

(Reference 2.5.1-212). As the ocean was closing, numerous microcontinents and oceanic terranes formed and were accreted as exotic or suspect terranes onto the eastern margin of North America; these recently have been recognized throughout the present Appalachian region. 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 2.5.1-216, Reference 2.5.1-249). The amalgamation of these terranes to North America, combined with the final collision of North America with the West African continent, created the Appalachian orogen. Four or more major collisions, resulting in widely recognized orogenic events (e.g., the Potomac, Taconic, Acadian, and Alleghany orogenies shown on Figures 2.5.1-242 and 2.5.1-243) or the equivalent Penobscottian, Taconic, Acadian, and Alleghanian orogenies of Hatcher,[a] have been interpreted from the Paleozoic record (Reference 2.5.1-207, Reference 2.5.1-349). Orogenic episodes, however, appear to have occurred nearly continuously throughout the Paleozoic in various parts of the Appalachian orogen. (Reference 2.5.1-212)

At the beginning of the convergent phase in the early Paleozoic, the Potomac orogeny occurred offshore of the future central Appalachians when magmatic arcs converged toward and accreted onto microcontinents a considerable distance away from the margin of North America (Reference 2.5.1-212). During this time, the microcontinent Carolinia was being formed as a magmatic arc above a subduction zone (Reference 2.5.1-211). About 600 million years before present (Ma), the Virgilina orogeny began, folding and faulting the older volcanic sequence during the next 50 million years (m.y.), as the ocean between Carolinia and North America began to close. The Penobscottian event, between about 550 and 490 Ma, is the earliest major orogeny recognized in the Appalachian belt and primarily is expressed in the northern Appalachians. Evidence for the Penobscottian orogeny has not been observed south of Virginia, where the orogeny is bracketed in age between Late Cambrian metavolcanic rocks and an Early Ordovician pluton (Reference 2.5.1-350). Collision of Carolinia with North America occurred during the Middle Cambrian to Middle Ordivician (between about 520 and 490 Ma) (Reference 2.5.1-211). The margin of North America was deformed and metamorphosed by this collision, then cooled during subsequent thrust-driven uplift over the Piedmont and Blue Ridge areas (at about 480 Ma) (Reference 2.5.1-211). Figure 2.5.1-204 shows the current location of the Carolinia (magmatic arc) terrane with respect to the site. This figure also shows the large uplift of the 1-billion-year-old Grenville basement and the early Taconic (Cambrian to Late Ordovician) suture that separates the Carolinia terrane from the Laurentian (proto-North American continent) passive margin (Reference 2.5.1-211).

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a. Different authors use different spellings for similar terms (e.g., the Carolinia or Carolina terrane and the Allegheny or Alleghanian orogeny). In this document, the terminology established by the author whose work is being described is followed.

The Taconic orogeny is represented by the first accretion of assembled terranes onto the North American continental margin from the Middle to Late Ordovician. This orogeny, which extended from Newfoundland to Alabama, profoundly affected the tectonic framework of the existing carbonate shelf, creating a Taconic mountain system of unknown dimension on the east and a vast intracontinental basin, the Appalachian basin, on the west. The newly uplifted Taconic highland on the east created a regional drainage having a dominant northwestward flow that transported erosional debris (clastic sediments) into the basin during the Early Silurian. (Reference 2.5.1-212) As the siliciclastic input from the southeast waned, carbonate deposition was dominant in most of the north-central region of the basin into the Early Devonian (Reference 2.5.1-213). The onset of the Taconian event is marked regionally throughout much of the Appalachian belt by an unconformity in the passive-margin sequence and deposition of clastic sediments derived from an uplifted source area or areas to the east. Rocks of the eastern Blue Ridge and Inner Piedmont are interpreted to have originated east of the Laurentian passive margin in Middle Ordovician time and thus were possibly formed during the Taconic collision(s). (Reference 2.5.1-249)

In the Early to Middle Devonian, the Acadian orogeny began, and siliciclastic materials were reintroduced into the eastern part of the Appalachian basin (Reference 2.5.1-213). The Acadian orogeny is most clearly recorded in the northern Appalachians, where deformed and metamorphosed sedimentary and volcanic rocks, a clastic delta wedge, and abundant plutons provide evidence of this event (Reference 2.5.1-218). Evidence for the extent of the orogeny in the central and southern Appalachians is provided primarily by Acadian plutons, discontinuous local zones of medium- to high-grade metamorphism (Reference 2.5.1-218), unconformities in foreland stratigraphic succession, activity of several major faults, and possibly ductile folding in the southern Appalachians (e.g., the Kings Mountain, Gold Hills, and Brevard fault zones) (Reference 2.5.1-350).

The convergence of North America and Africa and their subsequent suturing during a continent-to-continent collision produced the Alleghany orogeny during the Late Carboniferous and earliest Permian. This, the last major compressive tectonism to affect the Appalachian orogen, affected a larger area of the presently exposed central and southern Appalachians than any previous Paleozoic tectonic event. (Reference 2.5.1-212) The Alleghany orogeny in the central and southern Appalachians involved a very low-angle thrust (décollement) that originated in the mid-crust east of the presently exposed Appalachians, then rose toward the west to progressively higher levels in the upper crust. The youngest (and thus best-preserved) manifestations of the Alleghany orogeny are northeast-southwest-trending strike-slip faults and dextral shear zones in the Piedmont region (i.e., the Nutbush Creek and Hollister faults). Metamorphism and magmatism were significant events during the earlier phase of the Alleghany orogeny in the southern Appalachians. (Reference 2.5.1-214)

The Alleghanian orogeny produced the classic Appalachian folds and faults of the southern and central Appalachians (Reference 2.5.1-218). During this orogeny, the southern and central Appalachians were transported westward toward the North American craton in the form of a huge composite crystalline thrust sheet called the Blue Ridge – Piedmont thrust sheet (Reference 2.5.1-210). Evidence for a master detachment beneath the Blue Ridge and Piedmont regions is derived from both geophysical and structural data (Reference 2.5.1-218). Alleghanian deformation and uplift produced a thick, clastic sedimentary sequence of conglomerates, sandstones, shales, and marls that were deposited following erosion of the mountain ranges uplifted during the orogen. These deposits, which constitute a molasse, are found today from Alabama to Pennsylvania. (Reference 2.5.1-218) Zones within the site region where metamorphism and ductile deformation associated with Paleozoic orogenic events have been recognized are shown on Figure 2.5.1-205. Some of the effects of the Alleghanian orogeny in the Carolinas include numerous granitoid plutons southeast of the Brevard fault zone; amphibolite-facies regional metamorphism and deformation in the Kiokee and Raleigh metamorphic belts of the eastern Piedmont; strike-slip movement along major faults from the Brevard fault zone southeastward to the eastern Piedmont fault system; westward transport of a composite stack of crystalline thrust sheets which now constitutes the western Piedmont and Blue Ridge; and imbricate thrusting and folding in the Valley and Ridge province (Reference 2.5.1-351).

2.5.1.1.2.2 Mesozoic Geologic History

The breakup of the supercontinent Pangea, which comprised the North American and African continents that had been welded together in the late Paleozoic, began with the opening of the present Atlantic Ocean during the Middle Triassic to Early Jurassic (Reference 2.5.1-219). The passive trailing margin of central eastern North America developed during rifting that ceased in the Early Jurassic. The tectonic regime in eastern North America changed substantially after rifting stopped, according to the work of Withjack et al. (Reference 2.5.1-219). Their investigations also indicate that rifting ended and drifting began earlier in the southeastern United States and later in the northeastern United States and southeastern Canada. Sketches showing the evolution of central eastern North America during the Mesozoic are shown on Figure 2.5.1-206.

Numerous rift basins developed throughout eastern North America during the Middle to Late Triassic, including the Triassic Deep River basin in which the HAR site is located. (Mesozoic rift basins are described in more detail in Subsection 2.5.1.1.4.2.3.) The rift basins were mostly asymmetric and were bounded on one side by a normal fault or series of normal faults (Reference 2.5.1-219). The presence of conglomeratic facies near the boundary faults and thickening of packages of strata toward the faults indicate that deposition and movement on the boundary faults were coeval. In general, the attitudes of the boundary faults reflect the crustal fabric produced during the Paleozoic orogenies, and many of the normal faults were reactivated Paleozoic structures (Reference 2.5.1-219). The maximum compressive stress during rifting

was subvertical, and the maximum horizontal stress axis (S_{hmax}) and minimum horizontal stress axis (S_{hmin}) trended approximately northeast-southwest and northwest-southeast, respectively (Reference 2.5.1-219).

The period of extension in the Triassic resulted in reactivation of approximately northeast-striking Paleozoic faults. A second phase of extension, during the Early Jurassic, was oriented differently, however, producing sets of north-south and northwest-southeast faults and dikes throughout the rifting province. In addition, voluminous basalt flows occurred and diabase sheets were intruded in the Deep River basin. (Reference 2.5.1-220)

Rifting ceased in the southeastern United States during the Early Jurassic (about 200 Ma), and the passive margin subsequently experienced a change in deformational regime and stress state (Reference 2.5.1-219). The maximum compressive stress after rifting was subvertical, and S_{hmax} and S_{hmin} trended approximately northwest-southeast and northeast-southwest, respectively. In response to this reorientation of stress, rifting and the associated northeast-striking normal faulting ceased, northeast-striking reverse faults formed, and the rift-basin boundary faults experienced reverse displacements. (Reference 2.5.1-219) Near the continent-ocean boundary, a massive volcanic or volcaniclastic wedge began to develop (Reference 2.5.1-219). The North American and African continents separated, and centers of sea-floor spreading developed. The continental margin of the southeastern United States subsided, and post-rift strata progressively onlapped the erosional surface. (Reference 2.5.1-219)

2.5.1.1.2.3 Cenozoic Geologic History

Erosional processes have been dominant in the southeastern United States since folding ceased during the Mesozoic. Throughout the study region, uplift has played a role in the development of the post-orogenic landscape. The present mountains resulted from Tertiary uplift and continued differential erosion of dissected Mesozoic and Tertiary surfaces as the crust readjusted isostatically to erosional unloading (Reference 2.5.1-207).

The post-orogenic history of the Blue Ridge and Piedmont regions differs from that of the Coastal Plain because of the different factors that controlled sedimentation. In contrast to the Coastal Plain, where sedimentation resulted from large-scale transgressive-regressive cycles controlled by eustatic sea level, in the Blue Ridge and Piedmont regions erosional processes have dominated, with deposition of colluvium and alluvium generally controlled by smaller-scale climatic variations (Reference 2.5.1-209). For example, colluvial sedimentation in the Blue Ridge may be triggered by intense rainfall and may have been greatest during glacial maxima when vegetation was sparser. In contrast, sedimentation in the Coastal Plain, at least during the Pleistocene, occurred during interglacial periods. (Reference 2.5.1-209)

Although the northern part of the Appalachian region was covered by continental glaciers at least three times during the Pleistocene, the farthest advance occurred near latitude 40 degrees. Thus glaciation did not extend into the site region. (Reference 2.5.1-221) Surficial features that appear to be related to processes that operated during the cold periods of the Pleistocene, including block fields and stone stripes, have been reported in the unglaciated Appalachians, including in the site region (Reference 2.5.1-221).

2.5.1.1.3 Regional Stratigraphy

The complex stratigraphy and lithology of the HAR site region reflects the long and complex geologic history of this region (described in Subsection 2.5.1.2). The geologic units within a 320-km (200-mi.) radius of the HAR site are shown on Figure 2.5.1-207. The geologic units tend to be distributed in northeast-southwest-trending parallel-sided zones throughout the site region. A comparison of Figures 2.5.1-207 and 2.5.1-201 (which show the physiographic provinces of the site region) indicates that the northeast-southwest-trending linear physiographic provinces have similar stratigraphies, reflecting the similar geologic histories within each zone. The broad northeast-southwest-trending lithotectonic belts include the Piedmont, Blue Ridge, Coastal Plain, Valley and Ridge, and Appalachian Plateau provinces. With the exception of the Coastal Plain that unconformably overlies the Piedmont, these provinces are bounded by fault systems related to Paleozoic folding and faulting. Subdividing the broad physiographic units provides a basis for the discussion of regional stratigraphy in this report subsection. The discussion in the following subsections is focused on the stratigraphy of North Carolina and the geologic belts recognized in the state, as shown on Figure 2.5.1-209. Maps of the interpreted lithotectonic terrane boundaries within the site region are shown on Figures 2.5.1-244 and 2.5.1-245. A lithotectonic map of the Appalachian orogen that reflects current concepts regarding the age and tectonic associations of lithologic units within the study region is presented on Figure 2.5.1-246.

2.5.1.1.3.1 Stratigraphy of the Piedmont Province

The Piedmont region in North Carolina is underlain by predominantly crystalline and volcaniclastic rocks of Precambrian and Paleozoic age (Reference 2.5.1-202). These rocks can be divided into several broad northeast-southwest-trending belts based on the differences in metamorphic grade (Figure 2.5.1-209). To the northwest, the Piedmont is separated from the Blue Ridge province by the Brevard fault zone. To the southeast, the physiographic province extends to the Fall Line, where it is unconformably overlain by Cretaceous and Tertiary sedimentary rocks of the Atlantic Coastal Plain. The eastern boundary of the Piedmont lithotectonic province is thought to extend southeastward beneath the Atlantic Coastal Plain to the East Coast magnetic anomaly (Reference 2.5.1-352). The Piedmont province can be subdivided into the Piedmont zone and the Carolina zone based on tectonothermal histories. The two are separated by the central Piedmont shear zone that is interpreted to be a late Paleozoic thrust fault, previously believed to

be a suture zone (Reference 2.5.1-352). In the Piedmont zone, metamorphic grade is highest in the westernmost Piedmont belt and the Inner Piedmont, which contains high-grade amphibolite gneisses and schists. To the east in the Carolina zone, the Charlotte belt and the Raleigh belt contain rocks of lower amphibolite grade. Metavolcanic rocks and metasediments of mostly greenschist grade characterize the Carolina slate belt and the Eastern slate belt. The Triassic Deep River basin, in which the HAR site is located, is a sediment-filled trough at the eastern margin of the Piedmont physiographic province that is part of a Triassic basins lithotectonic terrane. Cretaceous sediments overlap the southern margins of the Eastern slate belt and the Raleigh belt, as well as the southeastern margin of the Deep River basin. Scattered small deposits of Eocene age locally overlie Cretaceous sediments and crystalline rocks of the Raleigh belt (Figure 2.5.1-207). (Reference 2.5.1-202) The description of stratigraphic units provided in the following subsections comes from the HNP FSAR, unless otherwise noted.

2.5.1.1.3.1.1 Precambrian and Paleozoic Rocks

Precambrian and Paleozoic rocks in the site region have been grouped into three broad categories, as follows (Reference 2.5.1-202):

- Metavolcanic rocks of the Carolina and Eastern slate belts.
- Gneisses and schists of the Raleigh belt.
- Intrusive rocks, predominantly of granitic composition, present in the Raleigh belt and both slate belts.

2.5.1.1.3.1.1.1.1 Metavolcanic Rocks

The metavolcanic rocks of the Carolina and Eastern slate belts include lithic and crystal tuffs, welded flow tuffs, flows, volcanic breccias, volcanic conglomerates, graywacke conglomerates, argillites, slates, phyllites, and thin limestone beds. Their aggregate thickness is believed to be at least 9000 m (30,000 ft.) in the Carolina slate belt. These rocks have been subjected to low-grade, greenschist facies metamorphism and in places have a well-developed slaty cleavage. (Reference 2.5.1-202)

2.5.1.1.3.1.1.1.2 Bedded Argillites

Rocks mapped as bedded argillites include finely bedded metashale exhibiting graded bedding, slate, phyllite, and phyllonite, as well as argillite. Graywacke and tuff are present in places. This group is represented by slates in the western part of the site region and by phyllites and phyllonites in the eastern part of southern Wake County. The rocks are deeply weathered in most places; outcrops of fresh rock are rare.

2.5.1.1.3.1.1.1.3 Felsic Volcanics

This group consists largely of volcanic fragmental and flow materials. The fragmental rocks, which range in composition from rhyolitic to dacitic, consist mostly of coarse and fine tuffs and subordinate breccias. The coarse tuff predominates; it contains the breccia and fine tuff in the form of interbedded bands and lenses. In places, the finer material grades into bedded argillite. The flows, which are essentially rhyolite, occur as narrow bands or lenses interbedded with the tuff and breccia. Small lenses of bedded slate and mafic volcanics also are present within this group. Felsic volcanics weather to a light gray soil commonly underlain by yellow to light red clay.

2.5.1.1.3.1.1.1.4 Mafic Volcanics

Mafic volcanics consist largely of flows ranging in composition from andesite to basalt and of fragmental rocks of mostly andesitic composition. The fragmental rocks are mostly tuffs, breccias, conglomerates, and graywackes. Andesite and basalt flows occur as narrow bands and lenses interbedded with the fragmental rocks. In places the andesite is dark green and coarsely porphyritic, but most commonly is fine grained and massive. The basalt is dark gray to nearly black and commonly amygdaloidal. Most rocks in this group weather to a dark red to maroon clayey residuum (Reference 2.5.1-202).

2.5.1.1.3.1.1.2 Gneisses and Schists

In the Raleigh belt, metamorphic rocks of amphibolite grade include hornblende gneisses, mica gneisses and schists, and felsic gneisses. The hornblende gneisses are mostly medium to coarse grained, massive amphibolites or well-foliated hornblende gneisses. These rocks commonly contain plagioclase (oligoclase) and quartz in various amounts, along with lesser amounts of biotite and epidote.

Mica gneisses and schists, which are medium to coarse grained, consist predominantly of feldspar, quartz, muscovite, and biotite. Biotite is more common in the gneisses, and muscovite is more common in the schists. Felsic gneisses are light-colored, medium-grained rocks characterized by high quartz and microline content and a predominance of muscovite over biotite. Interlayered with these gneisses are garnet-mica schist and two persistent belts of graphite schist (Reference 2.5.1-202).

2.5.1.1.3.1.1.3 Intrusive Rocks

A variety of intrusive igneous rocks, of which granitic rocks are by far the most common, intrude the gneisses, schists, and metavolcanics in the site region. These intrusive bodies range in size from small lenses to large single plutons and plutonic complexes.

Granitic intrusives in the site region include granite, adamellite, granodiorite, quartz diorite, and tonalite. Most of these can be classified into two broad groups, one pre-dating and the other post-dating major regional metamorphism. The nature of pre-metamorphic intrusives in this group is indicated by foliation that is parallel to regional trends and by microtextures that show cataclasis and recrystallization. Post-metamorphic granitic plutons contain massive, even-grained rocks that show little evidence of metamorphism. Most rocks in this group are medium to coarse grained and light to medium gray or light to medium pink (Reference 2.5.1-202).

2.5.1.1.3.1.1.4 Geochronology of Precambrian and Paleozoic Rocks

The Piedmont province of central Virginia contains the Goochland terrane, which trends northeast-southwest and adjoins the Raleigh belt of North Carolina. Age dates obtained from the State Farm gneiss, one of the major units in the Goochland terrane, are interpreted as being igneous crystallization and indicating a maximum emplacement interval of about 1057 to 1013 Ma (Reference 2.5.1-222).

Age dates from several rock types, including metavolcanics and granites, near Chapel Hill, North Carolina, document that the earliest known stage of magmatism in the Carolina terrane began by about 633 Ma and continued until at least 612 Ma. The rock units studied lack any evidence of an older, reworked zircon component, suggesting that the oldest part of the Carolina terrane was built on oceanic crust away from the influence of a continental crust. (Reference 2.5.1-223)

Metavolcanic and metaplutonic rocks of the Carolina terrane in South Carolina, similar to the rock association found in the Piedmont province of central North Carolina, were evaluated by Dennis and Shervais to obtain geochronologic data (Reference 2.5.1-224). Their work indicates that the Carolina terrane experienced regional metamorphism and heterogeneous penetrative deformation accompanied by arc magmatism between about 570 and 535 Ma. This association is interpreted to indicate rifting of a pre-existing volcanic arc.

2.5.1.1.3.1.2 Mesozoic Rocks

Mesozoic sedimentary and igneous rocks in the part of the site region that is located in the Piedmont physiographic province are found primarily in exposed rift basins. The basins were superimposed along Paleozoic structures that were reactivated within the Piedmont province (Reference 2.5.1-220). In North Carolina, these Mesozoic rift basins occur in two subparallel belts that strike northeasterly: the eastern belt includes the Deep River basin and the western belt includes the Dan River basin. Other Mesozoic basins, both exposed and buried, occur within the adjoining states of Virginia and South Carolina (Reference 2.5.1-220; Figure 2.5.1-210). The tectonic origin and deposition of the Mesozoic rift basins are described in more detail in Subsection 2.5.1.1.4.2.3.

2.5.1.1.3.1.2.1 Triassic Sediments

The sediments within exposed basins in the study region (Danville-Dan River, Deep River, Richmond, Taylorsville, Farmville, and associated basins) consist primarily of fluvial and lacustrine sequences of Late Triassic age (Reference 2.5.1-225). The depositional environments of the clastic sediments that comprise the basin fill that was deposited during the rifting phase were controlled strongly by local basin tectonics. Alluvial fans prograded into each basin from the topographically higher, faulted margins. Sediment transported along a basin axis by meandering river systems was deposited in large alluvial plains. Deltaic (delta), lacustrine (lake), and paludal (swamp) deposits accumulated in fresh-water lakes formed in basin depocenters. Rocks of the Triassic Deep River basin in which the HAR site is located are included in the Chatham Group of the Newark Supergroup. These rocks include varying amounts of conglomerates, sandstone, siltstone, claystone, shale, and coal, along with small amounts of limestone and chert (and gypsum in cuttings from several wells). (Reference 2.5.1-226, Reference 2.5.1-204)

The HNP FSAR contains descriptions of the Triassic sedimentary rocks of the Deep River basin. The three-layer system includes a lower unit, the Pekin Formation; a middle unit, the Cumnock Formation; and an upper unit, the Sanford Formation (Reference 2.5.1-202). More recent mapping in the vicinity of the Colon cross structure and southern Durham basin adopts a lithofacies system of mapping (Reference 2.5.1-204). The Triassic sedimentary rocks within the Deep River basin in the site vicinity are described in detail in Subsection 2.5.1.2.3.

2.5.1.1.3.1.2.2 Triassic-Jurassic Diabase

The HNP FSAR includes descriptions of the diabase intrusions in the study region (Reference 2.5.1-202). These intrusions are common within the Raleigh belt, Carolina slate belt, and Deep River basin. The intrusions most commonly have the form of dikes, which range from several centimeters to a few hundred meters (a few inches to several hundred feet) in width and from a meter to several kilometers (a few feet to several miles) in length. Sills and sill-like masses of diabase, present in the Deep River basin, can reach 120 m (400 ft.) in thickness.

The diabase dikes typically are dark gray to greenish black, fine to medium grained, and composed of about 50 percent plagioclase; 25 to 30 percent augite; 10 to 20 percent olivine; and accessory magnetite, ilmenite, apatite, pyrite, titanite, biotite, and amphibole. Many of the dikes in the Deep River basin and the Carolina slate belt show the effects of zeolitization.

2.5.1.1.3.1.2.3 Geochronology of Mesozoic Rocks

Fossil assemblages from the Pekin, Cumnock, and Sanford formations of the Deep River basin have been used to estimate the ages of those rocks. The

fossils consist of pollen and spores, fish, and mammal-like reptiles. The assemblages indicate sediment deposition from the middle Carnian through early Norian ages of the Triassic period, roughly 225 to 220 Ma (Reference 2.5.1-227). Correlation of the Deep River basin rocks with those of the Dan River basin indicate a similar age for the Dan River Triassic rocks. (Reference 2.5.1-220)

Intrusion of diabase sheets and extrusion of basalt flows occurred in the Early Jurassic during the second period of Mesozoic extension (Reference 2.5.1-220). Radiometric dates from the Newark Supergroup basalts, which include the diabase intrusions in the HAR site region, give a mean age of 203 Ma, with the median at 198 Ma (Reference 2.5.1-225).

2.5.1.1.3.1.2.4 Cenozoic Rocks and Sediments

Post-rift rocks in the Piedmont province are limited in extent. The eastern margin of the Piedmont and the western margin of the Coastal Plain are very ragged and in places are not well defined (Reference 2.5.1-209). Within the site region, the Eocene Castle Hayne Limestone occurs as scattered deposits lying unconformably on crystalline rocks of the Raleigh belt. The formation consists of light gray fossiliferous limestone and light gray marl. (Reference 2.5.1-202) In the eastern part of the site region, the Yorktown Formation of Miocene age locally overlaps Eastern slate belt rocks. This unit reveals primarily clay, sand, and shell marl in surface exposures. At one time, these Tertiary marine deposits evidently covered much of the neighboring Coastal Plain and extended onto the Piedmont Plateau. (Reference 2.5.1-202)

Upland terrace sediments of unknown age are mapped in the Piedmont and along the inner edge of the Coastal Plain. Small patches of undifferentiated upland gravel and sand of probable Pliocene age are observed on the highest areas of drainage divides in Wake County. The sediment is described as deeply weathered, poorly sorted, and containing subangular to rounded pebbles. Upland gravel units are identified primarily by the rounded quartz pebbles in the soil. The origin of these deposits is interpreted to be primarily fluvial, although locally colluvial. (Reference 2.5.1-228)

Younger terrace deposits inset below the highest interfluve areas are present along the Deep, Haw, and Cape Fear rivers and along some of their larger tributaries (Reference 2.5.1-202).

Surficial sediments in the Piedmont province generally consist of residual and transported material. The major product of weathering of bedrock in the Piedmont (and Blue Ridge) provinces is saprolite, a type of rock that has been subjected to intense chemical weathering. Thicknesses of saprolite vary widely, but the average rarely exceeds approximately 18 m (60 ft.). (Reference 2.5.1-209) The major processes that have resulted in the formation of saprolite and the evolution of the landscape in the Piedmont province are described by Cleaves (Reference 2.5.1-229).

2.5.1.1.3.2 Stratigraphy of the Blue Ridge Province

The Blue Ridge physiographic province of the southern Appalachian orogen can be divided into western and eastern zones in westernmost North Carolina and the adjoining area of Tennessee (Reference 2.5.1-230). The province is positioned adjacent to the Valley and Ridge province to the northwest and the Piedmont province to the southeast, the boundary of which is the Brevard fault zone. An abrupt transition exists between the lower Paleozoic sedimentary and metasedimentary rocks involved in the adjacent Valley and Ridge thrust sheets to the late Precambrian sedimentary and metasedimentary rocks and some basement in the Blue Ridge (Reference 2.5.1-215). A major boundary fault with large displacements does not correspond to principal changes in metamorphic grade and, therefore, no single fault or fault zone can be identified as the contact between metamorphic core (Blue Ridge) and the relatively unmetamorphosed foreland (Valley and Ridge) (Reference 2.5.1-215). The Western Blue Ridge contains a succession of progressively younger Grenville and Neoproterozoic granitic rocks, mafic dikes and intrusions, and overlying metasedimentary rocks that experienced relatively low-grade Paleozoic metamorphism. The Eastern Blue Ridge sequence consists of Neoproterozoic – early Paleozoic clastic metasedimentary rocks and mafic to ultramafic bodies of higher metamorphic grade, plus felsic Paleozoic intrusions and sparse exposures of Grenville-age granitoid gneisses. The Western Blue Ridge generally is thought to be part of Laurentia (native North America), whereas the Eastern Blue Ridge, structurally and lithologically more complex than the western zone, is considered to comprise one or more exotic or suspect terranes (Reference 2.5.1-230). Locally separating the Western Blue Ridge and the Eastern Blue Ridge is the Mars Hill terrane that is considered to be a piece of Paleoproterozoic crust (Reference 2.5.1-230). This exposure is in the vicinity of Roan Mountain, Tennessee – North Carolina, to the northwest of Asheville, North Carolina, and has been included by some as part of the Western Blue Ridge, but has no counterpart (Figure 2.5.1-211; Reference 2.5.1-230). The oldest rocks in the Blue Ridge province are of Middle Proterozoic age and occur in the middle of the zone. These rocks are felsic gneiss derived from sedimentary and igneous rocks, which locally and variably are interlayered with amphibolites, calc-silicate granofels, and rare marble. (Reference 2.5.1-208)

Late Proterozoic rocks border the Blue Ridge province on the northwest and southeast. Rocks along the western border are clastic metasedimentary and metavolcanic rocks that include slate, metasiltstone, schist, metagraywacke, calc-silicate granofels, quartzite, and felsic metavolcanic rock. These rocks are a part of the Ocoee Supergroup, Grandfather Mountain Formation, Mount Rogers Formation, and quartzite of the Sauratown Mountains anticlinorium. Rocks along the southeastern border are clastic metasedimentary rock and mafic and felsic metavolcanic rock, including gneiss, schist, metagraywacke, amphibolite, and calc-silicate granofels of the Ashe Metasedimentary Suite, Tallulah Falls Formation, and Alligator Back Formation. (Reference 2.5.1-208)

Surficial deposits in the unglaciated Blue Ridge physiographic province include alluvial stream terrace deposits, alluvial and debris-flow deposits, and hillslope colluvium (Reference 2.5.1-231).

2.5.1.1.3.2.1 Geochronology of the Blue Ridge

The Proterozoic Grenvillian crystalline rocks serve as the basement upon which late Precambrian and younger stratigraphic packages that ultimately became involved in the Appalachian orogenies were deposited (Reference 2.5.1-215).The Blue Ridge is an elongate external basement massif along which late Precambrian syn-rift sedimentary and volcanic rocks, as well as older basement rocks, have been translated and deformed by younger Appalachian compressional structures, especially large-scale Alleghanian (late Paleozoic) thrust faults (Reference 2.5.1-353).The rocks within the Blue Ridge province make up most of the basement rocks in the southern Appalachians and are considered to be uniformly broadly granitic in composition (Reference 2.5.1-230). In the northwestern Blue Ridge, rift-related rocks are laterally discontinuous and overlie Precambrian (~1000 Ma) crystalline basement. In the southeastern Blue Ridge, rift-related rocks and continental basement rocks are truncated by thrust faults (Reference 2.5.1-353). Basement rocks in the southern Appalachian are believed to be Grenville age (1000 to 1250 Ma), based on whole-rock rubidium-strontium isochrons and conventional uranium-lead dating methods (Reference 2.5.1-232). Ages of basement rocks in the southern Appalachians also may be documented in younger plutonic rocks and metasedimentary rocks as inherited or detrital zircon (Reference 2.5.1-232). Analyses of zircons from southern Appalachian basement rocks (sampled in a northeast-southwest-trending zone that includes western North Carolina) have delineated a granitic magmatic pulse from about 1165 to 1150 Ma (Reference 2.5.1-232).

The Mars Hill terrane in northwestern North Carolina and southeastern Tennessee is a lithologically diverse basement exposure that is distinct in age and metamorphic history from structurally adjacent sections of the Blue Ridge province (Reference 2.5.1-230). The Mars Hill terrane is interpreted as a continental fragment that may represent a portion of lower crust that underlies the more juvenile rocks of the Western Blue Ridge and possibly the Eastern Blue Ridge basement during Grenville time (Reference 2.5.1-230). The Mars Hill terrane is characterized by a great diversity of lithologies interspersed on all scales (Reference 2.5.1-230). Much of the Mars Hill terrane can be characterized by block-in-matrix mélange. This lithology is cut by dikes of the 730-Ma Neoproterozoic Bakersville dike swarm (Reference 2.5.1-230). Basement of the adjacent Eastern and Western Blue Ridge comprises relatively homogeneous granitic rocks, 1100 to 1200 Ma in age. Field relations and geochronology suggest that the lithologically diverse Mars Hill terrane may have an age of about 1900 Ma, thus containing the oldest rocks in the southern Appalachians; the terrane lacks Phanerozoic rocks (Reference 2.5.1-230). These rocks, as well as those in the adjoining Eastern and Western Blue Ridge, experienced a profound metamorphic event shortly before 1000 Ma (Reference 2.5.1-230).

2.5.1.1.3.3 Stratigraphy of the Coastal Plain Province

In the southeastern part of the site region, the Coastal Plain province lies as a northeast-southwest-trending band (Figure 2.5.1-209). Paleozoic rocks exposed in the Appalachian Mountains to the west can be traced beneath the cover of post-orogenic Mesozoic-Cenozoic Coastal Plain strata (Reference 2.5.1-217). The sediments of the Carolina Coastal Plain were deposited during transgressive-regressive cycles caused by eustatic sea-level fluctuations. Sequences of deposits from successive transgressive-regressive cycles are preserved along the Coastal Plain, with progressively younger sequences lying nearer the modern coast and topographically lower than older sequences. (Reference 2.5.1-209)

The Cretaceous rocks of the Carolinas consist primarily of a succession of sands and clays deposited in delta to marine shelf environments during the late Cretaceous (Reference 2.5.1-234). From oldest to youngest, these eastward-dipping map units are the Cape Fear, Middendorf, Black Creek, and Peedee formations (Figure 2.5.1-247; Reference 2.5.1-235). Generally, the Cape Fear and Middendorf units are fluvial, the Blackcreek is mostly back-barrier, estuarine to tidal flat, and the Peedee is open-shelf in environment (Reference 2.5.1-235).

Several post-Cretaceous units overlie the Cretaceous formations, and the surficial stratigraphies and ages of the deposits and associated geomorphic features, such as wave-cut shorelines, have been used to define geomorphic subdivisions. In South Carolina, three subdivisions are recognized: an Upper, Middle, and Outer Coastal Plain. In North Carolina, the Middle Coastal Plain has been included in the Outer Coastal Plain (Reference 2.5.1-209; Figure 2.5.1-248).

In general, the Upper Coastal Plain is underlain by Cretaceous and dissected remnants of Tertiary sediments that unconformably onlap Mesozoic to Precambrian rocks. The Cretaceous and Tertiary units that crop out on the Upper Coastal Plain are highly dissected, and all original, constructional topography has been deeply eroded (Reference 2.5.1-209). Remnants of old and formerly more continuous fluvial deposits of probable Pliocene age occur on drainage divides in Wake County inland from the contiguous Coastal Plain deposits, suggesting that the Coastal Plain margin extended farther inland prior to erosion (Reference 2.5.1-228).

Seaward of the Orangeburg scarp, which marks the outer boundary of the Upper Coastal Plain, constructional topography generally is preserved, and the land surface is underlain by younger sediments of several transgressive-regressive cycles (Reference 2.5.1-209).The Middle Coastal Plain (the western part of the Outer Coastal Plain in North Carolina) is underlain by Pliocene marine sediments that have local overlays of Quaternary eolian, lacustrine, colluvial, and alluvial deposits (Reference 2.5.1-233).

The Lower Coastal Plain, which is of Quaternary age, is underlain by a repetitive, cyclic sequence of Pleistocene and Holocene marine, estuarine, fluvial, and local eolian deposits, including the Waccamaw, Penholoway, Socastee, and Wando formations (Figure 2.5.1-249). (Reference 2.5.1-233) Erosion and deposition, similar to that which formed the Pliocene Bear Bluff Formation, occurred in younger early Pleistocene sequences (1.9 to 1.5 Ma), resulting in deposition of the Waccamaw Formation directly on Cretaceous units (Reference 2.5.1-235).

Within 80 km (50 mi.) of the HAR site, several post-Cretaceous units overlie the Cretaceous formations along the Cape Fear River and its immediate vicinity, including (1) remnants of the middle Eocene Castle Hayne Formation; (2) lag deposits that indicate deposition during the late Miocene or early Pliocene; (3) the upper Pliocene Duplin and Bear Bluff formations; and (4) the lower Pleistocene (?)[b] Waccamaw Formation (Figure 2.5.1-249; Reference 2.5.1-235). The late Tertiary through early Pleistocene beds on Cape Fear occur in successive, step-wise order, with the oldest unit, the Duplin Formation (3.0 Ma), farthest updip and directly overlying the Cretaceous beds (Figure 2.5.1-250; Reference 2.5.1-235). Downdip of that is the Bear Bluff Formation (2.5 Ma), which was deposited during a subsequent transgression in which the sea beveled off the Duplin sediments and deposited the Bear Bluff sediments directly on Cretaceous beds (Reference 2.5.1-235).

2.5.1.1.3.4 Stratigraphy of the Valley and Ridge Province

The Ridge and Valley province contains 9000 to 12,000 m (30,000 to 40,000 ft.) of Paleozoic sedimentary rocks of predominantly early Paleozoic age. The province is the leading edge of the Paleozoic fold and thrust belt and is thought to have deformed last in the southern and central Appalachians along with the western Blue Ridge (Reference 2.5.1-215). The lithotectonic province is between the Blue Ridge province to the southeast and the Appalachian Plateau province to the northwest. The eastern boundary of the province defines a change from lesser deformed Paleozoic sedimentary rocks to highly folded and faulted Precambrian rocks in the Blue Ridge province. Folding and faulting is extensive in the province; topographic forms are strongly influenced by alternating stratigraphic layers of resistant and nonresistant rocks. (Reference 2.5.1-206) Most of the province consists of alternating ridges and valleys. The valleys typically are underlain by carbonates, and the ridges are composed chiefly of orthoquartzite (Reference 2.5.1-236).

The Quaternary record in the Valley and Ridge, like the rest of the Appalachians, is thin, discontinuous, and difficult to date. Quaternary deposits include alluvial stream and fan deposits and hillslope colluvium. (Reference 2.5.1-231) Higher, older stream terraces have been recognized and dated in the Valley and Ridge

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b. Indicates uncertainty in age or type of fault.

physiographic province in the northwestern part of the study region (e.g., the New River terraces in Virginia [Reference 2.5.1-237]).

2.5.1.1.3.5 Stratigraphy of the Appalachian Plateaus Province

The Appalachian Plateaus province is underlain predominantly by clastic rocks, including conglomerate, sandstone, and shale with some interbedded coal. The Appalachian Plateaus province is bounded by the Ridge and Valley province to the east and the Interior Low Plateaus to the west. The rocks within the Appalachian Plateaus province are predominantly Mississippian and Pennsylvanian in age, and thus are younger than those of other Appalachian provinces. Rocks of the Appalachian Plateaus province were not subjected to the intense deformation that affected the other Appalachian provinces. (Reference 2.5.1-206) The deposits overlie Permian to Cambrian age sedimentary rocks that exhibit little to no deformation.

Similar to other Appalachian provinces, Quaternary deposits include alluvial stream and fan deposits plus hillslope colluvium (Reference 2.5.1-231).

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 EPRI-SOG in 1988, 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 (Reference 2.5.1-203). 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 geologic, seismologic, 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 HAR site. The following subsections describe the region in terms of:

- The contemporary tectonic stress environment (Subsection 2.5.1.1.4.1).
- Primary tectonic structures and zones of seismicity (Subsection 2.5.1.1.4.2).
- Significant seismic sources at distances greater than 320 km (200 mi.) (Subsection 2.5.1.1.4.3).
- Regional gravity and magnetic data (Subsection 2.5.1.1.4.4).

- Historical seismicity is described in Subsection 2.5.1.1.4.2.5.1.

2.5.1.1.4.1 Contemporary Tectonic Stress

The HAR site lies within a compressive midplate stress province characterized by reverse and strike-slip faulting. A relatively uniform east-northeast compressive stress field that extends from the mid-continent east toward the Atlantic continental margin and possibly into the western Atlantic basin was identified by Zoback and Zoback (Reference 2.5.1-238). 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. 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 2.5.1-238). 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 2.5.1-239).

Zoback and Zoback also conclude that the available data do not support a distinct Atlantic Coastal Plain stress province (Reference 2.5.1-238). 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 2.5.1-240). The 1980 reference, which was used by the EPRI-SOG teams, has been superseded by Zoback and Zoback, the findings of which are supported by higher-quality stress data (Reference 2.5.1-238).

Based on analysis of well-constrained focal mechanisms of North American midplate earthquakes, Zoback concludes that earthquakes in the CEUS occur primarily on strike-slip faults that dip between 43 and 80 degrees, primarily in the range of 60 to 75 degrees. The analysis demonstrates that CEUS earthquakes occur primarily in response to a strike-slip stress regime (Reference 2.5.1-241).

2.5.1.1.4.2 Regional Tectonic Structures (within 320-km [200-mi.] Radius)

The concepts of suspect terranes (allochthons) and exotic terranes, which were recognized at the time of the EPRI-SOG study (Reference 2.5.1-203), have been more widely employed to decipher the accretionary history and tectonic evolution of the Appalachian orogen (see discussion in Subsection 2.5.1.1.2) and to define lithotectonic units (Reference 2.5.1-207, Reference 2.5.1-348, Reference 2.5.1-354). A tectonic map of the important structures within the Appalachian orogen in the HAR site region, as defined by Hatcher et al. (Reference 2.5.1-207), is shown on Figure 2.5.1-211; related regional cross

sections are shown on Figure 2.5.1-212. A more recent lithotectonic map of the site region is shown on Figure 2.5.1-246.

Principal tectonic features in the site region's 320-km (200-mi.) radius are shown on Figure 2.5.1-213. The principal tectonic structures within the HAR site region can be divided into five categories based on their age of formation or reactivation: Late Proterozoic to early Paleozoic, Paleozoic, Mesozoic, Cenozoic, and Quaternary time. These categories provide the framework for the discussion that follows.

2.5.1.1.4.2.1 Late Proterozoic to Early Paleozoic Basement Structures

At the time of the EPRI-SOG study, it was recognized that potential seismic sources may lie below the Appalachian detachment or décollement (described later). Subsequent studies have focused on better defining the location and geometry of basement structures. Of significance in the southern Appalachian region are large, known or inferred, normal faults that originally formed along the passive margin of the Late Proterozoic to early Paleozoic Iapetus Ocean. Compressional reactivation of favorably oriented Iapetan faults has been suggested as the causal mechanism for several seismically active regions in the southern Appalachians, including Giles County, Virginia, and eastern Tennessee (see discussion of Quaternary tectonic structures, Subsection 2.5.1.1.4.2.5) (Reference 2.5.1-244). Bollinger and Wheeler suggest that the steep eastward rise seen in the unfiltered Bouguer anomaly field is the eastern limit for Iapetan normal faults and that most of the faults occur in the relatively intact continental crust of North America west of the gravity rise (Reference 2.5.1-245). This gravity rise, referred to as the Appalachian (Piedmont) gravity gradient, is interpreted to mark the transition from thick continental crust to less thick, and possibly more mafic (transitional), crust to the east (Reference 2.5.1-246, Reference 2.5.1-247).

Based on published interpretations of deep seismic reflection profiles across parts of the Appalachians and Coastal Plain, Wheeler infers the southeastern boundary of preserved Iapetan faults to coincide with a narrow zone of intense thinning of Grenville crust that extends along the Appalachians coincident with the Appalachian gravity gradient (Reference 2.5.1-248). Wheeler notes that reflection profiles within and southeast of this zone of intense thinning show structures that disrupted or destroyed the Grenville crust, and any Iapetan faults within it, during Paleozoic compressional and Mesozoic extensional deformation (Reference 2.5.1-244). Bollinger and Wheeler note that Iapetan normal faults likely decrease in size, abundance, and slip gradually and irregularly northwestward into the North American craton over a distance of perhaps 100 to 200 km (60 to 120 mi.) (Reference 2.5.1-245). The northwest boundary for Iapetan normal faults is based on the northwesternmost locations of known Iapetan faults, both seismically active and currently aseismic. This boundary coincides approximately with the transition from a more seismically active continental rim on the southeast to a generally less active cratonic interior toward the northwest. (Reference 2.5.1-244)

Hatcher et al. (Reference 2.5.1-355) present a regional structure contour map of the basement surface beneath the Valley and Ridge and Blue Ridge and Piedmont region of Alabama, Georgia, Tennessee, the Carolinas, and southwest Virginia. The basement surface is inferred from seismic reflection and surface geologic data obtained by industry, academia, and U.S. and state geological surveys, along with crustal seismic lines in the more internal parts of the orogen. The basement surface in this reconstruction, which dips gently southeast in the Tennessee embayment from Virginia to Georgia, contains several previously unrecognized rift-related normal faults that formed in the latest Proterozoic to earliest Cambrian (Reference 2.5.1-250).

2.5.1.1.4.2.2 Paleozoic Tectonic Structures

The central and southern Appalachians were collectively assembled into various terranes by a series of Paleozoic orogenies. All suspect terranes west of the Carolina and Albermarle volcanic arcs, including those amalgamated during the Penobscottian event, were accreted to the Laurentian native terranes and initially deformed and metamorphosed together during the Taconian orogeny. The Acadian orogeny in the southern and central Appalachians preceded the late Paleozoic Alleghanian orogeny; the significance of the Acadian orogeny in the southern and central Appalachians is uncertain (Figure 2.5.1-205).The collision of Laurentia and Gondwanaland marked the final stage of accretionary history in the Appalachians, and is responsible for the reactivation of earlier Paleozoic structures and the formation of folding and thrusting in the Blue Ridge and Ridge and Valley provinces (Reference 2.5.1-348).

Numerous tectonic structures that experienced Alleghanian deformation are exposed in the western half of the 320-km (200-mi.) radius around the HAR site region. These include the Brevard fault, Central Piedmont shear zone, Nutbrush Creek, Gold Hill/Hollister, and Augusta faults (Figure 2.5.1-214). In the eastern half of the site region, similar structures underlie the post-orogenic Mesozoic-Cenozoic sediments of the Coastal Plain. These structures, including the sub-Coastal Plain sediment extensions of the Nutbush Creek and Hollister faults, the Roanoke Island – Goldsboro fault, and inferred Pender fault, have been recognized using data sources that include exploration wells and geophysical techniques (Reference 2.5.1-217). The Alleghanian orogeny in the central and southern Appalachians is known as a thrust-dominated orogen (Reference 2.5.1-218). Alternating weak and strong horizons that assisted in the propagation of Alleghanian thrusts were provided by Paleozoic foreland basins and clastic wedges. The Blue Ridge – Piedmont (BRP) composite crystalline thrust sheet had a major role in Alleghanian deformation. (Reference 2.5.1-218)

The BRP thrust sheet is one of the largest intact composite crystalline thrust sheets in the world, extending from Alabama to Pennsylvania throughout the southern and central Appalachians. The BRP thrust sheet is bounded on the west by the Blue Ridge fault system. The eastern boundary probably is buried beneath the Coastal Plain. (Reference 2.5.1-207) Formed during the collision of North America with Africa, the thrust sheet drove the foreland deformation in front

of it. Accordingly, the BRP thrust sheet is composed of Paleozoic basement made up of intact, complexly deformed Late Proterozoic and early Paleozoic sedimentary sequences. (Reference 2.5.1-207) Several overprinted episodes of faulting are recognized in southern Virginia and western North Carolina.

Evidence for the extent of the Alleghanian BRP thrust sheet beneath the Blue Ridge and Piedmont provinces is derived from both geophysical and structural data. The acquisition and interpretation of seismic reflection profiles developed across the southern Appalachians by the Consortium for Continental Reflection Profiling (COCORP) in the late 1970s and early 1980s together with the subsequent interpretation of industry seismic data provided significant subsurface information to support a model for the development of the Appalachian thrust belt above a master décollement or detachment (Reference 2.5.1-252, Reference 2.5.1-253).

The Brevard fault zone is widely considered to be one of the most fundamental tectonic features of the southern Appalachian orogen (Reference 2.5.1-348). The Brevard fault zone is described in detail by Horton and McConnell (Reference 2.5.1-348) as a linear, southeast-dipping belt of mylonitic and cataclastic rocks, typically 1 to 2 km (0.6 to 1.2 mi.) wide, that extends more than 600 km (370 mi.) from the Gulf Coastal Plain onlap in Alabama almost to the North Carolina – Virginia border, where it apparently continues into Virginia as the Bowens Creek fault. The Brevard fault is a complex fault zone that is recognized as a major Alleghanian structure within the BRP thrust sheet (Reference 2.5.1-207). The fault zone has had a complex history that includes Alleghanian dip- and strike-slip motion preceded by a history of dip-slip motion. A summary of earlier studies and models for the origin and structure of the Brevard fault zone is provided in Hatcher (Reference 2.5.1-207, Reference 2.5.1-215, Reference 2.5.1-355). In a summary of literature from the 1970s and early 1980s Horton and McConnell conclude that the Brevard fault zone has experienced a long and complex Paleozoic history, including Taconic, Acadian (debated), and Alleghanian deformation, in which thrusting played an important role (Reference 2.5.1-348). An additional summary of work published on the Brevard fault zone from 1983 to 1988 by Horton and McConnell, contributed to the understanding and importance of the recognition and documentation of late Paleozoic (Alleghanian) dextral strike-slip along the fault zone (Reference 2.5.1-348). Hatcher credits work done in the 1950s through 1970s as recognizing that the Brevard fault is a major crustal break that was therefore speculated to be a continental suture (Reference 2.5.1-215). Hatcher recognized that as the understanding of the fault system has improved, the similarity of stratigraphic assemblages on both sides of the fault clearly indicates that it is not a suture (Reference 2.5.1-215). Whereas the multiple deformation histories of the fault zone are extensive and controversial, diabase dikes preclude post-Jurassic slip on the Brevard fault, and cooling age histories indicate that no slip has occurred on the Brevard fault since the late Paleozoic (Reference 2.5.1-348).

The Central Piedmont shear zone, previously referred to as the central Piedmont suture (Figure 2.5.1-211), is exposed from near the Georgia-Alabama boundary

northward into central Virginia. From central South Carolina to the north, the fault is traceable to and folded into the Kings Mountain belt along the North Carolina – South Carolina border. The Central Piedmont shear zone separates the volcanic island – arc association of the Avalon-Carolina exotic terrane from the rocks of the Inner Piedmont and eastern Blue Ridge block, which have North American plate affinities. (Reference 2.5.1-215) This tectonic boundary was originally viewed as a cryptic suture involving late Paleozoic tectonothermal overprinting of an older fundamental structure, and consequently was termed the central Piedmont suture. Recently however, this contact has been shown to be a Paleozoic shear zone along most of its length without evidence of earlier activity (Reference 2.5.1-352). The structure has been interpreted to be a major late Paleozoic ductile thrust that decapitated the original suture and marks the final emplacement of Carolina zone against Laurentian rocks during the Alleghanian orogeny. Consequently, the name was modified to the Central Piedmont shear zone (Reference 2.5.1-352).

The Eastern Piedmont fault system (Figure 2.5.1-213), which includes the Nutbush Creek fault and Modoc shear zone, extends from Alabama to Virginia and is described by Hatcher et al. (Reference 2.5.1-327) as a series of linear cataclastic zones that coincide with magnetic anomalies and parallel the trend of the Appalachians. These magnetic anomalies coincide with cataclastic rocks and also with a low velocity zone likely due to fracturing of the crystalline rocks during recurrent faulting. (Reference 2.5.1-327) The timing of movement on the fault system is constrained by faulted granites that are estimated to have formed approximately 400 Ma; the faults are intruded by Mesozoic dikes, and the faults do not offset Coastal Plain deposits. The Eastern Piedmont fault system may have been initiated within the slate-belt island arc as it collapsed and was accreted to North America during the early to middle Paleozoic (Reference 2.5.1-327).

The Nutbush Creek fault crops out in the Piedmont west of Raleigh (Reference 2.5.1-231). Based on apparent truncations of magnetic anomalies, the Nutbush Creek fault is inferred to extend south and southwest under the North Carolina Coastal Plain into South Carolina, where it may connect with the Modoc fault of Hatcher et al. (Reference 2.5.1-327). Movement on this fault is estimated to be 160 km (100 mi.) of dextral offset between 312 and 285 Ma.

The Modoc shear zone is a region of high ductile strain that can be traced through the Piedmont province of South Carolina and Georgia. The ductile zone marks the approximate northwestern limit of penetrative deformation and amphibolite-facies metamorphism. The zone trends northeast and can be invariably characterized by a deformed intrusive contact between upper Paleozoic plutonic orthogneiss in the Kiokee belt and older metamorphic and igneous rocks in the Carolina slate belt. The curving of lithologic contacts and structural elements in the zone, along with a very steep surface dip to the northwest, is evidence for a zone of flattening or right-lateral strike-slip displacement. Within the Modoc zone there are limited amounts of brittle faulting

that postdate the ductile shearing but have displacements too small to interrupt the continuity of major lithologic units (Reference 2.5.1-356).

The Augusta fault zone is concealed under the sediments of the Atlantic Coastal Plain for much of its length, and in the vicinity of its exposure in Augusta, Georgia, forms the northwest-trending boundary between the Kiokee belt and the Belair belt. The fault zone is evident as a magnetic anomaly for much of its covered length. The fault zone shows evidence of an early ductile shear history with minor postmetamorphic brittle faulting similar to the Modoc shear zone. The Augusta fault zone is a thrust fault, displacing low-grade rocks of the Belair belt over high-grade rocks of the Kiokee belt. The fault has a surface dip of about 40 degrees SE, traceable by geophysical data to where the dip flattens to less than 8 degrees about 60 km (37 mi.) to the south of the surface trace. (Reference 2.5.1-356)

The Gold Hill – Silver Hill shear zone records dextral strike-slip motion in the Carolina terrane of North Carolina. Geologic similarities and along-strike position of the Deal Creek shear zone (Figure 2.5.1-213) to the southwest suggest that it is an extension of the Gold Hill – Silver Hill shear zone. The two are separated by Inner Piedmont terrane rocks. The age of dextral strike-slip movement along the Gold Hill – Silver Hill fault is constrained between ~400 and ~325 Ma. (Reference 2.5.1-357)

The Sauratown Mountains anticlinorium is a structural window in western North Carolina and southwestern Virginia. The anticlinorium is made up of four stacked thrust sheets, each containing middle Proterozoic basement and an overlying sequence of late Paleozoic to early Cambrian metasedimentary and meta-igneous rocks (Reference 2.5.1-348).

Major thrusting and imbrication is thought to have occurred coincident with and following middle Paleozoic metamorphism. A thrust fault that borders the anticlinorium is thought to be coincident with the Inner Piedmont thrust stack that is related to the Taconic orogeny that occurred from about 440 to 470 Ma in the southern Appalachians (Reference 2.5.1-348).

Lawrence and Hoffman (Reference 2.5.1-251) show four other major basement faults underlying the Coastal Plain: the Hollister fault zone, the Roanoke Island-Goldsboro fault and unnamed southwest-trending splay, and the inferred Pender fault. (Figure 2.5.1-213).The locations of these structures are inferred by Lawrence and Hoffman from cuttings and cores from 124 boreholes to basement, combined with Bouguer gravity and magnetic maps. The Hollister fault crops out in the Piedmont and is mappable as magnetic lineaments to the south under the Coastal Plain cover. Dextral faulting on the Hollister fault zone is estimated to be 25 km (15.5 mi.) between 251 and 292 Ma. (Reference 2.5.1-251)

The Roanoke Island – Goldsboro fault, which is totally concealed by the Coastal Plain cover, is inferred primarily from truncated magnetic anomalies. Possible offsets suggest dextral displacement. The eastern end of the fault is on trend with

a high-angle, south-wall-downthrown fault that cuts the basement surface as well as the lower part of the Cretaceous section, suggesting possible later reactivation of an earlier basement fault. (Reference 2.5.1-251)

The inferred Pender fault is defined by an east-west discontinuity in the magnetic data. If this discontinuity is a fault, it would probably merge with the northeast-trending Piedmont fault system. The southern extensions of several of the north-south-trending faults, including the Hollister fault traces, are truncated by the inferred Pender fault (Reference 2.5.1-251).

2.5.1.1.4.2.3 Mesozoic Tectonic Structures

Post-Paleozoic tectonic activity in the Appalachian orogen is related directly to the breakup of the supercontinent Pangaea (which represented the welded-together North American and African continents) and the subsequent opening of the Atlantic Ocean (Reference 2.5.1-225). Rifting during the Triassic and early Jurassic resulted in the formation of more than 25 rift basins along the Atlantic margin of North America near the Proterozoic shelf edge (Reference 2.5.1-225; Figure 2.5.1-210). Almost all the exposed Mesozoic basins, and presumably the inferred buried basins, appear to have developed along reactivated Paleozoic structures (Reference 2.5.1-220). These structures include low-angle detachment surfaces that were Alleghanian thrust faults in the late Paleozoic, as well as dextral strike-slip faults (Reference 2.5.1-225). Most Mesozoic rift basins along the Atlantic margin are asymmetric, bounded on one side by a system of major high-angle normal faults and on the other side by gently sloping basement having sedimentary overlap or secondary normal faults (Reference 2.5.1-225).

More than a dozen Mesozoic rift basins lie within the region (320-km [200-mi.] radius) of the site. The HAR site area is located within the Deep River basin, the southernmost exposed rift basin; to the south are large buried basins. In North Carolina, the exposed rift basins form two bands: the eastern band includes the Deep River basin and associated outlier Ellerbe and Crowburg basins, and the western band includes the Dan River basin and the smaller outlier Davie County basin. Both the Deep River basin, in which the HAR site is located (Figure 2.5.1-215), and the Dan River basin are half-graben flanked by major normal fault zones toward which the basin strata dip. The Jonesboro fault system is on the eastern border of the Deep River basin. Both basins exhibit stratigraphy common to most of the Newark Supergroup — that is, a sequence of three distinct stratigraphic units that are interpreted to have been deposited in an extensional basin, the areal extent of which increased through time. (Reference 2.5.1-220)

The Deep River basin is approximately 240 km (150 mi.) long and ranges in width from about 8 to 24 km (5 to 15 mi.) (Reference 2.5.1-220). It is divided into three subbasins. From north to south, these are the Durham, Sanford, and Wadesboro basins. The Colon cross structure separates the Durham from the Sanford basin at a point where the Deep River basin narrows significantly.

(Reference 2.5.1-204) The three basins have differing internal stratigraphies. (Reference 2.5.1-220)

The Deep River basin is bordered on the east by the west-dipping, high-angle Jonesboro fault system that comprises normal faults. The Jonesboro fault separates Triassic sedimentary rocks from the Proterozoic and Paleozoic Raleigh metamorphic belt and the Carolina-zone metavolcanic and metasedimentary rocks. The total amount of displacement along the fault is estimated to be a minimum of approximately 3.0 – 4.5 km (1.8 – 2.8 mi.) of dip-slip displacement. (Reference 2.5.1-204) Several intra-basinal faults, both synthetic and antithetic to the Jonesboro fault system, also are recognized in the Deep River basin. The western boundary of the basin is indicated by minor faults, possibly post-depositional (Reference 2.5.1-204).

During the Triassic, the Jonesboro fault probably was similar seismologically to modern-day segmented, active basin-bounding normal faults in extensional tectonic settings. Four fault segments are recognized along the Jonesboro fault. (Reference 2.5.1-254) The HAR site area is located near the boundary zone between the Durham segment and the Holly Springs segment. These two segments account for most of the structural features that developed around the site area, including joint sets (Reference 2.5.1-254). (The Jonesboro fault is discussed in more detail in Subsection 2.5.1.2.4.) The intrabasinal faults accommodated differential displacements on the segments of the Jonesboro fault, resulting in a complex system of fault-bounded blocks within the basin (Reference 2.5.1-254).

2.5.1.1.4.2.4 Cenozoic Tectonic Structures

Arches and Embayments. Regional studies have identified broad areas of uplift and subsidence that have influenced Cenozoic sedimentation. (Reference 2.5.1-242, Reference 2.5.1-255). The uplift, tilting, and subsidence were manifested as arches, embayments, and troughs, which then were modified further by sediment loading (Reference 2.5.1-242). The basin and arch architecture of the U.S. Atlantic margin likely reflects strain associated with the application and removal of vertical loads in the form of denuded landscapes, sedimentary basin deposition, or ice, that manifest as flexural isostatic responses; this broad regional deformation is considered nontectonic in that it is not localized specifically on pre-existing faults (Reference 2.5.1-267). In the Carolinas, the Cape Fear arch, a southeast-dipping basement high, is a prominent feature along the North Carolina – South Carolina border (Figure 2.5.1-213) that has affected the thickness and distribution of strata ranging in age from Late Cretaceous to late Tertiary (Reference 2.5.1-242). Regional uplift on the arch has caused Cretaceous sediments to thin on the axis and sediments to thicken on the flanks. This is evident in the geomorphic differences between the Cape Fear River valley and the adjacent Pee Dee River to the south. (Reference 2.5.1-209). The Pee Dee River valley lies far down the southern limb of the Cape Fear arch and is dominated by finer sediment.

Sustained uplift related to the Cape Fear arch to the north-northeast of the Cape Fear River valley has forced the river to migrate southward over time.

Five river terraces, ranging in age from 2.75 to 0.1 Ma, are present on the northeast side of the Cape Fear River. The terraces are preserved only on the northeast side of the river because the river is eroding its southwest bank. (Reference 2.5.1-266) From analysis of longitudinal terrace profiles in the Cape Fear River valley, it is estimated that localized uplift rates on the Cape Fear arch since the late Pliocene have ranged from 0.06 m (0.2 ft.) to perhaps as much as 6 meters per 100,000 years (20 feet per 100,000 years), while a more regional uplift associated with the arch or Coastal Plain in general has been at a rate of approximately 0.6 m/100,000 years (2 ft./100,000 years) (Reference 2.5.1-209). Based on biostratigraphic data from marine deposits associated with the Cape Fear arch, vertical crustal movements were compared with eustatic sea-level changes, suggesting net post-Miocene vertical uplift rates in the outer Coastal Plain of the Carolinas of 1 to 3 cm per 1000 years (0.4 to 1.2 in. per 1000 years). Holocene tectonic movement of the arch has been implied by leveling and survey data (Reference 2.5.1-266). Crone and Wheeler (Reference 2.5.1-258) classify the Cape Fear arch as a Class C feature, based on lack of evidence for Quaternary faulting.

North of the Cape Fear arch, the North Carolina Coastal Plain consists of two blocks, the Onslow block and, farther north, the Albermarle block, which are separated by the northwest-southeast-trending Neuse hinge (Figure 2.5.1-213), a basement flexure zone that borders the southern part of the Graingers basin and has persisted since the Mesozoic (Reference 2.5.1-255). The Neuse hinge separates the Onslow block to the south and the Albemarle block to the north, delineating the boundary between two crustal blocks that have behaved differently throughout the Mesozoic and Cenozoic and have at times uplifted or subsided relative to each other. At the present time, the Albemarle block is down relative to the Onslow block, creating the Albemarle embayment. (Reference 2.5.1-255) The Neuse hinge has affected the spatial distribution of Paleogene and perhaps Neogene sediments (Reference 2.5.1-255).

The Neuse hinge is not included as a Quaternary structure by Crone and Wheeler (Reference 2.5.1-258) or Wheeler (Reference 2.5.1-259). The Neuse hinge (also referred to as the Neuse fault) was reviewed in detail as part of the HNP FSER (Reference 2.5.1-358). Based on a literature review and discussions with local experts on the geology of the Carolina Coastal Plain, it was noted that the postulated Neuse fault had been mapped in several different locations, that no seismicity is associated with the proposed fault, and that there is no unequivocal evidence that the proposed fault exists or that there has been any post-Cretaceous movement along its proposed alignment (Reference 2.5.1-358). The NRC staff concluded that there is no evidence to indicate that the proposed fault exists, and even if it does exist, there is no evidence that it is a capable fault and thus a hazard to the site (Reference 2.5.1-358). More recent interpretation of basement structure beneath the Coastal Plain section by Lawrence and Hoffman (Reference 2.5.1-251) does not show a fault coincident with the Neuse hinge.

Post-Cretaceous and Cenozoic Faults. (Prowell compilation [Reference 2.5.1-256]). Many investigators have recognized and documented post-rift faulting of Cretaceous and Cenozoic ages in the Atlantic Coastal Plain (Figure 2.5.1-213). Prowell compiled information and described evidence for possible Cretaceous and Cenozoic faults in the eastern United States and identified the Atlantic Coast tectonic province based on the presence of these faults (References 2.5.1-256 and 2.5.1-225). Most of the HAR site region is located within the Atlantic Coast tectonic province.

Most of the faults, which have displacements as great as hundreds of feet, trend roughly parallel to the regional fabric of Precambrian and Paleozoic crystalline rocks and are as long as 100 km (60 mi.) (Reference 2.5.1-242). The dips of faults range from 40 to 85 degrees, and may vary along a fault depending on the physical properties of the rocks in the adjacent fault blocks (Prowell in Reference 2.5.1-225). The most recent movement on many of these faults has been reverse motion due to the compressive stress regime of the southeastern United States (Reference 2.5.1-242) (see Subsection 2.5.1.1.4.1). A component of lateral slip has been reported for some of these reverse faults (Prowell in Reference 2.5.1-225). Slip rates on these faults vary locally from 0.3 to 1.5 m (1 to 5 ft.) per m.y., but during the past 110 m.y. have been relatively uniform at approximately 0.5 m (1.6 ft.) per m.y. (Reference 2.5.1-242).

Table 2.5.1-201 summarizes the ages of and evidence for recent movement on possible Cretaceous and post-Cretaceous faults within 320 km (200 mi.) of the site. Of the 33 Cretaceous and Cenozoic faults identified within the site region (32 localities in the Prowell compilation [Reference 2.5.1-256]) and an additional locality described by Parker (Reference 2.5.1-228), 25 lie within 161 km (100 mi.) of the site (Figure 2.5.1-213), and 5 are within the site vicinity (40-km [25-mi.] radius). Although faults are depicted by Prowell as lines approximately 20 km (12 mi.) in length on a regional map, the faults were observed at point localities and almost none could be traced laterally (Reference 2.5.1-256). Additionally, the latitude and longitude of fault localities have been rounded so that these locations do not match the description of where the faults were observed. Therefore, to portray the faults relative to other structures, the faults on Figure 2.5.1-213 are shown as points based on the description of the location where the fault was observed (e.g., “West side of Norfolk and Southern Railroad about 1080 ft north of County Rt. 2724, about 0.3 mi. northeast of Banks, N.C.”) (Reference 2.5.1-256). Faults that lie within the site vicinity (40-km [25-mi.] radius) are described further in Subsection 2.5.1.2.4.

Graingers Wrench Zone. Post-Paleocene faulting along the Graingers wrench zone has been studied by McLaurin and Harris (Figure 2.5.1-213; Reference 2.5.1-257). The Graingers wrench fault is a reactivated structure that coincides with the western border of the Graingers structural basin, a northeast-southwest-oriented feature delineated on magnetic and gravity maps. The western border of the Graingers basin appears to coincide with an unnamed fault in crystalline basement rocks below the Coastal Plain section, as mapped

by Lawrence and Hoffman (Reference 2.5.1-251). The history of motion on this northeast-southwest-trending fault system and the intervening Graingers basin has been reconstructed using stratigraphic information. The most recently active faults, which developed during the Paleocene, overprint late Cretaceous and early Paleocene east-west fault trends. The combination of the younger northeast-southwest-trending faults superimposed over the older east-west fault geometry has resulted in the Graingers basin area's being broken into a series of fault blocks. McLaurin and Harris suggest that the large-scale control on fault development may have been due to changes in the direction of movement of the North American plate in the Cenozoic post-rift period (Reference 2.5.1-257).

Post-Paleocene fault activity in the Graingers wrench zone is interpreted to be down to the east with a minor component of strike-slip. Preservation of topographic features along the zone, including a fault-line scarp and triangular facets, led to the speculation that the faults have continued to be active into the present (Reference 2.5.1-257). No direct evidence of Quaternary fault activity has been reported, however. The faulting was not included in either the Crone and Wheeler (Reference 2.5.1-258) or Wheeler (Reference 2.5.1-259) compilations of known or suggested Quaternary tectonic faulting in the CEUS.

Tectonic Features in the Charleston 1886 Earthquake Epicentral Region. A number of tectonic features near Charleston, South Carolina, that are within 320 km (200 mi.) of the HAR site have been postulated as possible sources of the Charleston 1886 earthquake. These include the Adams Run, Ashley River, Charleston, Cooke, Sawmill Branch, Summerville, and Woodstock faults, and the southern segment of the postulated ECFS. A discussion of the faults in the meizoseismal region of the Charleston earthquake is provided in Subsection 2.5.1.1.4.3. The postulated ECFS is described in Subsection 2.5.1.1.4.2.5.2.2.

2.5.1.1.4.2.5 Quaternary Tectonic Structures and Seismic Zones

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 2.5.1-258). An update containing new assessments was published by Wheeler (Reference 2.5.1-259). Tectonic features, described by Crone and Wheeler and by Wheeler, within a 320-km (200-mi.) radius (the region) of the HAR site are shown on Figure 2.5.1-216 (Reference 2.5.1-258, Reference 2.5.1-259). The features within the site region are categorized into three classes (Class A, B, or C)[c] based on information about the features' Quaternary activity.

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[c] Crone and Wheeler (Reference 2.5.1-258) 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

Only three features within a 320-km (200-mi.) radius of the site were identified as demonstrating convincing evidence of Quaternary activity (Class A): the central Virginia seismic zone and the Charleston, Blufftown, and Georgetown liquefaction features. Only one feature, the Pembroke faults in Virginia, was identified as having possible evidence of Quaternary deformation, although additional study is required to provide confidence in this assessment (Class B). Features for which geologic evidence is insufficient to demonstrate Quaternary deformation (Class C) are listed and shown on Figure 2.5.1-216. Information on these features was reviewed for this study, but distant features (e.g., the Mtn. Run/Everona fault zone, the Lebanon Church fault, Old Hickory faults, Stanleytown – Villa Heights faults, and Hopewell fault) are not considered significant for the seismic hazard assessment and therefore are not described in this report.

The known or suggested Quaternary tectonic features that may be significant for the assessment of seismic hazard at the HAR site are described in the following subsections:

- Subsection 2.5.1.1.4.2.5.1 describes distinct seismic source zones that were previously recognized by the EPRI-SOG earth science teams (ESTs) and that are also recognized in the Quaternary tectonic features database (Reference 2.5.1-258). These include the Charleston seismic zone (also referred to as the Middleton Place – Summerville seismic zone) (Class A), the Central Virginia seismic zone (Class A), and two other regions of the Valley and Ridge province of the southern Appalachians that exhibit relatively high rates of low- to moderate-magnitude earthquakes (i.e., the Giles County seismic zone [Class C] and Eastern Tennessee seismic zone [Class C]).

- Subsection 2.5.1.1.4.2.5.2 describes postulated neotectonic features that have been evaluated as potential seismic sources. These postulated faults or neotectonic features include the Pembroke faults (Class B), East Coast fault system (Class C), and Weems fall zones (Class C).

- Subsection 2.5.1.1.4.2.5.3 summarizes the results of paleoliquefaction studies that have been conducted within the HAR site region.

2.5.1.1.4.2.5.1 Seismic Zones

Detailed studies of seismicity and field data conducted since completion of the EPRI-SOG study (Reference 2.5.1-203) provide new information regarding the characterization of seismic zones that were previously recognized and considered as seismic sources by the EPRI-SOG ESTs. Additional discussion of

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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.

the seismicity and evidence regarding the magnitude and recurrence of earthquakes within the seismic zones based on the updated earthquake catalog is provided in Subsection 2.5.2.

2.5.1.1.4.2.5.1.1 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 (Reference 2.5.1-304, Reference 2.5.1-305, Reference 2.5.1-306, Reference 2.5.1-290). The MPSSZ lies approximately 300 km (190 mi.) south of the HAR site. 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 2.5.1-306). The elevated seismic activity of the MPSSZ has been attributed to stress concentrations associated with the intersection of the Ashley River and Woodstock faults (Reference 2.5.1-307, Reference 2.5.1-306, Reference 2.5.1-290, Reference 2.5.1-308). Persistent foreshock activity was reported in the MPSSZ area (Reference 2.5.1-296) and it has been speculated that the 1886 Charleston earthquake occurred within the MPSSZ (e.g., Reference 2.5.1-307, Reference 2.5.1-304, Reference 2.5.1-294).

2.5.1.1.4.2.5.1.2 Central Virginia Seismic Zone

At closest distance, the Central Virginia seismic zone (CVSZ) lies approximately 200 km (130 mi.) north of the HAR site. The CVSZ is a roughly circular area that has a diameter of approximately 120 to 145 km (75 to 90 mi.) and contains a low level of seismicity (Reference 2.5.1-258). The geologic evidence for Quaternary faulting consists of two sites at which probable small Holocene sand dikes are interpreted to be paleoliquefaction features (Subsection 2.5.1.1.4.2.5.3.1). The faults that generated the earthquakes that caused these paleoliquefaction features have not been identified.

About three-quarters of the earthquakes in this zone have occurred in the upper 11 km (7 mi.) of crust (Reference 2.5.1-258). Most of the earthquakes occur in an east-west-trending belt of activity, the western limit of which lies east of the Mountain Run fault and the eastern limit of which lies slightly east of the Hylas shear zone (Reference 2.5.1-359). Although hypocenters of micro-earthquakes plot within an upper crustal complex of thrust sheets, the location uncertainties for both the earthquakes and individual faults are large enough that no association can be postulated confidently (Reference 2.5.1-258). Chapman (Reference 2.5.1-359) postulates that earthquakes in the CVSZ may be occurring along older Paleozoic brittle and ductile compressional features that were reactivated during the Mesozoic, creating a fabric of minor high-angle brittle faults with a variety of orientations. The largest historical earthquake in the zone occurred in 1875 near the center of the zone; the intensity was MMI VII with an estimated moment magnitude (**M**) 4.8. Neither surface rupture nor liquefaction was reported from the 1875 event. (Reference 2.5.1-258)

Investigations for the North Anna Early Site Permit (ESP) Application (Reference 2.5.1-272) made the following observations regarding the CVSZ:

- The broad spatial distribution of the seismicity made it difficult to attribute the seismicity to any known geologic structure.
- The seismicity extends both above and below the Appalachian detachment.
- No capable tectonic sources have been identified within the CVSZ.
- Paleoliquefaction sites reflect prehistorical occurrences of seismicity within the CVSZ but do not indicate the presence of a capable tectonic source.

Based on these observations, the North Anna ESP Application concluded that no new information has been developed since 1986 that would require a significant revision to the treatment of the CVSZ in the EPRI seismic source model. Based on a review of seismicity and recent information, including a careful review of the results of previous paleoliquefaction investigations and geologic evaluation of structures, the North Anna ESP Application concluded that the EPRI-SOG seismic source characterizations for the CSVZ did not need to be updated. The North Anna FSER (Reference 2.5.1-360) further states that there is no reported geomorphic expression, historical seismicity, or Quaternary deformation along either the Hylas shear zone or the Lake of the Woods thrust fault that lie within the CVSZ. Diffuse, scattered seismicity occurs throughout the CVSZ, but it is not spatially concentrated or aligned with either of these two structures.

A feature speculated to be of possible tectonic origin was identified in LIDAR imagery in the Shenandoah Valley of Central Virginia by Wieczorek et al. (Reference 2.5.1-260). The northwest-southeast-trending lineament extends for at least 6 km (3.75 mi.) across Tertiary and Quaternary alluvium that overlies Paleozoic formations. Wieczorek et al. interpret sheared saprolite in Paleozoic bedrock along the LIDAR lineament at one location to represent a possible fault (referred to as the Harriston fault) (Reference 2.5.1-260). The linear patterns in the LIDAR image suggest a compressional stepover in a left-lateral strike-slip fault system. The possible fault may be of Paleozoic or Mesozoic age. Both tectonic and nontectonic mechanisms are considered by Wieczorek et al. (Reference 2.5.1-260) to explain the linear patterns in the LIDAR image that coincide with the possible fault. The geomorphic feature that is observed in the LIDAR data may be the result of subsidence caused by dissolution in the underlying karst from groundwater circulating along the fault (i.e., nontectonic). Alternatively, the topographic expression in the alluvium may be the result of Pleistocene or younger tectonic movement on the Harriston fault. The northwest orientation and steep dip of the fault are consistent with focal mechanisms of monitored earthquakes deeper than 8 km (5 mi.) in the CVSZ. However, preliminary trenching investigations to evaluate the postulated fault showed no evidence for faulting in one trench and equivocal evidence (possible aligned gravel clasts) for faulting in a second trench.

Based on a review of literature and available information on the LIDAR lineament, completed for this study, it is concluded that there is no definitive new information that suggests that the EPRI-SOG characterization of the CVSZ as outlined in Subsection 2.5.2.2.1 needs to be updated.

2.5.1.1.4.2.5.1.3 Giles County, Virginia, Seismic Zone

The Giles County seismic zone (GCVSZ), located in southwestern Virginia adjacent to West Virginia, straddles the northwest-flowing New River (Reference 2.5.1-259; Figure 2.5.1-216). At closest distance, the GCVSZ lies approximately 250 km (155 mi.) from the HAR site. This zone of seismicity is approximately 40 km (25 mi.) long, 10 km (6 mi.) wide, and from 5 to 26 km (3 to 16 mi.) deep (Reference 2.5.1-245). Earthquake foci define a tabular zone that strikes north 44 degrees east and has a near-vertical dip within Precambrian basement beneath Appalachian thrust sheets (Reference 2.5.1-245). The largest known earthquake in Virginia, the 1897 Giles County earthquake (MMI VIII, **M** 5.9), is inferred to have occurred within the GCVSZ near the Virginia – West Virginia border (Reference 2.5.1-245, Reference 2.5.1-259).

The GCVSZ probably is a result of compressional reactivation of a late Proterozoic or early Paleozoic Iapetan normal fault or fault zone. Fault reactivation by late Paleozoic compression and Mesozoic extension is also possible. (Reference 2.5.1-245) The strike of the seismic zone is subparallel to the surface and near-surface structures of the central Appalachians and is at an angle of about 30 degrees to the thrust-faulted tectonic fabric. The hypocentral depths of the seismicity, which are below the deepest likely thrust fault, suggest that there is no simple relationship to surface geology (Reference 2.5.1-245).

There is no recognized geomorphic expression of the GCVSZ and no geologic evidence to demonstrate Quaternary surface deformation caused by tectonic faulting above the Appalachian detachment within the seismic zone. Accordingly, Wheeler assigns the GCVSZ to Class C (Reference 2.5.1-259). Detailed studies of the Pembroke faults, which lie within the GCVSZ, favor a nontectonic origin for faulting and antiformal surface deformation (Reference 2.5.1-361) (see discussion of the Pembroke faults in Subsection 2.5.1.1.4.2.5.2.1). Regional Quaternary studies of downcutting rates for the New River, Virginia, region provide evidence only for regional tilting (Reference 2.5.1-237). The occurrence of a significant historical earthquake and of continuing smaller earthquakes in the seismic zone indicates tectonic activity in the area (Reference 2.5.1-259), but definitive evidence for a capable tectonic source and for the recurrence of large earthquakes similar to or larger than the 1897 Giles County earthquake (e.g., results of paleoliquefaction investigations) has not been reported in the literature reviewed for this study. There is no definitive new information that suggests that the EPRI-SOG characterization of the GCVSZ as outlined in Subsection 2.5.2.2.1 needs to be updated.

2.5.1.1.4.2.5.1.4 Eastern Tennessee Seismic Zone

The East Tennessee seismic zone (ETSZ) is a well-defined, northeasterly trending belt of seismicity, approximately 300 km (185 mi.) long by up to about 100 km (60 mi.) wide, that lies principally within the Valley and Ridge physiographic province of eastern Tennessee and adjacent parts of North Carolina just beyond the HAR site region boundary (Reference 2.5.1-259, Reference 2.5.1-278; Figure 2.5.1-216). This area is one of the most active seismic regions in eastern North America (Reference 2.5.1-278). The largest recorded earthquakes in this zone are the 1973 (m_b 4.7) Maryville, Tennessee, earthquake and the April 2003 **M** 4.6 Fort Payne earthquake that occurred in northeastern Alabama near the Georgia border (Reference 2.5.1-279, Reference 2.5.1-280).

Focal depths of most earthquakes in the ETSZ range from 5 to 23 km (3 to 14 mi.) beneath detached Alleghanian thrust sheets (Reference 2.5.1-281, Reference 2.5.1-278). Focal mechanisms indicate strike-slip faulting on steeply dipping planes and a uniform regional stress field in which horizontal maximum compression trends north 70 degrees east (Reference 2.5.1-278). Chapman et al. note that the seismicity is not distributed uniformly; instead, epicenters form northeasterly trending en-echelon segments (Reference 2.5.1-278). One interpretation is that the linear segments and the locations of their terminations may reflect the basement fault structure that is being reactivated in the current stress regime (Reference 2.5.1-278). More recent analyses, using the spatial distribution of relocated earthquake hypocenters combined with focal mechanism solutions, provide additional insights into fault orientation in the ETSZ (Reference 2.5.1-282). These data suggest that the orientation of a diffuse, west-striking, north-dipping zone of hypocenters within the central, most seismically active part of the ETSZ is consistent with reflective, mid- to upper-crust structures having apparent dips of approximately 35 degrees to the north. Relocation of a smaller cluster of earthquakes nearer the Tennessee – North Carolina border suggests possible reactivation of a steeply dipping, northwest-southeast-trending fault in the basement in that area.

The association of seismicity with regional structures in the ETSZ was reviewed by Wheeler (Reference 2.5.1-259). Most earthquakes occurred in crystalline basement rocks buried beneath the exposed thrust sheets of Paleozoic rocks. The structures associated with the earthquakes, therefore, have no clear relationship to structures in the thrust sheets or exposed geology. In this respect, the ETSZ is similar to the GCVSZ, described previously. The faults on which ETSZ earthquakes have occurred may have formed during the late Precambrian or Cambrian rifting that led to the formation of the Iapetan Ocean. (Reference 2.5.1-259)

The majority of the seismic events associated with the ETSZ occur on the Ocoee block, between the Ocoee, Clingman, and New York – Alabama lineaments (Figure 2.5.1-213), which are formed by deep-seated basement structures (Reference 2.5.1-283).

The western margin of the ETSZ is sharply defined, coinciding with a prominent gradient in the total intensity magnetic field, which corresponds to the New York – Alabama lineament (Reference 2.5.1-278). Alternative structural models have been postulated to explain the association of seismicity with the magnetic gradient. Powell et al. propose that the ETSZ is an evolving seismic zone in which slip on north- and east-striking surfaces is slowly coalescing into a northeast-southwest-trending zone (Reference 2.5.1-284). It is suggested that the ETSZ represents seismic activity that results from the regional stress field and is coalescing near the juncture of a relatively weak seismogenic block and the relatively strong crust to the northwest, which may be strengthened by the presence of mafic rocks associated with an inferred Proterozoic rift. (Reference 2.5.1-284) Powell et al. note that the densest seismicity and the largest of the instrumentally located epicenters in the ETSZ generally lie close to and east of a magnetic lineament on the western margin of the seismic zone (Reference 2.5.1-284). It is postulated that deformation within the ETSZ eventually may evolve into a throughgoing strike-slip fault in eastern Tennessee. (Reference 2.5.1-284) Strike-slip motion would be consistent with both the sharp, apparently near-vertical nature of the boundary, as inferred from the magnetic signature, and the orientation of the boundary in the contemporary stress field.

An alternative model to explain the localities of seismicity in the eastern Tennessee region is given by Long and Kaufmann (Reference 2.5.1-285). Based on an analysis of the velocity structure of the region, Long and Kaufmann conclude that the seismically active areas apparently are not constrained by crustal blocks as defined by the magnetic lineament (Reference 2.5.1-285). Rather, their locations are determined by low-velocity regions at mid-crustal depths. The data may support the conjecture that intraplate earthquakes occur in crust that may be weakened by anomalously high fluid pressures (Reference 2.5.1-285, Reference 2.5.1-391).

Detailed geologic studies focused on locating paleoseismic evidence of large-magnitude prehistoric events have been conducted in limited areas only. Whisner et al. investigated a 300-square-kilometer (km^2) (116-square-mile [$mi.^2$]) area within the most active part of the ETSZ and found no concrete evidence of large prehistoric earthquakes (Reference 2.5.1-286). It is noted, however, that two other sites warrant further study. At the Gray fossil site in northeastern Tennessee, fractures and joints having little offset exist throughout Miocene clay units. These features have orientations that are not inconsistent with the late Tertiary to Holocene stress field. In addition, apparent dewatering features are observed in the same region. Deformation is postulated to be related to either strong ground motion or, more likely, sinkhole collapse. At a site in Tellico Plains, Tennessee, disturbed and folded sediments were recently discovered in an older landslide or terrace deposit beneath younger Tellico River alluvium. Deformation at this site may be the result of soft-sediment deformation and liquefaction related to a prehistoric earthquake, or it could be the result of dewatering and folding at the toe of a prehistoric landslide. (Reference 2.5.1-286)

Wheeler (Reference 2.5.1-259) assigns the ETSZ to Class C based on the lack of surficial geologic evidence that clearly demonstrates the occurrence of large earthquakes within the seismic zone.

An NRC-sponsored research effort was initiated in the ETSZ in the last half of 2009 to help clarify the Late Quaternary earthquake history and potential of this seismic zone. At locations east to northeast of Knoxville, TN within Late Quaternary terrace deposits, Vaughn et al. (Reference 2.5.1-391) and Hatcher et al. (Reference 2.5.1-392) reported the occurrence of outcrop-scale strike-slip, reverse, and normal faults and prevalent fractures; minor paleoliquefaction features; and anomalous fractured and disrupted features attributed to liquefaction and forceful expulsion of groundwater during one or more major Late Quaternary earthquakes. These preliminary findings suggest that the ETSZ has a long history of Late Quaternary movement, but the recurrence interval and size of prehistoric earthquakes is yet to be determined. The available data are not sufficient to determine if the ETSZ could be considered a zone of repeated large-magnitude earthquakes. Therefore, despite the occurrence of moderate historical earthquakes and of continuing smaller earthquakes in this seismic zone that indicates tectonic activity in the area, based on the review of literature and new unpublished information, it is concluded that there is no definitive new information that suggests that the EPRI-SOG characterization of the ETSZ as outlined in Subsection 2.5.2.2.1 needs to be updated.

2.5.1.1.4.2.5.2 Postulated Nontectonic Features

2.5.1.1.4.2.5.2.1 Pembroke Faults

Faults in terrace deposits of probable Quaternary age have been mapped along the north side of the New River valley, between Pembroke and Pearisburg, Virginia, approximately 240 km (150 mi.) from the HAR site (Reference 2.5.1-258). The faults overlie a steeply dipping tabular zone of hypocenters (see discussion of the Giles County, Virginia, seismic zone in Subsection 2.5.1.1.4.2.5.1.3). The terrace deposits are underlain by folded and thrust-faulted Ordovician carbonate rocks of the southern Appalachian Valley and Ridge province. The faults were mapped in an excavation and have no topographic expression. The five largest extensional faults have dip separations from 1.0 to at least 8.5 m (3 to at least 28 ft.). These faults, which bound two graben and a half-graben in or near the hinge of an anticline, have a normal sense of slip with a component of strike-slip. The ages of the faults are constrained by the terrace deposits, which could be as old as latest Pliocene. (Reference 2.5.1-258)

Law et al. (Reference 2.5.1-261, Reference 2.5.1-362) discuss alternative models for the formation of the faults by tectonic and nontectonic processes (i.e., surficial processes, associated with either solution collapse or landsliding). Crone and Wheeler rate the faults as Class B because the fault origin has not been determined (Reference 2.5.1-258). A tectonic origin for these structures would involve folding and faulting within the river terrace sediments in response to a

listric extensional fault detaching within the river terrace sediments, whereas a nontectonic origin could be driven by solution collapse of the underlying Cambro-Ordovician limestone. No major sinkholes have been reported in the excavation site area; however, examples of sinkhole formation can be seen in the limestone bedrock immediately to the east (Reference 2.5.1-261). More recent investigation of surface and geophysical subsurface data suggests a model involving initial deposition of sediments within an elongate east-northeast-trending surface depression at the bedrock/sediment interface. Continuous deposition of sediments during progressive solution of limestone at the bedrock/sediment interface leads to eventual inversion of sediment layers to produce an antiformal structure (Reference 2.5.1-361). The depression in the bedrock surface oriented subparallel to the strike of normal faults is further supported by seismic refraction, electrical resistivity sounding, and borehole data (Reference 2.5.1-363).The preservation of delicate grain-scale textures in clay-rich faults indicates slow slip rates that preclude sudden slip (Reference 2.5.1-361). The model of the antiformal structure driven by solution of the underlying limestone coupled with evidence of slow slip rates favors a nontectonic origin of the observed structures.

2.5.1.1.4.2.5.2.2 Postulated 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 East Coast fault system (ECFS), was identified by Marple and Talwani (Figure 2.5.1-213; Reference 2.5.1-243). 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 2.5.1-217). The southern segment is located primarily in South Carolina; the central segment is located primarily in North Carolina and is located approximately 55 km (35 mi.) from the HAR site; and the northern segment extends from northeastern North Carolina through Virginia (Figure 2.5.1-213). 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 2.5.1-243).

The conclusions reached by Marple and Talwani suggest that the ECFS should be considered as a potential capable tectonic source (Reference 2.5.1-243). In addition, 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 (Reference 2.5.1-347, Reference 2.5.1-364). 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 USGS compilation of known or suggested Quaternary tectonic faulting in the CEUS, Wheeler (Reference 2.5.1-259) noted that evidence for the southern section is strongest, with evidence becoming successively weaker northward (Reference 2.5.1-259). 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 (see the descriptions of the Charleston and Georgetown liquefaction features in Subsection 2.5.1.1.4.2.5.3.2) (Reference 2.5.1-259). 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 2.5.1-259)

- 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 2.5.1-274). 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 2.5.1-243), 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 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. A summary of the observations and conclusions from this study regarding the assessment of the ECFS-N as a capable tectonic source are provided below.

- 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 2.5.1-360)

- Southern Nuclear Company (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 2.5.1-264) 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) (Figures 2.5.2-212 and 2.5.2-213).

- 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 magnetic 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 2.5.1-365)

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 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 are 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 Southern Nuclear Company (Subsection 2.5.2.4.1.1.2).

EVALUATION OF THE EVIDENCE PRESENTED BY MARPLE AND TALWANI (2000)

Geologic, geophysical, seismological, and geomorphic data used by Marple and Talwani (Reference 2.5.1-243) to infer the presence of the southern, central, and northern segments of the ECFS are reviewed and evaluated in the following sections.

East Coast Fault System — Southern (ECFS-S) Segment

Marple and Talwani initially identified and proposed the southern segment, located in South Carolina, as a possible source of the 1886 Charleston earthquake (Reference 2.5.1-262). At its southern end, this segment is associated with microseismicity, a linear magnetic anomaly, and buried faults interpreted from seismic reflection data (Reference 2.5.1-243).

The Woodstock fault, which is inferred to be a reactivated fault associated with the eastern margin of a Mesozoic basin, is identified in the subsurface, based on interpretation of seismic profiles, associated seismicity as indicated by detailed analysis of seismicity using double difference methods, and assessment of possible related surface deformation that may have occurred during the 1886 Charleston earthquake. The approximately 30-km- (19-mi.-) long Woodstock fault 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 the fault (see Subsection 2.5.1.1.4.3).

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, there are no mapped faults that coincide with ECFS-S (Figure 2.5.1-219). 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 (Figure 2.5.1-213).

The Duke Energy William States III Lee FSAR (Reference 2.5.1-365) 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.

East Coast Fault System — Central (ECFS-C) Segment

Marple and Talwani (Reference 2.5.1-243) extended the zone of river anomalies that characterize the ECFS northward into North Carolina, citing studies by Markewich (Reference 2.5.1-265) and Soller (Reference 2.5.1-266) that interpreted an approximately 95-km- (60-mi.-) long north-northeast-trending buried fault or zone of flexure based on evidence from fluvial geomorphology, soil profiles, and surface fault exposures.

The features described by Marple and Talwani (Reference 2.5.1-243) to support the continuation of a regional structure or presence of buried faults beneath the North Carolina Coastal Plain include uplift and cross-valley tilt of river terraces along the Cape Fear River, increase in channel sinuosity downstream on several rivers, incision through Pleistocene floodplain deposits, alignment of river anomalies, coincidence with buried faults, alignment with magnetic anomalies, and evidence of brittle faulting. The observations and data sets, which can be grouped into geological, seismological, and geomorphological categories, are reviewed in light of published information, seismological data, and independent analysis of topographic and stream profiles generated from LIDAR data completed for the HAR COL Application, as follows.

<u>Geological and Geophysical Data</u>

Marple and Talwani report coincidences with buried faults, magnetic anomalies, and surface faults in Cretaceous and post-Cretaceous sediments as geological evidence for the existence and activity of the postulated ECFS-C (Reference 2.5.1-243). However, geological evidence to support the presence of a throughgoing bedrock fault associated with the ECFS-C segment is not presented by Marple and Talwani (Reference 2.5.1-243). The primary evidence cited for faulting associated with the postulated ECFS-C is distributed brittle faulting north of Smithfield in an area above the fall zone where bedrock is locally exposed (Figures 2.5.1-213 and 2.5.1-251). In addition to surface faults exposed in road cuts that offset post-Cretaceous sediments included in a Prowell Cretaceous and post-Cretaceous fault compilation (Reference 2.5.1-256), two

brecciated phyllite and argillite zones (sites B1 and B2 on Figure 2.5.1-252) are reported in the vicinity of the postulated ECFS-C. Both breccias have varying strike and dip that deviate from the regional structural trends. None of the brecciation sites or fault exposures is associated with topographic scarps (Reference 2.5.1-243).

Table 2.5.1-201 shows summary information (fault trace identification number, fault type, strike and dip orientations, and basement rock and sedimentary rocks affected) for the faults included in the Prowell compilation (Reference 2.5.1-256). Thirty-three faults were identified between latitudes 38°03' and 34°05'. These faults are not localized along the ECFS, but rather are distributed across an approximately 50-km- (35-mi.-) wide zone at the boundary between the upper Coastal Plain and the Piedmont physiographic provinces (Figures 2.5.1-213 and 2.5.1-219). Of the 33 faults reported in this region, Marple and Talwani report offsets of Pliocene-Pleistocene sediments of less than a few meters for six of the faults (faults 46, 48, 52, 53, 55, and 56) that are proximal to the postulated ECFS-C (Figure 2.5.1-219; Reference 2.5.1-243). Most of the faults are interpreted by Prowell as near-vertical reverse faults (Reference 2.5.1-256). With the exception of fault 46, which is reported to displace the Coharie Formation of Pliocene-Pleistocene age approximately 2.8 m (9 ft.) and fault 48, which disrupts Upper Cretaceous to questionable Pliocene sediments 1.5 m (5 ft.), the other four faults cited by Marple and Talwani are reported only to offset sediments of Upper Cretaceous to questionable Pliocene age by an unknown amount (Table 2.5.1-201). Given the distributed nature and small displacements exhibited by these faults, the presence of brittle faulting in association with the postulated ECFS-C is not strong evidence for a major active strike fault capable of generating large-magnitude earthquakes.

Marple and Talwani also report that the northern end of the postulated ECFS-C (near Bailey, North Carolina) coincides with a 7-km- (4-mi.-) long west-side-up fault or flexure in the Piedmont basement rocks beneath 6 to 12 m (20 to 40 ft.) of sediments and with a 40-km- (25-mi.-) long linear magnetic high between Smithfield and the Tar River. The relief across the inferred fault or flexure is interpreted to be approximately 7 m (23 ft.). (Reference 2.5.1-243) Marple and Talwani do not provide details regarding the evidence for the fault or flexure zone. Hoffman and Carpenter, cited as the original source of the evidence for a fault or flexure in basement, describe evidence for marine planation (terracing) of the crystalline basement surface during an early Pliocene transgression and the development of subsequent recessional shoreline deposits and erosional escarpments in this area (Reference 2.5.1-268). Based on a structure contour map on the top of crystalline basement defined by numerous auger holes and surface outcrops, there are at least two planation surfaces with an intervening curvilinear escarpment (Figure 2.5.1-253). Based on examination of these structure contour maps and on cross sections presented by Hoffman and Carpenter, there does not appear to be a consistent scarp coincident with the ECFS-C. The curvilinear nature of the basement scarp, which appears to be directly related to the location of outcropping granite, and the general elevation of the base of the scarp at approximately 60 m (200 ft.) around the granite outcrop

support the interpretation that this is a shoreline-related scarp. The scarp coincides only very locally with the northern end of the ECFS-C (Figure 2.5.1-253).

The magnetic anomaly described by Marple and Talwani (Reference 2.5.1-243) as coincident with the northern end of the ECFS-C corresponds to the eastern margin of the Rolesville batholith and is not continuous along most of the ECFS-C (Figure 2.5.1-238).

The postulated ECFS-C fault is not expressed as a throughgoing feature in the gravity or magnetic data, as shown on Figures 2.5.1-237 and 2.5.1-238, suggesting 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.

The postulated ECFS-C segment does not coincide with any of the basement structures identified by Lawrence and Hoffman (Figure 2.5.1-213). In addition, the postulated fault crosscuts geologic units, structure contours on the top of basement, and locally diabase dikes as mapped by Lawrence and Hoffman (Reference 2.5.2-235) with no apparent offset (Figure 2.5.1-254).

In summary, the above observations support the conclusion that the postulated ECFS-C fault does not appear to have deep crustal expression or continuity on a regional extent.

Seismological and Paleoseismological Data

The ECFS-C is not associated with alignments or concentrations of seismicity or moderate-size historical earthquakes (Figure 2.5.1-255). There is no direct geologic evidence to suggest that large-magnitude earthquakes have occurred along the postulated ECFS-C segment. Except for isolated paleoliquefaction sites observed in beach and near-shore marine deposits at the South Carolina/North Carolina border approximately 100 km (60 mi.) east of the ECFS-C segment, no evidence for paleoliquefaction was observed during regional reconnaissance investigations conducted by Amick et al. (Reference 2.5.1-270) along much of the North Carolina coastline region. Although detailed focused paleoliquefaction surveys have not been conducted in much of Coastal Plain region adjacent to the ECFS-C segment, no paleoliquefaction features have been reported by Owens, Soller, or Markewich, who conducted detailed mapping of Quaternary terraces along the Cape Fear River (Reference 2.5.1-271, Reference 2.5.1-266, Reference 2.5.1-265).

Geomorphic Data

Geomorphic evidence presented by Marple and Talwani for the ECFS-C is summarized in Table 2.5.1-202 and includes channel incision, upward displaced fluvial surfaces, cross-valley tilt, changes in sinuosity, anastomosing stream

patterns, and river deflection. Most of the features described below can be explained by nontectonic geomorphic processes or more regional patterns of flexure or differential uplift/subsidence rather than localized fault-specific tectonic deformation. An evaluation of the geomorphic and geologic evidence for the postulated ECFS-C presented by Marple and Talwani (Reference 2.5.1-243) and alternative interpretations is presented below.

River Incision and Upwarped Displaced Fluvial Surfaces

Channel incision and changes in graded profile concavity and reach-scale slope is dictated by changes in rock uplift, downstream changes in base level, and upstream changes in basin-scale hydrology (Reference 2.5.1-267). The fluvial systems in the Piedmont and the Coastal Plain regions of North Carolina likely have evolved in response to all of the above. Changes in sediment supply and overall base level related to fluctuating climates and sea level changes during the formation of fluvial terraces, therefore, are factors in addition to tectonically induced uplift that may give rise to apparent stream channel anomalies.

As tablulated by Marple and Talwani, channel incision is not reported for all of the major streams along the ECFS-S, nor is there a systematic relationship among the observations of channel incision and reported displacements in associated fluvial surfaces along these drainages denoted as “upward displaced fluvial surfaces” (Table 2.5.1-202). Channel incision is noted only for three of the six rivers studied by Marple and Talwani (Reference 2.5.1-243). The amounts of reported incision range from 3 m (10 ft.) of Holocene to post-130 – 70 ka incision along the Lumber River, to 18 m (59 ft.) of post-130 – 70 ka incision along the Cape Fear River, to 35 to 40 m (115 to 131 ft.) of incision of a surface of unknown age along the Neuse. Details regarding the incision along the Lumber and Neuse rivers are not provided by Marple and Talwani (Reference 2.5.1-243). In the vicinity of the Neuse River, the ECFS-C generally coincides with the Coats scarp, which may be a relic littoral feature correlative with the Orangeburg scarp of early Pliocene age. As noted by Soller and Mills (Reference 2.5.1-209), rivers are entrenched where they exit the higher upland northwest of the Orangeburg scarp. The incision noted by Marple and Talwani along the Neuse River, therefore, may represent long-term incision related to regional uplift and Pleistocene sea level fluctuations rather than localized tectonic uplift.

A map denoting the reach of anomalous river incision and representative topographic profiles across the Cape Fear River and its youngest terrace (Wando, early Pleistocene) shown by Marple and Talwani (Figure 8 in Reference 2.5.1-243) indicates that incision occurs well upstream and downstream of the ZRA-C. The pattern of incision occurs over a reach of the river extending approximately 50 km (30 mi.) upstream and at least 7 km (4.3 mi.) downstream of the ZRA-C as mapped by Marple and Talwani (Reference 2.5.1-243). Incision may extend as far as 35 km (22 mi.) downstream based on profile 7 of Marple and Talwani (Reference 2.5.1-243), which shows minor incision into the Wando terrace at a location downstream of Elizabethtown. This pattern is more consistent with simple tilting of the Coastal Plain along the valley

length (up from the direction of the Piedmont that caused deep entrenchment of the Cape Fear River into the Wando terrace in the upper valley concurrent with subsidence in the lower valley), as proposed by Soller (Reference 2.5.1-266), than is localized deformation along a strike-slip or oblique slip fault centered on the postulated ECFS-C. Soller (Reference 2.5.1-266) further suggests that the terrace pattern in the upper Cape Fear River valley, which may indicate a more localized zone of higher uplift, is related to a small-scale flexure that is parallel to and superimposed on the southern flank of the Cape Fear arch. Most of the Cape Fear River valley lies over the local bulge, which is inferred from a bulge in the basement structure contours (Figure 2.5.1-218). This localized uplift lies in the correct position relative to the Cape Fear River valley to account for the uplift history of the valley, and is therefore considered by Soller to be the source of the uplift that shaped the valley.

Marple and Talwani imply that channel incision is associated with localized tectonic uplift along the ECFS-C, but there is no consistent relationship between the amount of channel incision and the amount of upward displaced fluvial surface deformation recorded along individual rivers. The only locality in which the amount of reported Holocene incision (3 m [10 ft.]) is roughly correlative with the upward displaced fluvial surface measurement (2 m [7 ft.]) is along the Lumber River. However, inspection of the Pembroke, NW Lumberton, SW Lumberton, SE Lumberton, and Evergreen 7.5-minute quadrangles shows that distinct stream incision is not apparent as suggested by Marple and Talwani (Figure 2.5.1-217). There are no examples of confirmatory measurements of multiple surfaces to support the hypothesis that incision is occurring in response to ongoing tectonic uplift associated with the ECFS-C. Similar evidence for differential uplift of older fluvial surfaces of the same magnitude or greater is not evidenced by systematic warping or deformation of associated floodplains or fluvial terraces.

Examples of convex-upward longitudinal valley profiles along the South, Lumber, and Little Pee Dee rivers are cited as another possible indicator of vertical uplift (Figure 10 in Reference 2.5.1-243). It appears that most of the upward displaced fluvial surfaces identified by Marple and Talwani (Reference 2.5.1-243) are in fact such convexities rather than discrete displacements. Although convexities in longitudinal profiles may be produced by localized uplift of a channel and adjacent floodplain, other nontectonic processes can perturb a stream from an equilibrium condition and produce a convexity in its longitudinal profile (Reference 2.5.1-366). For example, apparent convexities in a stream profile may occur at the confluence of two streams, where increased discharge and sediment load downstream of the confluence commonly lead to a steeper gradient. This may be the cause of the convexities in the profiles of the floodplains along the Lumber and Little Pee Dee rivers. Major tributaries to the Lumber and the Little Pee Dee rivers intersect these drainages in the vicinity of the ZRA-C (ECFS-C) (Figure 2.5.1-256). The cause of the apparent convexity in the floodplain of the South River that Marple and Talwani cite as evidence for a 6.5- to 8-m (21- to 26-ft.) displacement is not clearly associated with a major tributary confluence; however, a small tributary joins the South River a few

kilometers upstream of the ECFS-C, and may influence sedimentation in the South River (Figure 2.5.1-219). The width of this anomaly (approximately 80 km [50 mi.]) extends well beyond the 20-km (6-mi.) width of the ZRA-C as designated by Marple and Talwani (Reference 2.5.1-243). The inferred tectonic displacement is assumed to have occurred subsequent to deposition of the fluvial terrace, which is estimated to be either Holocene or late Pleistocene (130 to 70 ka). However, there is no associated reported channel incision or inflections in the present channel (Profile P-P', Figure 2.5.1-220) to suggest a discrete uplift event associated with oblique slip fault movement along the ECFS-C. An influx of sediment to the South River from increased erosion and reworking of the thick older Pliocene and younger eolian sand deposits in the upper reaches of the South River triggered by changing Pleistocene climates is another possible mechanism that could result in stream disequilibrium and development of a convex stream profile.

Marple and Talwani infer uplift rates along the entire postulated ECFS from these inferred upwarped floodplains and terraces (Reference 2.5.1-243). These generally range from 0.02 to 0.3 millimeter per year (mm/yr) (0.0008 to 0.01 inch per year [in/yr]); faster uplift rates of 0.14 to 1.8 mm/yr (0.006 to 0.07 in/yr) and 0.05 to 0.65 mm/yr (0.002 to 0.026 in/yr) are inferred for the central segment based on the amounts of incision along the Cape Fear River (18 m [59 ft.] in 10 to 130 ka) and inferred warping of a terrace surface along the South River (6.5 to 8 m [21 to 26 ft.] in 10 to 130 ka), respectively.

The overall uplift rates that Marple and Talwani suggest for the ECFS-C, particularly the high rates based on assumed Holocene ages of incision, are not consistent with the general elevation of the Piedmont and Coastal Plain and long-term rates of uplift inferred from older Pliocene and early Quaternary terrace surfaces. Assuming that the uplift rates suggested for the region between the Cape Fear River and South River are correct, older surfaces such as the Bear Bluff (approximately 2.75 Ma) and Waccamaw (1.25 Ma to 1.65 – 1.75 Ma) (Reference 2.5.1-266, Reference 2.5.1-367) should exhibit significant vertical displacement across the ECFS-C. Along the Cape Fear River, the inferred rates of 0.14 to 1.8 mm/yr over the past 1.25 Ma to 730 ka, which Marple and Talwani postulate to be the age of the inception of the faulting along the ECFS-C, would suggest localized vertical offset of 102 to 1314 m (335 to 4310 ft.) across these surfaces. This is not observed in the topography of the interfluve region or in the general elevation of the projected Bear Bluff and Waccamaw surfaces above the modern channel upstream of the postulated ECFS-C. Topographic profiles on the Waccamaw terrace where it transects the ECFS-C suggest that there may be less than approximately 10 m (33 ft.) of cumulative vertical displacement across the postulated ECFS-C.

The basis for concluding that the incision and warping all occurred during Holocene time in response to localized tectonic uplift is not provided by Marple and Talwani, and does not seem warranted, based on a review of the data. Stream profiles developed for the HAR FSAR based on topographic data from a LIDAR survey show no consistent vertical anomalies in the modern drainages

where the postulated ECFS-C crosses the profiles (see Figures 2.5.1-219 and 2.5.1-220 and discussion in Subsection 2.5.1.1.4.2.5.2.3). A small apparent vertical down-to-the-southeast step may be present along the Cape Fear River channel, but is not observed in the Lumber River profile to the south. Aerial reconnaissance conducted for this study also did not reveal any obvious topographic expression (e.g., escarpments or lineaments) of the postulated ECFS across interfluve areas.

Marple and Talwani state that the uplift rate of ~0.64 mm/yr (~0.025 in/yr) along the Wando terrace (120 to 70 ka) measured downstream of the postulated ECFS-C is probably low because the location is downstream from the area of greatest incision (Reference 2.5.1-243). As noted above, this statement suggests that uplift is not localized at the ECFS-C as suggested by Marple and Talwani. Soller suggests that uplift along the Cape Fear River increases gradually from 32 km (20 mi.) downstream to 32 km (20 mi.) upstream from Elizabethtown (Reference 2.5.1-266).

The high vertical rates based on the assumed maximum rates of incision assigned to the postulated ECFS-C are not warranted. Assuming that the maximum incision observed upstream from this area (18 m [59 ft.]) is due to uplift on the postulated ECFS-C as defined by Marple and Talwani, a better estimate of the long-term slip rate across this structure if it exists would be to subtract the estimated uplift rate downstream (~0.06 mm/yr [~0.002 in/yr]) from the post-Wando incision rate (18 m [59 ft.] per 70 to 130 ka = 0.26 to 0.14 mm/yr [0.01 to 0.006 in/yr]) measured where the postulated ECFS-C is assumed to cross the Cape Fear River. This would suggest a maximum slip rate of 0.08 to 0.2 mm/yr (0.003 to 0.008 in/yr) for the postulated ECFS-C. Assuming that the inferred upward displaced fluvial surface measurements are tectonic, lower slip rates are suggested by the 6.5 m (21.3 ft.) and 2 m (6.6 ft.) of warping of the 70 to 130 ka terrace across the South River (of 0.05 to 0.065 mm/yr [0.002 to 0.0026 in/yr]) and Little Pee Dee (i.e., 0.02 mm/yr [0.0008 in/yr]), respectively (Reference 2.5.1-243). Based on interpretation of topographic profiles constructed on the Waccamaw terrace as mapped by Owens (Reference 2.5.1-271) and Soller (Reference 2.5.1-266) and possible upstream terrace correlations, there may be no or less than 10 m (33 ft.) of possible vertical offset of this surface across the ECFS-C.

In conclusion, the evidence of incision and localized uplift along the ECFS-C are not consistent along strike, are not evidenced in the topography of the interfluve regions, and in the case of the upward displaced fluvial surfaces may be related to nontectonic fluvial responses to factors other than localized tectonic uplift on the postulated ECFS-C. The high rates of uplift, particularily those based on assumed Holocene ages, are not supported by other geologic data and therefore are judged not to be viable.

Cross-Valley Change

Marple and Talwani (Reference 2.5.1-243) cite cross-valley changes in the morphology of the Lumber and Cape Fear river valleys across the ZRA-C as evidence for Quaternary tectonic tilting and folding. Specifically, Marple and Talwani suggest that the large, isolated, entrenched meander of the Cape Fear River within the Wando terrace was deflected 6 km (3.7 mi.) northeastward toward the Cape Fear arch axis in response to a gentle cross-valley tilt in contrast to the southward migration of the channel downstream of the postulated ZRA-C. The scroll pattern and interpreted northeastward tilt of terrain inside the meander are given as evidence that the meander formed by northeastward river migration along the ZRA-C during late Pleistocene (130 to 70 ka) before becoming deeply entrenched. Alternative nontectonic explanations of the development of the meander loop and slip-off terrace along the Cape Fear River are not considered by Marple and Talwani. There is no consideration, for example, of the fact that a large tributary intersects the Cape Fear River just upstream of the meander bend and that sediment influx from this tributary may have been a factor in the northward migration of the channel during deposition of the Wando terrace.

Marple and Talwani also cite an abrupt change from a relatively symmetric cross-valley shape upstream from the ZRA to a down-to-the-southwest cross-valley tilt as evidence for localized uplift and tilting along the ZRA-C (ECFS-C) (Reference 2.5.1-243). Soller (Reference 2.5.1-266), however, presents an alternative model in which the Cape Fear River terraces have been uplifted and preserved in response to (1) a persistently low rate of uplift from the north to northeast, transverse to the valley length that is largely responsible for the succession of unpaired terraces in the central part of the valley, and (2) uplift from the direction of the Piedmont (parallel to the valley length) that has been intermittently active and most recently tilted up to the northwest and down to the southeast, causing deep entrenchment of the Cape Fear arch into the Wando terrace and burial of terraces beneath the floodplain in the lower valley.

River Deflection, Change in Sinuosity, and Anastomosing Stream Patterns

The interpretation of the inferred river anomalies as neotectonic features is questionable. Weems (Reference 2.5.1-273) does not identify anomalies in the river profiles that would coincide with the central segment of the zone of river anomalies that Marple and Talwani cite as evidence for the existence and activity of the postulated ECFS-C (Figure 2.5.1-213; Reference 2.5.1-243). A recent unpublished compilation map by Dr. Weems showing possible neotectonic features in the Coastal Plain includes only the postulated ECFS-S as a possible neotectonic feature.

Many of the streams in the vicinity of the ECFS-C display river deflection, changes in sinuosity, and confluences of several streams coincident with the ECFS-C. Many of these changes in stream geomorphology are coincident with

the transition from the Piedmont to the Coastal Plain provinces, and may be due to nontectonic fluvial responses to steps in the underlying elevation of basement associated with terrace shoreline features or changes in lithology that may have influenced the location of such shorelines. Variations of channel characteristics can be attributed either to downstream variations in discharge, sediment load, and type of sediment moved through the channel or to local geology (Reference 2.5.1-366). Marple and Talwani cite the abrupt change from an entrenched, V-shaped cross-valley profile to a broad flood plain along the Neuse River as evidence for tectonic uplift across the ZRA-C. The Neuse River and a number of tributaries coalesce at the postulated ECFS-C. The Coats scarp, which wraps around the southern margin of the Rolesville batholith, also is roughly coincident with the ECFS-C where it intersects the Neuse River, Buffalo Creek, and Little River (Figure 2.5.1-219). The confluence of several streams near the ECFS-C along the Neuse River is coincident with a change from Paleozoic metamorphic rocks to Cretaceous Coastal Plain deposits and may reflect a lithology change rather than tectonism. No changes in stream gradient or vertical steps in the present stream channel are seen along this part of the Neuse River.

Marple and Talwani (Reference 2.5.1-243) report changes in sinuosity that they associate with the ECFS-C only along the Lumber and Cape Fear rivers. As noted above, the intersection of tributaries just upstream of the ZRA-C as mapped along both of these rivers may be responsible for variations in discharge and sediment influx that could influence the downstream channel morphology. A change in sinuosity on the South River near the ECFS also is coincident with a tributary joining the river from the northwest, and may be due to fluvial response to changes in sediment load and stream flow.

Changes in the stream pattern from anastomosing to meandering are cited as evidence of differential uplift along the ZRA-S, but are not noted along the ZRA-C.

Marple and Talwani conclude that the abrupt southwestward deflection of the Neuse River just upstream of the ZRA-C toward the Cape Fear arch axis was not produced by channel migration, but instead may have been offset by right-lateral strike-slip faulting. Although it cannot be precluded that the channel is coincident with a Paleozoic fold or minor fault, a diabase dike interpreted from magnetic anomalies and Paleozoic boundaries marking differing grades of metamorphism at the margin of the Roseville batholith extends across the river deflection without significant right-lateral offset (Figure 2.5.1-254).

<u>Lithospheric Flexure Models</u>

Although the geomorphic features cited in support of the ECFS-C do not clearly demonstrate the presence of an active regional-scale crustal fault that is localizing seismicity in the present tectonic regime, the evidence of uplift and tilting may be consistent with a tectonic mechanism, that of lithospheric flexure (i.e., long-wavelength bending or warping of the lithosphere). As suggested by

Soller (Reference 2.5.1-266), the distribution of terraces along both the Cape Fear and Pee Dee rivers may be the cumulative effect of several variables. For example, based on mapping and longitudinal profiles on terraces estimated to range in age from 2.75 to 0.1 Ma, Soller interprets that the locus and intensity of local tectonism in the vicinity of the Cape Fear River has varied with time (Reference 2.5.1-266). Soller concludes that since at least 750 ka, the valley northwest of Elizabethtown near the Piedmont has been uplifted while subsidence is likely for the river's lower course (Reference 2.5.1-266). An inflection point southeast of Elizabethtown may mark the boundary between these two regimes and a simple tilting of the Coastal Plain along the valley length, up from the direction of the Piedmont, could account for changes in uplift rate measured along the river on terraces of differing ages. A small-scale flexure, related to larger-scale tectonism on the Cape Fear arch, can explain the pattern of uplift (Figure 2.5.1-218). Additionally, an apparent bulge in basement structure contours that is subparallel to and superimposed on the broad outline of the Cape Fear arch may represent a localized flexure that accounts for the uplift patterns (Figure 2.5.1-218). (Reference 2.5.1-266) This feature does not coincide with or subparallel the postulated ECFS-C as interpreted by Marple and Talwani (Reference 2.5.1-243).

As described by Pazzaglia, geodynamic models that simulate flexural deformation along the U.S. Atlantic margin in response to erosion in the Appalachian Mountains and sediment loading in the offshore regions, could also explain in part the possible broad zone of flexure or localized uplift described by Markewich, which Marple and Talwani link to the ECFS-C (Reference 2.5.1-267, Reference 2.5.1-265). Pazzaglia describes a similar broad, flexural warp centered across the fall zone, between the upwarped central Appalachians and subsided Salisbury embayment in the region to the north of the Norfolk arch (Reference 2.5.1-267). Smaller tectonic features, such as the small-displacement Cretaceous and Cenozoic faults and possible seismicity, may coincide with this zone of flexture, which coincides with a more prominent, well-defined fall zone in the vicinity of the Susquehanna River (Reference 2.5.1-267). However, the complex interaction between the regional and flexural stress field and their relationship to pre-existing faults is poorly constrained (Reference 2.5.1-267).

Marple and Talwani (Reference 2.5.1-243) discount the possibility that isostatic uplift of the Appalachians to the west and sediment loading offshore is a viable cause because the ZRA-C's trend is oblique to the Appalachian trend, sediment loading offshore and in the outer Coastal Plain of the Carolinas is too widely distributed, and the hinge zone is approximately 200 to 250 km east of the area. The first of these objections presupposes that the observations that define the ZRA-C as a unique and well-constrained feature as mapped are valid. As noted in the previous discussions, this is not the case. Unloading of the crust from erosion in the Appalachians and deposition of the sediment in depocenters adjacent to the Cape Fear arch likely does result in broad regional flexure that influences major drainages such as the Cape Fear River. Marple and Talwani acknowledge that migration of the river to the southwest is still occurring in the lower reaches of the Cape Fear River in response to such regional flexure; they

do not, however, acknowledge a more complex pattern that would factor in higher uplift upstream toward the Piedmont, as suggested by Soller (Reference 2.5.1-266).

East Coast Fault System — Northern (ECFS-N) Segment

The following discussion of evidence cited in support of the ECFS-N segment is summarized from the Dominion North Anna FSAR and supporting documentation (Reference 2.5.1-368, Reference 2.5.1-369).

Geological Data

It was noted in the Dominion North Anna assessment that most of the data used by Marple and Talwani (Reference 2.5.1-243) to support their interpretation of the ECFS apply exclusively to the southern and central segment of the fault (ZRA-S and ZRA-C). The actual number and quality of the data used to infer the presence of the northern segment of the fault system (ZRA-N) is significantly less than that for the ZRA-S and ZRA-C segments. The only geologic evidence cited by Marple and Talwani in support of the ECFS-N is its coincidence with the westward termination of the axis of the Norfolk arch axis, which was originally depicted by Pazzaglia (Reference 2.5.1-267). The westward termination of the Norfolk arch axis, however, was modified by Marple and Talwani to end approximately 25 km (15 mi.) east of the ECFS-N, with no additional references, interpretations, or data to justify the modification of the location. Dominion concludes that "the location of the Norfolk arch axis, as presented by Marple and Talwani [Reference 2.5.1-243], does not provide independent geologic evidence in support of the ECFS-N, and therefore there is no known geologic evidence to support the existence of ECFS-N" (Reference 2.5.1-272, Reference 2.5.1-360).

Geophysical and Seismological Data

Geophysical data presented by Marple and Talwani (Reference 2.5.1-243) is an east-west-trending seismic reflection profile along Interstate 64 in central Virginia that was originally presented by Pratt et al. (Reference 2.5.1-323). Pratt et al. interpret an east-dipping shear zone approximately 30 km (19 mi.) beneath the inferred location of the ECFS-N, but do not interpret a steeply dipping crustal shear zone in the vicinity of the ECFS-N. The seismic data, therefore, do not support the interpretation by Marple and Talwani. The Dominion application further states that the I-64 seismic reflection profile is the only geophysical or seismological data presented by Marple and Talwani (Reference 2.5.1-243) and that Marple and Talwani (Reference 2.5.1-243) do not associate any seismicity with the ZRA-N.

Geomorphic Data

The geomorphic data used by Marple and Talwani to postulate the ECFS-N are inferred river anomalies, including channel incision, upward displaced fluvial surfaces, cross-valley change, sinuosity change, anastomosing stream pattern,

and stream deflections. Dominion examined each of the river anomaly categories with reference to the ECFS-N to weigh evidence for its existence and concluded that there was no evidence of a fault and that there is direct stratigraphic evidence against the types of deformation postulated by Marple and Talwani (Reference 2.5.1-243).

The key observations and conclusions from the Dominion assessment are summarized as follows:

- No consistent co-occurrence of two or more anomalies along each of the drainages was observed, as may be expected if they have developed in response to uplift of the ZRA-N.
- There is no consistent pattern of anomalies along the trend of the ZRA-N, as would be expected if the structure was active along its entire length.
- It was not possible to verify or duplicate geomorphic observations such as channel incision.
- The “upward displaced fluvial surfaces” are inferred only from qualitative analysis of convexities of river profiles, and therefore, this type of “anomaly” does not provide evidence for tectonic uplift and is inconsistent with other geomorphic observations. These features in most cases are more objectively characterized as convexities, or local increases, in the gradient of the longitudinal profiles of floodplains due to the intersection of concave profiles at river confluences.
- Direct stratigraphic evidence for no Quaternary deformation was documented in the vicinity of a large meander of the Nottoway River that Marple and Talwani (Reference 2.5.1-243) interpreted to have formed in response to systematic folding and northeastward tilting.
- The fluvial geomorphic features cited by Marple and Talwani (Reference 2.5.1-243) are likely produced by nontectonic fluvial processes, are not anomalous, and thus do not support their interpretation of the presence and activity of the ZRA-N (ECFS-N).

SUMMARY OF ASSESSMENT OF THE EAST COAST FAULT SYSTEM AS A CAPABLE TECTONIC SOURCE

There is supporting geological, geophysical, and seismological information to suggest that geomorphic anomalies identified along the southern half of the ECFS-S segment may be associated with Quaternary displacement on the Woodstock fault and that this fault may be the source of the Charleston earthquake (Reference 2.5.1-370).

There is no similar evidence to suggest that the northern part of the ECFS-S, ECFS-C or ECFS-N segments are capable tectonic structures as defined by Regulatory Guide 1.208.

The observations and lines of evidence presented by Marple and Talwani (Reference 2.5.1-243) do not provide convincing arguments in support of a buried north-northeast-trending strike-slip fault (the postulated ECFS-C) through the North Carolina Coastal Plain region. Evidence of neotectonic deformation (i.e., differential uplift of the Piedmont relative to the Coastal Plain regions, regional tilting, and broad zones of tilting or flexure) can be explained by lithospheric flexure (i.e., long-wavelength bending or warping of the lithosphere) related to regional patterns of erosion and Cenozoic deposition. Localized Cenozoic faulting observed near the Piedmont – Coastal Plain boundary may be related to stresses in the region of greatest flexure (Reference 2.5.1-267). The possibility that some local structures along the general trend of the East Coast fault system may be present and may be favorably oriented for reactivation in the present tectonic setting cannot be precluded given the available data. There is no geological data, however, to demonstrate Quaternary surface faulting. There is no associated seismicity or reported evidence of paleoliquefaction to indicate activity along the ECFS-C segment. The implication that the postulated central and northern segments of the ECFS, if they exist, may produce earthquakes of a similar size to the 1886 Charleston earthquake as inferred by Marple and Talwani (Reference 2.5.1-243) is not demonstrated. Therefore, the ECFS-C and ECFS-N segments are included in the updated PSHA for the HAR site with low probability of existence (weight of 0.1) and activity given the fault exists (weight 0.1).

2.5.1.1.4.2.5.2.3 Fall Lines of Weems

Weems (Reference 2.5.1-273) examined longitudinal profiles of large drainages that generally flow northwest to southeast across the Piedmont and Blue Ridge provinces in North Carolina, Virginia, and southeastern Tennessee. Weems identified and named seven fall lines, based on correlation of fall zones (short stream segments that have anomalously steep gradients and typically contain rapids or waterfalls) along adjacent streams and rivers. The fall lines generally trend northeastward, parallel to the regional structural grain, and merge toward the northeast. Apparent warping of the late Pliocene fluvial terraces associated with the fall zones was cited by Weems to suggest probable formation within the past 2 m.y. Although evidence is minimal and other hypotheses for the origin of these features (i.e., variable erosion of rocks of varying hardness or response to late Cenozoic climatic and sea level fluctuations) were considered, Weems proposes that the available evidence favors neotectonic control of the fall lines, perhaps by intermittent faulting (Figure 2.5.1-216).

Based on more detailed evaluation of the fall lines that was conducted as part of the North Anna Early Site Permit Application, Dominion presented observations and analyses to support the conclusion that the seven fall lines defined by Weems do not represent a capable tectonic source (Reference 2.5.1-274, Reference 2.5.1-371). The observations and conclusions include the following:

- The features are not defined by formal, consistently applied criteria and thus are not as well defined and laterally continuous as depicted.
- In some cases, various features were electively correlated to form a laterally continuous fall line, while in other cases similar features were not correlated.
- Differential erosion due to variable bedrock hardness is a more viable and plausible explanation than Quaternary tectonism.
- There is no complementary geomorphic expression of tectonism, such as tectonic escarpments, along the trend of the fall lines between drainages, where it would be expected that such features would be better preserved.

Based on these arguments and detailed geomorphic analyses of the two fall lines within the North Anna ESP site vicinity (the Tidewater and Central Piedmont fall lines), NRC staff concurred with Dominion's interpretation that differential erosion is a more plausible explanation, as detailed in NUREG-1835. Additionally, Wheeler states that the identification of the fall zones is subjective, and the criteria for recognizing them are not stated clearly enough to make the results reproducible (Reference 2.5.1-259). Accordingly, it was concluded that tectonic faulting is not demonstrated, and fall lines are assigned to Class C (Reference 2.5.1-259).

Fall Lines in the HAR Site Vicinity (40-km [25-mi.] Radius)

Additional analysis of the fall lines within 80 km (50 mi.) of the HAR site was completed for this study. Longitudinal stream profiles derived from LIDAR data were compared with underlying geologic units as shown on the 1985 Geologic Map of North Carolina (Reference 2.5.1-208), with faults and folds mapped by the NCGS (Reference 2.5.1-275), and with zones of steeper gradients identified by Weems (Reference 2.5.1-273; Figures 2.5.1-219 and 2.5.1-220). Sixteen stream profiles (Lower Little River [A-A'], Crane Creek [B-B'], Upper Little River [C-C'], Rocky River [D-D'], Haw and Cape Fear rivers [E-E'], Mill Creek [F-F'], Neuse River [G-G'], Swift Creek and Neuse River [H-H'], Buffalo Creek [I-I'], Little River [J-J'], Moccasin/Contentrea Creek [K-K'], Toisnot Swamp [L-L'], Tar River [M-M'], Cape Fear [N-N'], Lumber [Q-Q'], and South River [P-P']) were created using stream locations from the U.S. Geological Survey National Hydrography Dataset and LIDAR data to evaluate stream channel morphology across the fall lines and other faults present within the area (Reference 2.5.1-276, Reference 2.5.1-269).

The Durham and Nutbush fall lines (designated DFL and NFL, respectively) are located within the HAR site vicinity and were defined by Weems based on steps in topography and changes in gradient along stream profiles (Reference 2.5.1-273; Figures 2.5.1-219 and 2.5.1-257). Observations regarding these two proposed fall lines are summarized as follows.

- Although designated as a single line on the regional map, the width of the anomalies as shown on longitudinal profiles presented by Weems vary from less than 1.2 km (2 mi.) to 22 km (13.5 mi.) (Figure 2.5.1-257) The criteria used by Weems to define the location and width of fall zones does not appear to be consistently applied in the case of fall zones upstream and downstream of the Durham and Sanford basins. For example, in the case of the NFL anomaly along the Tar River, Weems shows the anomaly to include a double inflection with a lower gradient stretch across the intervening Durham basin. A similar situation, albeit a wider zone, along the Cape Fear River shows a much narrower NFL that does not encompass the Triassic basin or the adjacent inflection upstream of the basin.

- Northeast of the Cape Fear and Haw rivers, the DFL coincides with the western margin of the Triassic basin (Figure 2.5.1-219). Weems originally showed the DFL diverging from the basin boundary to the south where it cut across the basin and eastern boundary of the Sanford basin (Reference 2.5.1-273). On a more recent unpublished map, Weems (Reference 2.5.1-372) relocated the southern part of the DFL to follow the western border of the Sanford basin south of the Cape Fear River (Figure 2.5.1-219). In the new interpretation, a previously uncorrelated fall zone (labeled ifz on Profile 11, Figure 2.5.1-257) likely represents the DFL. This highlights the difficulty in correlating fall zones between different river systems, and suggests that correlations may not be unique.

- Weems (Reference 2.5.1-273) shows sharp changes in stream gradient along the DFL at the Tar, Neuse, and Cape Fear rivers. The anomaly mapped as the DFL along the Tar River on Figure 2.5.1-257 is a misinterpretation of the typically steep headwaters reach of a drainage as a fall zone. Similar steep upper reaches along Crane Creek (Profile B-B') and Rocky River (Profile D-D') on Figure 2.5.1-220 are roughly coincident with the DFL and CPFL, respectively, suggesting these features may not be anomalous features that would qualify as fall zones. As noted in the Dominion North Anna assessment (Reference 2.5.1-371), such steep upper-river reaches are not anomalous because the gradients of all streams typically steepen dramatically in the upstream third of their profiles, especially with proximity to the headwaters. The upstream increase in gradient is a logarithmic function and is characteristic of the typical concave longitudinal profile of a stream. The logarithmic increase in gradient with proximity to the headwaters is especially pronounced by the vertical exaggeration in Weems's profiles, contributing to the appearance of a fall zone. Weems (Reference 2.5.1-273) does not explain why these particular headwater reaches should be considered anomalous and thus characterized as fall zones. In addition, Weems (Reference 2.5.1-273) does not explain why steep headwater reaches of the majority of other rivers in the study area are not considered fall zones.

- The most pronounced step occurs along the Haw and Cape Fear rivers where the DFL crosses the western margin of the Durham basin

(Figure 2.5.1-220). Where the DFL crosses the Haw River, a major tributary of the Cape Fear River, it coincides with the dam at the southern end of Jordan Lake, making it difficult to evaluate the original stream morphology across the DFL (Profile E-E', Figure 2.5.1-220).This location generally correlates with a zone of Mesozoic basin-bounding faults (designated MF) that extend into the basin to the north as intrabasinal faults (Figure 2.5.1-219); these western basin margin faults juxtapose bedrock lithologies of differing strength and erosional resistance, going from harder igneous and metamorphic rocks upstream to softer Triassic sediments within the basin. The northernmost of these western basin margin faults is an approximately N70E-trending fault that extends into the basin and is referred to as an unnamed intrabasinal fault (See Subsection 2.5.1.2.4.1.1). Approximately 5 km (3 mi.) south of this unnamed fault, the DFL crosses two approximately N70E-trending Mesozoic faults that locally bound the western margin of the Deep River basin and juxtapose Paleozoic metamorphic rocks to the west against Triassic sediments to the east. The DFL fall zone anomaly as defined by Weems (Reference 2.5.1-273) is not a narrowly constrained feature that suggests reactivation of a single fault or fault zone. There is no reported evidence to suggest that faults along the western boundary of the Sanford basin have been reactivated or are capable tectonic sources.

- Detailed mapping of the Farrington 7.5-minute quadrangle (Reference 2.5.1-373) along the western margin of the Durham basin provides additional information to evaluate the DFL. The DFL in this region crosscuts the strong northeast-trending structural fabric, the northeast-trending Bush Creek fault, and numerous northeast-trending lithologic boundaries within the Proterozoic-to-early-Paleozoic Carolina lithotectonic belt. Differential uplift across the DFL is not expressed in the upland terrain adjacent to the river drainages along this margin. These observations do not support a tectonic interpretation of a continuous DFL as a neotectonic structure that is localizing uplift across this boundary.

- Weems (Reference 2.5.1-273) notes that the DFL in eastern North Carolina, like the CPFL in Virginia, marks the eastern boundary of uplifted areas that were eroded to produce numerous inselbergs. The flatter upper reaches of the Rocky River (from its headwaters to approximately 40 km downstream [Profile D-D']) and the Haw River (from upstream of the CPFL to approximately 40 km downstream [Profile E-E']) have the appearance of older graded profiles that have been uplifted. The steeper, more concave downstream sections may represent the adjustment of the profile to lower base level related to eustatic fluctuations or sediment loading and isostatic adjustments in the Coastal Plain embayments. These observations are consistent with a model of more regional rock uplift of the Appalachian Piedmont due to erosion, isostatic adjustments, and eustatic fluctuations and development of long-wavelength flexural deformation in the general vicinity of the fall zone between the Piedmont and Coastal Plain regions (Reference 2.5.1-267).

- The NFL, which is located approximately 11.3 km (7 mi.) east of the Jonesboro fault, roughly coincides with a Paleozoic mylonite shear zone (the Nutbush fault zone) along much of its mapped length (Figure 2.5.1-219). Weems (Reference 2.5.1-273) notes this strong correlation and states that this represents the most consistent geographic correlation between a fall line and a known tectonic structure. There are anomalies in the stream profiles that coincide with the Nutbush Creek fault zone at some locations (e.g., Profiles E-E' and G-G', Figure 2.5.1-220). However, as shown on the Tar River stream profile (Profile M-M', Figure 2.5.1-220), there is not an anomaly coincident with the mapped Nutbush Creek fault at this location. The toe of the anomaly upstream of the fault identified on the Weems stream profile (Profile 9, Figure 2.5.1-257) appears to correlate to a lithologic boundary. The stream profile across the buried Nutbush Creek fault zone also does not show an anomaly at or upstream of the NFL (Profile C-C', Figure 2.5.1-220).Thus there does not appear to be a consistent step coincident with the mapped fault trace.

- Where the NFL crosses Swift Creek, a major tributary to the Neuse River, it coincides with the Paleozoic Nutbush Creek fault zone, which juxtaposes felsic mica gneiss (CZfg) against injected gneiss (CZig). The reach of the drainage upstream of this intersection subparallels an east-west-trending Mesozoic fault. Further downstream, where the channel crosses another east-west-trending Mesozoic fault, the Swift Creek fault, there is a small step in the stream profile (Profile G-G', Figure 2.5.1-220). Steps in the stream profile across the Nutbush Creek fault zone and the Swift Creek fault coincide with lithologic boundaries and may be due to differences in the hardness of and erosional resistance of different units. No evidence of Quaternary reactivation of either the Nutbush Creek or Swift Creek faults have been reported or documented from recent geologic mapping investigations, and both faults are judged to be not capable tectonic sources (see Subsections 2.5.1.1.4.4.2 and 2.5.1.2.4.1.2).

- To the south where the NFL crosses the Fuquay-Varina quadrangle (Reference 2.5.1-374), the mapped NFL coincides with the Leesville fault zone (along the projected trend of the Nutbush Creek fault zone) in the northern part of the quadrangle, but diverges from it in the southern part of the quadrangle. There are no aligned scarps or topographic expression of faulting in Coastal Plain sediments in the interfluve areas along the NFL or the Leesville fault zone as shown by unpublished mapping (Reference 2.5.1-374).

- There is evidence of post-Cretaceous faulting proximal to the NFL at the Parker locality (Nos. 49 and 150, Figure 2.5.1-219) The relationship of this reverse fault, which is oriented N15°W to the northeast-trending Nutbush Creek fault is unknown. No geomorphic expression of the fault was observed during aerial reconnaissance (see Subsection 2.5.1.2.4.3). Additionally, there are no anomalies or expression of faulting in a stream profile along Middle Creek, just south of the Parker locality (Profile Q-Q', Figure 2.5.1-220).

In summary, analysis of the two fall lines (DFL and NFL) within the site vicinity suggests that where the fall lines do coincide with changes in stream gradient, the changes are more likely the result of fluvial stream adjustments related to differential erosion of bedrock varying in hardness. Additionally, although the fall lines locally correspond with changes in stream gradient, profiles derived using 6-m- (20-ft.-) resolution topographic data from LIDAR indicate that in some cases there are no marked changes where the fall lines cross the streams, and changes in stream gradient of similar magnitude to the fall lines also exist where no fall lines are mapped (Figure 2.5.1-220).

The overall conclusion from this analysis is that although anomalies in the longitudinal stream profiles at the fall lines coincide locally with Mesozoic or Paleozoic tectonic features, the fall lines are not as well defined or continuous as suggested by Weems (Reference 2.5.1-273) and there is no clear indication of localized neotectonic deformation or reactivation of structures along the length of the fall lines in the site vicinity. These conclusions are consistent with the observations and results of the Dominion North Anna ESP analysis (Reference 2.5.1-375, Reference 2.5.1-274).

2.5.1.1.4.2.5.3 Paleoliquefaction Features

2.5.1.1.4.2.5.3.1 Paleoliquefaction Features Within the Central Virginia Seismic Zone

Two sites of Holocene liquefaction have been reported within the CVSZ (Reference 2.5.1-258, Reference 2.5.1-376). These sites include an area of probable late Holocene (2000 to 3000 years old) liquefaction along the James River and a possible area of early to mid-Holocene (approximately 5000 years old) liquefaction along the Rivanna River (Reference 2.5.1-376). The Dominion North Anna ESP FSAR also indicates that there may be a possible third site of possibly early to mid-Holocene liquefaction along the South Anna River.

Based on the absence of widespread paleoliquefaction, Obermeier and McNulty (Reference 2.5.1-376) conclude that an earthquake of magnitude 7 or larger has not occurred within the CVSZ in the past 2000 to 3000 years, or in the eastern portion of the seismic zone in the past 5000 years. They note that the geologic record of one or more magnitude 6 or 7 earthquakes might be concealed between streams, but that such events could not have been abundant in the seismic zone. It is also possible that these isolated locations of paleoliquefaction were produced by local shallow moderate-magnitude earthquakes of **M** 5.5 to 6.5.

Based on evaluation of these data, it was stated in the Dominion North Anna ESP Application that the presence of these liquefaction features does not indicate a change in the smallest maximum-magnitude level assigned to the CVSZ in the 1986 EPRI study. In response to RAI 2.5.1-1 for the Dominion North Anna ESP, the applicant concluded that the liquefaction features identified by

Obermeier and McNulty (Reference 2.5.1-376) represent one or possibly two moderate-magnitude (magnitude approximately 5.5 to 6.5) earthquakes in the CVSZ in the middle to late Holocene. Field reconnaissance by Obermeier of thousands of meters of liquefiable deposits in the area found that liquefaction features occur in a restricted area and are not indicative of a magnitude 7 earthquake or abundant magnitude 6 to 7 earthquakes within the CVSZ during the Holocene. The NRC concurred that the characterization of the impact of paleoliquefaction features on the seismic characterization of the CVSZ was accurate and that characterization of the occurrence of earthquakes is consistent with EPRI seismic source recurrence estimates for the CVSZ. (Reference 2.5.1-360) Because the causative faults remain unidentified, the CVSZ is best characterized as a seismogenic source and not a capable tectonic source. (Reference 2.5.1-272)

2.5.1.1.4.2.5.3.2 Paleoliquefaction Features Within the Charleston Region

During the 1886 Charleston earthquake (intensity MMI X and **M** 7.3), eyewitnesses in central coastal South Carolina reported widespread liquefaction. The distribution and density of the liquefaction associated with the 1886 Charleston earthquake were documented by Dutton (Reference 2.5.1-296) and provide useful information on the epicentral location of the earthquake. In the same region are middle to late Holocene craters, sand blows, and sand fissures produced by large prehistoric earthquakes.

Searches for paleoliquefaction features in the 1886 Charleston epicentral area and in the southeastern U.S. coastal region were performed by a number of researchers to better define the location and geometry of the Charleston seismic source. Obermeier et al. (Reference 2.5.1-297, Reference 2.5.1-298, Reference 2.5.1-302) investigated the spatial distribution, size, and abundance of paleoliquefaction features in the Charleston region and beyond. Obermeier et al. (Reference 2.5.1-297, Reference 2.5.1-298) observed that both the abundance and diameters of pre-1886 Holocene sand blow craters are greatest within the meizoseismal zone of the 1886 Charleston earthquake. No paleoliquefaction features were observed beyond 100 km (60 mi.) from Charleston (Reference 2.5.1-302).

Amick et al. (Reference 2.5.1-270) searched for paleoliquefaction features in late Quaternary beach and near-shore deposits (i.e., deposits susceptible to liquefaction) at over 1000 potential liquefaction sites in Virginia, North Carolina, South Carolina, Georgia, and the Wilmington, Delaware, area. A map showing the 7.5-minute quadrangles in which Amick et al. conducted reconnaissance studies in North Carolina and South Carolina (their Area 2) is shown on Figure 2.5.1-258. Over 320 sites were evaluated in this area. Their search identified liquefaction features almost exclusively in South Carolina, with one exception, the Calabash liquefaction feature discovered directly north of the South Carolina – North Carolina state line (see discussion in Subsection 2.5.1.1.4.2.5.3.3 below). The lack of paleoliquefaction features outside the Charleston area provided by Obermeier et al. (Reference 2.5.1-302)

and Amick et al. (Reference 2.5.1-300) strongly suggests that the seismic source that produced the 1886 Charleston earthquake and earlier large-magnitude earthquakes is localized in the Charleston meizoseismal area.

Talwani and Schaeffer (Reference 2.5.1-301) and others evaluated the distribution of 1886 liquefaction and earlier paleoliquefaction features to assess the geometry as well as the stationarity or nonstationarity of the Charleston seismic source. Evaluations of the location, magnitude, and recurrence of large-magnitude events in the Charleston region based on the size, age, and distribution of paleoliquefaction features are presented in Subsection 2.5.1.1.4.3.

Liquefaction features near Georgetown, South Carolina, and two sites in North Carolina near the border with South Carolina identified by Amick et al. (Reference 2.5.1-270) and Talwani and Schaeffer (Reference 2.5.1-301) are distant from the Charleston epicentral region. These sites, discussed below, could be indicative of a local source rather than a distant source.

Georgetown Liquefaction Features (Class A). Crone and Wheeler (Reference 2.5.1-258) provided a summary of prehistoric liquefaction features found near Georgetown, South Carolina, approximately 100 km (60 mi.) northeast of Charleston, South Carolina (Reference 2.5.1-258). These features lie approximately 320 km (200 mi.) south of the HAR site. Although the zone in which those features occurs adjoins or merges with the area of liquefaction that is attributed to earthquakes near Charleston, the features near Georgetown are interpreted to be caused by local earthquakes instead of by a distant earthquake source near Charleston. The Georgetown liquefaction features are attributed to a single large prehistoric earthquake, as the causative fault has not been identified. (Reference 2.5.1-258)

Talwani and Schaeffer (Reference 2.5.1-301) assessed the sand blows at the Georgetown site to indicate two or possibly three episodes of liquefaction. Based on calibrated ages of radiocarbon samples from sand blows at multiple sites in South Carolina, seven episodes of prehistoric liquefaction in the past 6000 years and two scenarios for prehistoric seismicity in the region are identified by Talwani and Schaeffer (Reference 2.5.1-301; Table 2.5.1-205). In the first scenario there are three possible seismic source zones: near Charleston, near Georgetown (northern source), and near Bluffton (southern source). In the second scenario, all earthquakes occur in the Charleston seismic zone. Talwani and Schaeffer (Reference 2.5.1-301) state that more data are needed to resolve which of the two scenarios is most probable.

2.5.1.1.4.2.5.3.3 Paleoliquefaction Features Identified in North Carolina

Paleoliquefaction features have been identified at two sites in North Carolina: the Calabash site of Amick et al. (Reference 2.5.1-270) and the Southport site of Talwani and Schaeffer (Reference 2.5.1-301). The Calabash site was one of eight liquefaction sites identified north of the meizoseismal area of the 1886 Charleston earthquake. Detailed investigations were conducted at five of the

eight sites, but the Calabash site was not included among the five. The only site-specific information Amick et al. (Reference 2.5.1-270) provide on the Calabash site is “several pre-1886 features” as a note on a map showing the location of the site (Figure 11.1 in Reference 2.5.1-270). The Southport site is described as the northernmost site examined from which datable material was recovered (Reference 2.5.1-301). This material consisted of pieces of charcoal embedded deeply in intensely deformed soil profiles; a maximum age of 9743 $\pm$ 167/–208 years B.P. was obtained on this material. As described above for the Georgetown features, the paleoliquefaction features at the Southport site could be related to either a northern seismic source or the Charleston seismic source; there are insufficient data available to resolve between the two scenarios.

2.5.1.1.4.3 Charleston Seismic Source

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 2.5.1-309). Based on the felt intensity reports defining the meizoseismal area (area of maximum damage) and the occurrence of continuing seismic activity (the Middleton Place – Summerville seismic zone), the epicentral region of the 1886 earthquake is considered to be centered northwest of Charleston, 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 HNP site (Reference 2.5.1-203). Several investigations that postdate the EPRI-SOG evaluation (Reference 2.5.1-203) indicate that parameters related to location, maximum magnitude, and recurrence of possible seismic sources in the Charleston region should be updated. In support of the Southern Nuclear Company Vogtle ESP Application, a thorough review and analysis of the new data were completed (Reference 2.5.1-264). The UCSS model for the Vogtle ESP Application, which was adopted for this study, is further discussed in 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 as follows.

2.5.1.1.4.3.1 Location and Geometry

Several recent studies provide direct or indirect evidence regarding the location and geometry of the Charleston seismic source. 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 2.5.1-221,2.5.1-222, and 2.5.1-223. Features are differentiated to show both pre- and post-1986 EPRI

information. Recent post-EPRI studies that have identified tectonic features in the 1886 Charleston meizoseismal area include those by Marple and Talwani (Reference 2.5.1-262, Reference 2.5.1-243, Reference 2.5.1-287); Weems et al. (Reference 2.5.1-288); Weems and Lewis (Reference 2.5.1-289); Talwani and Katuna (Reference 2.5.1-290); and Talwani and Durá-Gómez (Reference 2.5.1-370, Reference 2.5.1-377, Reference 2.5.1-378). In particular, five postulated faults have been identified in the Charleston area since 1986 (Table 2.5.1-203 and Figure 2.5.1-221) and additional information has been developed on the offshore Helena Banks fault zone. A description of the East Coast fault system is provided in Subsection 2.5.1.1.4.2.5.2.2. Descriptions of the other faults or postulated faults in the Charleston area are described below.

Adams Run Fault. Weems and Lewis (Reference 2.5.1-289) postulated the existence of the Adams Run fault on the basis of microseismicity and borehole data (Figures 2.5.1-221 and 2.5.1-224). Weems and Lewis's interpretation of borehole data suggests the presence of areas of Cenozoic uplift and subsidence separated by the inferred fault (Figure 2.5.1-224). Based on a review and analysis of these data for the SNC Vogtle ESP Application (Reference 2.5.1-264), it was concluded that (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. It was also noted that there is no geomorphic evidence for the existence of the Adams Run fault, and a 3-D analysis of microseismicity in the vicinity of the proposed Adams Run fault completed for that study did not clearly define a discrete structure.

Talwani (Reference 2.5.1-307) associates seismicity observed in the Adams Run seismic zone with the Woodstock fault, and this fault is not shown as a structure on the current seismotectonic framework map of the Charleston area by Durá-Gómez and Talwani (Reference 2.5.1-378; Figure 2.5.1-259).

Ashley River Fault. Talwani (Reference 2.5.1-307), 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 (Reference 2.5.1-307, Reference 2.5.1-289, Reference 2.5.1-379). 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. (Reference 2.5.1-370).

Charleston Fault. Lennon (Reference 2.5.1-380) proposes the Charleston fault on the basis of geologic map relations and subsurface borehole data. Weems

and Lewis (Reference 2.5.1-289) 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 by 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 2.5.1-264; Figure 2.5.1-225)

Talwani and Durá-Gómez (Reference 2.5.1-370) 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 2.5.1-370) 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 2.5.1-381) and Hamilton et al. (Reference 2.5.1-382) 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.) (Reference 2.5.1-262, Reference 2.5.1-243). Marple and Talwani (Reference 2.5.1-243, Reference 2.5.1-262) reinterpret these data to suggest that the Cooke fault may be part of a longer, more northerly striking fault (i.e., the ZRA of Marple and Talwani [Reference 2.5.1-262] and the ECFS of Marple and Talwani [Reference 2.5.1-243]). Crone and Wheeler (Reference 2.5.1-258) classify the Cooke fault as a Class C feature based on lack of evidence for faulting younger than Eocene.

Helena Banks Fault Zone. The Helena Banks fault zone is clearly imaged on seismic reflection lines offshore of South Carolina (Reference 2.5.1-292, Reference 2.5.1-293) 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 2.5.1-264):

- 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 (Figures 2.5.1-221 and 2.5.1-222).

- Bakun and Hopper (Reference 2.5.1-294) 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 2.5.1-294) ultimately conclude, however, that the epicentral location most likely lies onshore in the Middleton Place –

Summerville area (Figures 2.5.1-222 and 2.5.1-226) based on the concentrated seismicity in this area.

- Crone and Wheeler (Reference 2.5.1-258) 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 2.5.1-264), 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 >1 km (>0.6 mi.). Furthermore, the occurrence of 2002 earthquakes and the location of the Bakun and Hopper (Reference 2.5.1-294) 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 2.5.1-364).

Lincolnville Fault. This fault is defined by Durá-Gómez and Talwani (Reference 2.5.1-378) as one of three northwest-southeast-striking reverse faults recognized within an approximately 6–km- (3.7-mi.-) long antidilational compressional left step of the Woodstock fault near Middleton Place. The Lincolnville fault dips steeply to the northeast. Minor earthquake activity is associated with this fault.

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 of microseismicity (Reference 2.5.1-290). According to Talwani and Katuna (Reference 2.5.1-290), 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 2.5.1-221). 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 (<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 2.5.1-264). The general conclusion from this study was that the features are almost certainly the product of shaking effects as opposed to fault rupture (Reference 2.5.1-264).

Durá-Gómez and Talwani (Reference 2.5.1-378) 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 2.5.1-378).

Summerville Fault. Weems et al. (Reference 2.5.1-288) postulated the existence of the Summerville fault on the basis of microseismicity (Figure 2.5.1-221). Based on a review and analysis of these data for the SNC Vogtle ESP Application (Reference 2.5.1-264). it was concluded that there is no geomorphic or borehole evidence for the existence of the Summerville fault, and microseismicity in the vicinity of the proposed Summerville fault does not clearly define a discrete structure (Figure 2.5.1-225).

Woodstock Fault. Talwani (Reference 2.5.1-288) 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 2.5.1-377) 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 2.5.1-370). Based on a review of available geomorphological, geodetic, shallow stratigraphic, seismic reflection, refraction, and potential field data, Talwani and Durá-Gómez (Reference 2.5.1-370) 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 2.5.1-370). Talwani and Durá-Gómez (Reference 2.5.1-370) 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 2.5.1-378) 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 2.5.1-383) 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.

Association with Mesozoic Basins

Johnston et al. (Reference 2.5.1-295) evaluated the correlation of large-magnitude intraplate earthquakes to specific tectonic environments throughout the world. They concluded that large-magnitude earthquakes generally occur in tectonic environments characterized by Mesozoic and younger rifted crust. The Charleston meizoseismal region occurs in a region of Mesozoic extended crust along the southeastern margin of the North American craton (Reference 2.5.1-295). 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 Southern Nuclear Company in the Vogtle ESP assessment of the updated Charleston seismic source to characterize alternative models of the source zone geometry (Reference 2.5.1-264). Talwani and Durá-Gómez (Reference 2.5.1-370) inferred that the northern segment of the Woodstock fault correlates spatially to the southeast margin fault of the Mesozoic Jedberg basin, suggesting that a

Mesozoic basin-bounding fault has been reactivated as an oblique right-lateral-slip fault with up to the northwest displacement.

Paleoliquefaction Features

Based on the geographic and temporal distribution of paleoliquefaction features in coastal South Carolina, Talwani and Schaeffer (Reference 2.5.1-301) 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 2.5.1-301) second scenario, all events recorded in the paleoliquefaction record were centered at Charleston with magnitudes ≥ approximately 7.

Intensity Data

Intensity data for the 1886 Charleston earthquake reported by Dutton (Reference 2.5.1-296) and reinterpreted by Bollinger (Reference 2.5.1-303) indicate a meizoseismal area centered on Charleston (Figures 2.5.1-221 and 2.5.1-222). Bakun and Hopper (Reference 2.5.1-294) calculated an intensity center for the 1886 Charleston earthquake that is located offshore about 200 km (125 mi.) east of Charleston (Figure 2.5.1-226). 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 2.5.1-294). Bakun and Hopper's (Reference 2.5.1-294) preferred intensity center for the 1886 Charleston earthquake is onshore within the Middleton Place – Summerville seismic zone.

2.5.1.1.4.3.2 Maximum Magnitude

As outlined in the SNC Vogtle ESP Application (Reference 2.5.1-264), 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 2.5.1-295) 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 2.5.1-204). The latter approach has used both intensity data (Reference 2.5.1-309, Reference 2.5.1-294) and the size and geographic distribution of the liquefaction fields (Reference 2.5.1-297, Reference 2.5.1-298, Reference 2.5.1-302, Reference 2.5.1-309) to estimate the magnitude of the 1886 event.

Because the causative fault for the 1886 event is unknown, the Southern Nuclear Company Vogtle ESP Application (Reference 2.5.1-264) 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 2.5.1-295) 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 2.5.1-295). 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 2.5.1-295) 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 2.5.1-310) 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 2.5.1-311) and the cyclic stress method (e.g., Reference 2.5.1-312). Martin and Clough (Reference 2.5.1-310) 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 2.5.1-204).

Johnston (Reference 2.5.1-309) 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 2.5.1-204). The Johnston study (Reference 2.5.1-309) 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 2.5.1-295) of **M** 7.56 ± 0.35. Johnston (Reference 2.5.1-309) 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 2.5.1-309) 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 2.5.1-313) showed that

eastern North America exhibits lower attenuation of seismic energy than other worldwide stable continental regions. Bakun and McGarr (Reference 2.5.1-313) 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 2.5.1-309) may have overestimated the magnitude of the 1886 Charleston earthquake.

Bakun and Hopper (Reference 2.5.1-294) estimated the magnitude and location of the 1886 Charleston earthquake using eastern North America intensity models that relate intensity and epicentral distance (Reference 2.5.1-314). 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 2.5.1-294) estimated a magnitude range of **M** 6.4 to 7.2 at the 95 percent confidence interval. Bakun and Hopper's (Reference 2.5.1-294) preferred magnitude estimate for the Charleston earthquake is M_I 6.9 (M_I is considered equivalent to **M**). The Bakun and Hopper (Reference 2.5.1-294) magnitude estimate suggests that the 1886 event may have been smaller than the Johnston (Reference 2.5.1-309) estimate. Both estimates are considered credible and are included in the UCSS model (Reference 2.5.1-264).

Obermeier et al. (Reference 2.5.1-297, Reference 2.5.1-298, Reference 2.5.1-302) 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 2.5.1-298) 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 2.5.1-302) suggested that prehistoric Charleston-area earthquakes were probably at least as strong as the 1886 Charleston earthquake.

For paleoearthquakes, Talwani and Schaeffer (Reference 2.5.1-301) estimated the magnitudes of past Charleston-area events based on the spatial distribution and areal extent of paleoliquefaction sites (Figure 2.5.1-227). Talwani and Schaeffer (Reference 2.5.1-301) 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. (Reference 2.5.1-315, Reference 2.5.1-316) used the event chronology as interpreted by Talwani and Schaeffer (Reference 2.5.1-301) 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 2.5.1-316) 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 2.5.1-317) and Leon et al. (Reference 2.5.1-318) also estimated the magnitudes of past Charleston-area earthquakes using the event chronology as interpreted by Talwani and Schaeffer (Reference 2.5.1-301), but the Leon (Reference 2.5.1-317) and Leon et al. (Reference 2.5.1-318) 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 2.5.1-318) 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 2.5.1-318) 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 2.5.1-263, Reference 2.5.1-264) concluded the following:

- The magnitude ranges estimated for earthquakes that produced liquefaction over extended areas (Reference 2.5.1-315, Reference 2.5.1-316, Reference 2.5.1-318) have significant overlap with magnitude estimates of the 1886 earthquake by Johnston (Reference 2.5.1-309) and Bakun and Hopper (Reference 2.5.1-294). 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 2.5.1-204).

- The magnitudes estimated from the paleoliquefaction record for earthquakes that produced liquefaction over limited areas may have been less than **M** 6.3 (Reference 2.5.1-318). 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 (Reference 2.5.1-310, Reference 2.5.1-309, Reference 2.5.1-294; Table 2.5.1-204). The 95 percent confidence interval of Bakun and Hopper (Reference 2.5.1-294) 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.

2.5.1.1.4.3.3 Recurrence

Post-1986 EPRI studies of paleoliquefaction features (e.g., Reference 2.5.1-299, Reference 2.5.1-270, Reference 2.5.1-300, Reference 2.5.1-301) 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 2.5.1-298) or **M** 4.3 – 6.4 (Reference 2.5.1-318). It was noted in the SNC Vogtle ESP Application (Reference 2.5.1-264) 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 2.5.1-299) and Amick et al. (Reference 2.5.1-300, Reference 2.5.1-270) 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 2.5.1-301) combined previously published data with their own studies of liquefaction features in the South Carolina coastal region (Figure 2.5.1-227). Talwani and Schaeffer (Reference 2.5.1-301) used the spatial distribution of paleoliquefaction features in combination with estimates on the timing of the formation of the liquefaction features in order to derive possible earthquake recurrence histories for the region. Talwani and Schaeffer's first scenario (Reference 2.5.1-301) 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 2.5.1-301), all earthquakes in the record are large shocks (approximately **M** 7+) located near Charleston. Talwani and Schaeffer's (Reference 2.5.1-301) 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 2.5.1-301) 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 2.5.1-384). (The 1-sigma error bands used by Talwani and Schaeffer [Reference 2.5.1-301] 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 2.5.1-301) scenario in which some smaller moderate-magnitude events are recognized.

2.5.1.1.4.4 Regional Gravity and Magnetic Data

Kane and Godson and Jachens et al. present regional maps of the gravity and magnetic fields of North America at scales of 1:7,500,000 and ~1:20,000,000, respectively, and discuss the interpretations of these data with regard to evaluating crustal-scale structures (Reference 2.5.1-319, Reference 2.5.1-320). These data are described in the following subsections. Regional-scale maps showing the gravity and magnetic fields are presented on Figures 2.5.1-228 and 2.5.1-229. More detailed geophysical investigations that provide information on the structural geology within the site vicinity (40-km [25-mi.] radius) are discussed in Subsection 2.5.1.2.4.1.

2.5.1.1.4.4.1 Gravity Data

Jachens et al. (Reference 2.5.1-320) present an isostatic residual gravity map of the conterminous United States and an evaluation of mid- to upper-crust structures based on correlation of conspicuous short-wavelength anomalies (widths less than several hundred kilometers [several hundred miles]) with mapped or near-surface geologic features (Reference 2.5.1-320). Sources of interpretative gravity references for the HAR site region cited by Jachens et al. include Mann and Zablocki, Best et al., and Pratt et al. (Reference 2.5.1-321, Reference 2.5.1-322, Reference 2.5.1-323). A more recent compilation of gravity data for the United States by Dater et al. is shown on a Bouguer anomaly map on Figure 2.5.1-228 (Reference 2.5.1-324).

On a regional scale, the Appalachian suture zone, which represents a collisional boundary between major plates, is marked by a nearly continuous, northeast-southwest-trending, 2900 km (1800 mi.) long gravity high. This suture is revealed on a vertical derivative map by contrasting patterns (Reference 2.5.1-320).

Kane and Godson note that geophysical anomalies in the Appalachians trend predominantly northeast-southwest and that the gravity pattern in the site region is characterized by two parallel northeast-southwest-trending chains of elongate highs that extend from northern Vermont to central Alabama (Reference 2.5.1-319). The chains generally are separated by a series of lows, are characterized by moderate to high intensity, and are flanked outwardly by lows that generally are less intense and continuous. It is suggested that the sources of the southeast chain, which is characterized by higher, steeper, and more asymmetric gradients, are shallower or more massive than those that form the northwest chain and that these structures form interfaces with surrounding rock units that dip at steeper angles. (Reference 2.5.1-319) The northwestern gravity high follows the trend of the Blue Ridge province, whereas the southeast gravity high follows the trend of the central Piedmont suture. The gravity highs parallel the structural fabric of the Appalachians and lie over crystalline terrane. Deformation of the terrane is implied by the steep gradients and discontinuous sharp outlines of anomalies (Reference 2.5.1-319).

The Appalachian gravity gradient, also referred to as the Piedmont gravity gradient, separates low Bouguer (~75 milligals [mGal]) regions in the interior from high Bouguer (~0 mGal) gravity in the Piedmont and Coastal Plain. Phinney and Roy-Chowdhury suggest that the region east of this gravity gradient underwent general extension and thinning on the order of 5 to 10 percent during the opening of the Atlantic, and as a result, nearly every major thrust, detachment, or suture may have reactivated by normal faulting during that period (Reference 2.5.1-325).

The southeastern limit of Iapetan normal faults defines the extent of the Iapetan margin of the craton containing normal faults that accommodated extension during the late Proterozoic to early Paleozoic rifting of the Iapetan Ocean and is nearly coincident with the Appalachian gravity gradient (Figure 2.5.1-228) (Reference 2.5.1-244).

2.5.1.1.4.4.2 Magnetic Data

Continent-scale aeromagnetic maps data presented by Kane and Godson and the North American Magnetic Anomaly Group (NAMAG) reveal the overall northeast-southwest structural features of the Appalachians (Reference 2.5.1-319, Reference 2.5.1-326). Kane and Godson identify two broad areas within the Appalachians subzone, which include the entire site region, that are typified by broad, low-amplitude regional highs and relatively few local anomalies (Reference 2.5.1-319). The trends of the magnetic anomalies in the site region are generally consistent with the orientations of the gravity anomalies, but there is no direct correlation between the magnetic and gravity anomalies. (Reference 2.5.1-319) The magnetic anomalies are attributed to sources in the crystalline basement, volcanic units within about a kilometer of the surface, or igneous intrusions (Reference 2.5.1-319). Short-wavelength magnetic features most likely represent the detailed structure of the near-surface basement rock units.

Several large-scale magnetic anomalies in the eastern United States form lineaments that have been associated with seismic zones or basement structures. Magnetic anomalies extending from Alabama to Virginia mark the Eastern Piedmont fault system (Figure 2.5.1-229). These magnetic anomalies coincide with cataclastic rocks and also with a low velocity zone likely due to fracturing of the crystalline rocks during recurrent faulting. (Reference 2.5.1-327) The New York – Alabama lineament is marked by a series of aligned magnetic gradients that roughly coincide with the west side of the Appalachian gravity low. The lineament appears to mark the southeast edge of a stable crustal block that acted as a buttress for deformation in the Appalachians. (Reference 2.5.1-328) The Clingman and Ocoee lineaments lie to the east of the New York – Alabama lineament (Figure 2.5.1-229) and are magnetic anomalies produced by deep-seated basement structures (Reference 2.5.1-283). These three lineaments bound the Eastern Tennessee seismic zone (see Subsection 2.5.1.1.4.2.5.1.4). The east coast magnetic anomaly (Figure 2.5.1-229) marks the present continental/oceanic crustal transition and the late Paleozoic suture between North America and Africa (Reference 2.5.1-329).

The aeromagnetic map of North Carolina, at a scale of 1:1,000,000, reveals more detailed features of the study region (Reference 2.5.1-330). The use of magnetic maps to delineate features in the Coastal Plain of the Carolinas has focused on identifying the magnetic character or grain of distinctive terranes (Reference 2.5.1-220). Prominent magnetic anomalies in the Coastal Plain near the fall line are inferred to represent the subsurface continuation of the Piedmont crystalline rocks. It is suggested that farther east, where the field is smoother, the Piedmont basement terrane is depressed and overlain by a thick section of nonmagnetic early Mesozoic sedimentary rocks. Intrusions of diabase sheets are suggested by subtle arcuate magnetic anomalies in areas of a generally flat magnetic field. (Reference 2.5.1-220)

2.5.1.2 Site Geology

The following subsections provide a summary of geologic conditions in the HAR 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 HAR site. The information presented is based on a review of previous HNP reports and documents, a review of 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 HAR site. Geologic maps of the site vicinity (40-km [25-mi.] radius), site area (8-km [5-mi.] radius), and site (1-km [0.6-mi.] radius) are shown on Figures 2.5.1-230, 2.5.1-231, and 2.5.1-232, respectively.

2.5.1.2.1 Site Physiography and Topography

The HAR site is located in the upland section of the Piedmont province, approximately 6.5 km (4 mi.) west of the eastern margin of the Deep River basin,

where the Jonesboro fault separates Triassic rift-fill sediments from igneous and metamorphic rocks of the Piedmont Plateau (Figures 2.5.1-230 and 2.5.1-231). The Deep River basin is filled by a complex, wedge-shaped block of Triassic rocks consisting mostly of claystone, shale, siltstone, sandstone, and conglomerate. These sedimentary rocks are intruded in places by diabase dikes and sills of Triassic-Jurassic age (e.g., Reference 2.5.1-331, Reference 2.5.1-202, Reference 2.5.1-220). The basin is bounded on the northwest, north, and east by the upland surface of the Piedmont Plateau and on the southeast and south by the upland surface of the Coastal Plain (Figure 2.5.1-201). It has been divided into three subbasins: the Durham, Sanford, and Wadesboro basins (Figures 2.5.1-202 and 2.5.1-215). The site lies in the southern part of the Durham basin, just north of the Colon cross structure, a basement high that separates the Durham and Sanford basins. (Reference 2.5.1-204) The Triassic sediments of the Deep River basin form a trough-like topographic lowland, because the sediments are more easily eroded than the igneous and metamorphic rocks of the Piedmont Plateau or the porous sand and gravel deposits of the Coastal Plain. The Triassic lowlands in the site area, which generally lie between 15 and 91 m (50 and 300 ft.) above sea level, are characterized by gently rolling hills and low-gradient streams. The gentle topography is the result of a long period of erosion that has stripped away large volumes of relatively soft Triassic rocks, leaving the Deep River basin as a muted topographic low. (Reference 2.5.1-202)

Most of the streams in the Triassic lowland have a dendritic pattern and are actively deepening their valleys. In general, valley sides are comparatively steep and valley bottoms are narrow, but some of the larger streams are bordered by narrow floodplains. Interstream divides are sharp and narrow near the major streams, becoming higher, broader, and flatter away from major drainages. Because of the muted topographic features produced by a long period of topographic inactivity, there are no landslides, areas of uplift or subsidence, or other natural features that potentially could be hazardous to the plant. (Reference 2.5.1-202)

The Piedmont Plateau west and northwest of the Deep River basin is characterized by Precambrian and Paleozoic intrusive rocks. East of the Jonesboro fault and within 40 km (25 mi.) of the site, the Piedmont is characterized by igneous and metamorphic rocks and gently rolling hills and valleys that become progressively flatter to the east as the topography transitions into Coastal Plain sediments (Reference 2.5.1-208).

2.5.1.2.2 Geologic History of Site Area

This subsection presents an overview of the geologic history of the site area and vicinity. The overall geologic history and tectonic framework of the region are outlined in Subsections 2.5.1.1.2 and 2.5.1.1.4. A detailed discussion of the age of faulting within the site vicinity is provided in Subsections 2.5.1.2.4 and 2.5.3. The following geologic history of the area around the HAR site is summarized on Figure 2.5.1-260 and is based on the HNP FSAR and more recent detailed

studies (e.g., Reference 2.5.1-202, Reference 2.5.1-204, Reference 2.5.1-220, Reference 2.5.1-226, Reference 2.5.1-251, Reference 2.5.1-332).

During the Late Proterozoic and Paleozoic, igneous and metamorphic rocks of the Carolina zone and Raleigh slate belt were formed during convergence of the North American and African continents (see Subsection 2.5.1.1.2). In the site vicinity, these rocks comprise the Carolina, Raleigh, and Spring Hope terranes (Figure 2.5.1-244; Reference 2.5.1-352). The three terranes are composed of Precambrian and Paleozoic igneous and metamorphic rocks that underlie the Triassic Deep River basin sediments. Late Proterozoic and Paleozoic rocks associated with this convergence are exposed east of the Jonesboro fault in the area of the Nutbush Creek fault zone (Figure 2.5.1-230, Sheet 1). The site is located in the Carolina terrane, which is believed to represent a long-lived suprasubduction zone magmatic arc system (Reference 2.5.1-352). Igneous and metamorphic rocks of the Carolina, Raleigh, and Spring Hope terranes are discussed in Subsection 2.5.1.2.3.1.

During the Alleghanian orogeny in the Late Carboniferous and earliest Permian, the site vicinity was affected by the convergence of North America and Africa and their subsequent suturing during a continent-to-continent collision (Reference 2.5.1-212). During this time, the late Proterozoic to Paleozoic rocks and a number of younger intrusive rocks in the site vicinity were subjected to regional greenschist to amphibolite facies metamorphism (Reference 2.5.1-385). Also during this time, the Rolesville batholith was emplaced, and late Paleozoic ductile mylonite zones were formed that had dominantly right-lateral strike-slip displacement of the Nutbush Creek fault zone (Figure 2.5.1-230, Sheet 1; Reference 2.5.1-251).

During the early Mesozoic, rifting of the supercontinent Pangea created a series of irregularly shaped half-graben along the Atlantic margin of North America, including the Deep River basin (Reference 2.5.1-204). Triassic extension between the North American and African plates resulted in reactivation of some northeast-striking Paleozoic faults, including major structures associated with the Deep River basin (i.e., the Jonesboro and Bonsal-Morrisville faults) (Reference 2.5.1-220). During the Triassic and Jurassic, movement on the Jonesboro fault was tensional, with a component of right-lateral slip. Movement on the Harris fault was primarily tensional, with a minor left-lateral component. This difference in styles resulted in a clockwise sense of rotation between the two faults. This episode of movement on the Jonesboro fault, Harris fault, and other minor faults and folds in the Durham basin was accompanied by intrusion of diabase dikes during Late Triassic – Jurassic time (see Subsection 2.5.1.2.4.1.1). (Reference 2.5.1-202)

The development and timing of deformational events within the Deep River basin are consistent with a regional model for the development and timing of rifting, drifting, and basin inversion for Mesozoic basins along the eastern North American margin proposed by Schlische et al. (Reference 2.5.1-333). In this model, the initial period of rifting is followed by a period of transition from rifting to

drifting. Widespread magmatism and shortening/inversion in eastern North America are thought to be related to active upwelling of the asthenosphere that culminated during the rift-drift transition. Inversion, a common feature along volcanic passive margins, is associated with a change in the strain state from (a) extension at a high angle to the margin during rifting to (b) shortening at a high angle to the margin during drifting. The dikes that are oriented at a high angle to the trend of the Deep River basin margin and the evidence for lateral slip along the Harris fault may reflect this change in strain state associated with inversion. (Reference 2.5.1-333)

Development and subsequent geologic evolution of the Durham basin during the Triassic and Jurassic is outlined in the following sequence:

Triassic subsidence of the Durham basin in the hanging wall of the Jonesboro fault and concurrent uplift of the rift flank produced a half-graben that was filled with continental fluvial and lacustrine sediments (Reference 2.5.1-202). The sediments were deposited as (1) alluvial fans along faulted margins of the basin; (2) large alluvial plains around meandering river systems; and (3) deltas, lacustrine deposits, and paludal deposits (Reference 2.5.1-204). During the Triassic, sediments accumulated in the rift basin, probably attaining a thickness several thousand feet greater than what remains today and covering a much broader area (Reference 2.5.1-202). The thick sedimentary package resulted in consolidation of the deeper sediments and in the lithification of the remaining rocks (Reference 2.5.1-202). Sedimentation may have ended before or near the Triassic-Jurassic boundary, with some subsequent erosion of synrift sediments (Reference 2.5.1-220).

A second phase of extension that began during the Early Jurassic was oriented differently from the Late Triassic extension, producing sets of north-south and northwest-southeast faults and dikes and the intrusion of diabase sheets in the Durham subbasin of the Deep River basin (Reference 2.5.1-220). Diabase dike intrusion at the site was contemporaneous with movement on the Harris fault, although the Harris fault was active both before and after intrusion of the dikes. Ages of the diabase dikes range from approximately 150 to approximately 225 Ma, based on remnant magnetization studies, and from approximately 168 to approximately 260 Ma, based on potassium-argon dating. (Reference 2.5.1-202) Secondary minerals formed in amygdules in the dikes include heulandite and harmotome, which would form at temperatures of approximately 150°$\pm$50°C. Ambient temperatures in the sedimentary rocks at shallow depth of intrusion are suggested by undeformed amygdules and crystal-lined open cavities in the dikes as being lower than 100°C (around 40°C at 300 m [1000 ft.] with a 50°C/km geothermal gradient).

Because there is no evidence of other activity in this area since middle Mesozoic time, it is likely that the heat necessary for hydrothermal solutions to produce these minerals came from the dikes. (Reference 2.5.3-202) Amygdules in some of the dikes at current surface levels in the site vicinity indicate that the dikes were intruded when the ancestral surface was less than about 300 m (1000 ft.)

above the present surface. This association indicates that a major period of erosion followed deposition of the original thicknesses of Triassic sediments, but occurred before intrusion of the diabase dikes. It is possible that some of these secondary minerals were formed during a later burial metamorphic event, rather than by deuteric hydrothermal activity. If this is the case, then continued downfaulting of the graben and sediment accumulation must have buried the dikes much deeper than the level of crystallization. Assuming a high geothermal gradient (50C/km), a temperature of 150°C requires a depth of burial of almost 3 km; it would have been necessary for this much overburden to have been removed since middle Mesozoic time. The Jurassic burial-metamorphic event, more than 150 Ma, is the only such event recorded in the dike rocks (as paleomagnetic chemical remnant magnetization [CRM]). (Reference 2.5.1-202)

Petrographic and chemical studies on the dikes completed during the HNP siting investigations provided a critical test for field observations concerning relative ages of dikes and the Harris fault. Field evidence established that composite East Dike 2 was intruded during fault movement; West Dike 3S was intruded after most movement had occurred but before a final, minor element of movement; and West Dike 3 was probably intruded early during movement and offset about 3 m (10 ft.) left-laterally (Figure 2.5.3-201). Observations where the Harris fault crosses East Dike 2 suggest that the fault is a minor, late contemporary feature to the Jonesboro fault. (Reference 2.5.3-201) The Jonesboro fault is deeper and rooted in the crust, whereas the Harris fault is likely rooted in the Triassic sediments (Reference 2.5.3-202). Detailed mapping of the diabase dikes in relation to the Harris fault suggests the following sequence of events on the fault (Reference 2.5.3-202):

- Movement on the fault.
- Intrusion of the easternmost dike segment.
- Continued movement along the fault.
- Intrusion of the central dike segment.
- Continued movement along the fault.
- Intrusion of the westernmost dike segment.
- Minor continuing movement along the fault.
- Crystallization of laumontite.
- Final movement on the fault.
- Low-grade burial metamorphism, with crystallization of zeolites harmotome and heulandite.

After intrusion of the dikes, major movement continued on the Jonesboro fault south of the site. Movement may have ended more quickly north of the site, because a dike, probably of this period, appears to cross the Jonesboro fault with very little offset. Alternately, the dike was emplaced during the last stages of movement on the fault. Little movement took place on the Harris fault after intrusion of the youngest dikes, which are Jurassic in age. (Reference 2.5.1-202)

During later Jurassic time, the surface of Triassic sediments, which was only slightly higher than the present surface, was buried under 600 to 2700 m (2000 to 9000 ft.) of sediments. This burial is suggested both by the regional low-grade metamorphic event determined from chemical remnant magnetization of the dikes before 150 Ma and by the crystallization of higher-temperature secondary minerals, including zeolites, in the gouge of the Harris fault at that time. The range in possible depth of burial noted previously reflects the lack of precise knowledge of the heat flow regime in the rocks at that time. If a normal continental heat-versus-depth relationship is postulated to provide the temperatures needed for crystallization of the secondary zeolite minerals, then the depth of burial was great. Following the Jurassic metamorphic event, erosion of large quantities of sediments from the Triassic basin and west of the basin resulted in deposition of great quantities of Late Jurassic – Cretaceous marine sediments in the Coastal Plain. (Reference 2.5.1-202)

During Cenozoic time, the area around the HAR site has been subject to erosion along the passive continental margin. Streams continue to downcut through Triassic sediments in many areas of the basin. Younger sediments associated with higher stands of sea level, such as those occurring during the Cretaceous and Cenozoic, are preserved in some areas of the site vicinity (Figure 2.5.1-230, Sheet 1), and Quaternary terrace deposits occur along the Cape Fear River. (Reference 2.5.1-202) Cretaceous and Cenozoic units preserved in the site vicinity are shown on Figure 2.5.1-230 (Sheet 1), and include the Cretaceous Cape Fear and Middendorf formations and the Tertiary Castle Hayne and Duplin formations and younger terrace deposits (Reference 2.5.1-202). These Coastal Plain sediments were deposited during transgressive-regressive cycles caused by eustatic sea level fluctuations (see Subsection 2.5.1.1.3.3) (Reference 2.5.1-235).

Cenozoic deformation in the site region consists of broad regional uplift and folding, the Cape Fear arch being the most significant example of regional Cenozoic tectonism near the site vicinity (Reference 2.5.1-242). Prowell and Obermeier note, however, that evidence for neotectonism across the Cape Fear arch is difficult to evaluate because of the limited distribution and thicknesses of post-Miocene strata. A discussion of the Cape Fear arch is provided in Subsection 2.5.1.1.4.2.4. Cretaceous and post-Cretaceous faulting is recognized in the region (Table 2.5.1-201), but there are no recognized Quaternary faults in the site vicinity (Reference 2.5.1-258, Reference 2.5.1-259) (see discussion in Subsection 2.5.1.1.4.2.5). Possible faulting (Pliocene or Pleistocene) was recognized at only one locality within the site vicinity, approximately 23 km (14 mi.) from the HAR site where upland gravels of unknown age are thrust over

metamorphic rocks (Reference 2.5.1-228). This feature is discussed in more detail in Subsection 2.5.1.2.4.3.

2.5.1.2.3 Stratigraphy of Site Area

Bedrock within the site area (an 8-km [5-mi.] radius) consists predominantly of well-consolidated Triassic siltstones and sandstones interbedded with subordinate shale, claystone, and conglomerate. Diabase intrusive rocks also are present in the site area and site vicinity (Figure 2.5.1-231; e.g., Reference 2.5.1-202). Additionally, Proterozoic and Paleozoic igneous and metamorphic rocks; remnants of the flat-lying, poorly consolidated sediments of the Coastal Plain; terrace gravels; and Holocene alluvial deposits occur in the site vicinity (Figures 2.5.1-230, 2.5.1-231, and 2.5.1-232).

2.5.1.2.3.1 Precambrian and Paleozoic Igneous and Metamorphic Rocks

Three distinct lithotectonic terranes are exposed in the site vicinity. The Carolina terrane, Raleigh terrane, and Spring Hope terrane are within the eastern portion of the Piedmont physiographic province defined as the Carolina zone (Figure 2.5.1-244; Reference 2.5.1-352). The three terranes are composed of Precambrian and Paleozoic igneous and metamorphic rocks that underlie the Triassic Deep River basin sediments. The Carolina terrane is the largest in the Carolina zone and is exposed largely to the west of the site, underlying sediments of the Deep River basin (Figure 2.5.1-245) The Carolina terrane is made up of four metavolcanic-dominated sequences that extend for more than 500 km from central Virginia to Georgia (Reference 2.5.1-352). Metamorphosed rocks of the Carolina terrane in the site vicinity include metamudstone; meta-argillite; and metavolcanic rocks (metamorphosed tuff, basalt, andesite, dacite, rhyolite, and volcaniclastic sediments) (Reference 2.5.1-208). The Carolina terrane is separated from the Raleigh and Spring Hope terranes by the Nutbush Creek fault zone (Figure 2.5.1-245). The Raleigh terrane is to the east of the site and is bounded to the west by a distinct, elongate body of strongly lineated granitoid orthogneiss in the Nutbush Creek fault zone (Figure 2.5.1-230; Reference 2.5.1-352). The metamorphic rocks of the Raleigh terrane in the site vicinity include amphibolite, biotite gneiss and schist, mica schist, felsic mica gneiss, phyllite, and schist (Reference 2.5.1-208). The Raleigh terrane is truncated to the southeast and juxtaposed against the Spring Hope terrane by the Macon fault zone (Reference 2.5.1-352). The Spring Hope terrane is bounded by the Nutbush Creek fault zone to the west and the Hollister fault zone to the east (Reference 2.5.1-352). The Spring Hope terrane is predominantly composed of greenschist facies metavolcanic rocks and associated metaplutonic and metasedimentary rocks (Reference 2.5.1-352). There is some controversy distinguishing the Raleigh terrane from the Spring Hope terrane; however, it is generally accepted that the biotite gneiss (620 Ma) is considered part of the Raleigh terrane, whereas younger dated (590 and 544 Ma) metavolcanic rocks are part of the Spring Hope terrane (Figure 2.5.1-245; Reference 2.5.1-352).

Excavations at the Main Dam south of the Jonesboro fault (Figure 2.5.1-231) during construction of the HNP encountered granite, hornblende-mica gneiss, mica schist, and layered quartz-feldspar gneiss (Reference 2.5.1-331). These rocks are described by Ebasco Services, as follows (Reference 2.5.1-331):

> Granites exposed at the Main Dam are composed predominantly of feldspar (orthoclase and plagioclase) and quartz, with minor amounts of biotite, chlorite, muscovite, and pyrite. They are light gray when fresh, creamy white when slightly weathered, and tan or buff when moderately to highly weathered. The granites are commonly foliated, although foliation is weak or absent in places. In many places, they are interlayered with intricately folded mica schist and/or hornblende-mica gneiss. Xenoliths of hornblende gneiss occur in the granite, especially where foliation is weak or absent. Much of the highly foliated granite appears to contain indistinct compositional layering. Hornblende-mica gneiss exposed at the Main Dam is a hard, strong, medium- to fine-grained rock composed of hornblende with subordinate amounts of plagioclase and biotite; chlorite and pyrite are common accessory minerals. Hornblende-mica gneiss is gray to black when fresh, bluish or greenish gray when slightly weathered, and rusty brown when moderately weathered. Mica schist occurs as complexly folded layers 0.03 cm to 3 m (0.01 to 10 ft.) thick within the granite and as thin layers along joint surfaces within the hornblende-mica gneiss. The schist is soft to moderately hard, weak to moderately strong, moderately weathered, fine grained, highly fissile, and composed predominantly of chlorite, biotite, and amphibole. Schist layers contain abundant quartz. Layered quartz-feldspar gneiss is light gray to dark brown, fine- to medium-grained, moderately hard, moderately strong, and slightly to moderately weathered. It is characterized by intimate interlayering of gneissic rock, composed mostly of quartz and feldspar, with schistose rock, composed of biotite and muscovite or, less commonly, hornblende and mica. Individual layers are from 0.03 cm to 3 m (0.01 to 10 ft.) thick. The higher quartz content of the gneiss distinguishes it from rocks mapped as granite. Orientation of foliation and compositional layering in rocks of the Main Dam area is highly variable due to the highly complex nature of the folding these rocks have undergone. These rocks appear to have undergone several periods of folding with isoclinal folding predominating.

2.5.1.2.3.2 Triassic Sedimentary Rocks of the Chatham Group

Triassic sedimentary rocks of the Deep River basin are part of the Chatham Group of the Newark Supergroup.The Newark Supergroup consists of sediments that fill the Mesozoic basins along the east coast of North America (Figure 2.5.1-213). (Reference 2.5.1-204) The sediments of the Deep River basin are composed largely of debris from nearby pre-Triassic metamorphic and igneous rocks; in places, the sediments contain abundant debris from nearby granite intrusive bodies. These sediments were deposited as alluvial fans, stream channel and floodplain deposits, and lake and swamp deposits (Reference 2.5.1-202, Reference 2.5.1-204). In the Deep River basin, the

Chatham Group consists of varying amounts of conglomerate, sandstone, siltstone, claystone, shale, and coal, and small amounts of limestone, chert, and gypsum. The alluvial fans, angularity of sand grains, poor sorting of fines, size of boulders in fanglomerates, and freshness of feldspar in the Durham basin sediments all suggest short transport distances from a nearby source to a valley floor or graben, similar to a setting in the modern Basin and Range province. Paleocurrent data suggest a source to the northeast. (Reference 2.5.1-334)

A long history of investigations in the Deep River basin has led to a stratigraphic nomenclature that varies based on location. Stratigraphic units previously used in defining and mapping the Deep River basin were based on studies in the Sanford basin, and included the Sanford, Cumnock, and Pekin formations of the Chatham Group. Triassic sediments differ appreciably among the Durham, Sanford, and Wadesboro subbasins of the Deep River basin, and the stratigraphic divisions used in the Sanford subbasin are not applicable to the Durham basin (Reference 2.5.1-334). Bain and Harvey (Reference 2.5.1-386) proposed the first map units internal to the Durham basin, based on depositional facies. As noted by Bain and Harvey (Reference 2.5.1-386), lateral correlation of stratigraphic units in the Durham basin is complicated by the rapid facies changes and extensive faulting that has placed units of different ages in juxtaposition. Without age control from fossil or faunal information, correlations of units based strictly on lithologies may be erroneous. It has been shown that the traditional assumption that the lacustrine-paludal facies position near the middle of the basins also indicates middle position in a simple monoclinal half-graben model is erroneous. For example, part of the eastern part of the Sanford basin mapped as Sanford Formation is likely Pekin in age. (Reference 2.5.1-386) Later more detailed mapping by Hoffman and Gallagher (Reference 2.5.1-387) identified seven distinct lithofacies based on both lithology and depositional environment, which were then grouped into three lithofacies associations by Clark et al. (Reference 2.5.1-204). A correlation chart showing the relation of the formations of the Chatham Group in the Deep River basin to the lithofacies associations from Clark et al. (Reference 2.5.1-204) for the Durham basin is shown on Figure 2.5.1-261.

2.5.1.2.3.2.1 Formations of the Chatham Group

The sedimentary rocks of the Deep River basin previously were divided into three units, which from oldest to youngest are the Pekin, Cumnock, and Sanford formations. These formations were based on studies in the Sanford basin, where gray and black claystone, shale, siltstone, fine-grained sandstone, and two beds of coal of the Cumnock Formation are underlain by the Pekin Formation and overlain by the Sanford Formation. The Pekin and Sanford formations are dominated by fluvial and alluvial fan deposits; the Cumnock Formation is dominated by lacustrine and paludal deposits (Reference 2.5.1-204). The Pekin and Sanford formations are mostly red, brown, or purple siltstone; claystone; shale; sandstone; conglomerate; and fanglomerate. Because the Sanford Formation and underlying Pekin Formation are distinguished by the presence of the Cumnock Formation stratigraphically between the two, it is difficult to extend

the stratigraphy of these formations into the Durham basin, where the Cumnock formation is absent. The formations of the Chatham Group as described in the HNP FSAR (Reference 2.5.1-202) and previous studies are summarized below.

The Pekin Formation lies unconformably on pre-Triassic metamorphic and igneous rocks and crops out in a narrow belt 1.6 to 5 km (1 to 3 mi.) wide along the northwestern side of the Deep River basin to the northwest of the HAR site. The best exposures of this formation occur in the Wadesboro basin near the town of Pekin. The total thickness of exposed rocks in the Pekin Formation ranges from 550 to 1200 m (1800 to 4000 ft.). Practically all the rocks in this formation are red, brown, or purple. In general, they are medium- to fine-grained clastic rocks consisting of claystone, shale, siltstone, and sandstone, with a few beds of conglomerate and fanglomerate near the base of the formation. (Reference 2.5.1-202)

The Cumnock Formation lies conformably on the Pekin Formation and crops out in a narrow belt in the southern end of the Durham basin. It consists of gray and black claystone, shale, siltstone, fine-grained sandstone, and two beds of coal. The northernmost outcrop mapped by Reinemund is approximately 5 km (3 mi.) to the southwest of the plant site. (Reference 2.5.1-202)

The Sanford Formation lies conformably on the Cumnock Formation in the Sanford basin and parts of the Durham basin, but apparently lies unconformably on the Pekin Formation in the Colon cross structure, where the Cumnock Formation is missing. The Sanford Formation is more than 900 m (3000 ft.) thick in the Sanford basin and 600 to 900 m (2000 to 3000 ft.) thick in the southern edge of the Durham basin. It borders much of the southeastern edge of the Deep River basin and is composed of claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate, more than three-fourths of which are red, brown, or purple. In the Durham basin, rocks of the upper part of the Sanford Formation are exposed in a zone 1.6 to 3.2 km (1 to 2 mi.) wide along the eastern and southeastern side of the basin adjacent to the Jonesboro fault. Between the zone of conglomerates and fanglomerates and the zone of rocks of Pekin age along the western and northwestern side of the basin, fine-grained sediments of the lower part of the Sanford Formation, consisting of shale, siltstone, and sandstone, are present. The HAR site is located in this area of fine-grained sediments. The sediments of the Sanford Formation underlying the HAR site and much of the southeastern part of the Durham basin were deposited as alluvial fans and stream channel and floodplain deposits. These materials are characterized by abrupt changes in composition and texture, both horizontally and vertically. They contain few distinctive beds and subdivisions that are consistently mappable. The beds vary in thickness from less than an inch to a maximum of 4.5 to 6 m (15 to 20 ft.). As a result, exposures only about a meter (a few feet) apart may vary considerably in texture and composition. Notwithstanding these variations in composition and texture, the beds and lenses interfinger and overlap into compact masses. (Reference 2.5.1-202)

2.5.1.2.3.2.2 Lithofacies Associations in the Deep River Basin

Detailed stratigraphic and paleontologic studies were performed in the Deep River basin, leading to a new nomenclature by which Triassic sediments in the Durham basin were described as lithofacies rather than formations (Reference 2.5.1-204). Geologic units included in the lithofacies are being used in recent unpublished geologic quadrangle mapping by the North Carolina Geological Survey (NCGS) and are used in this report (Reference 2.5.1-275). Without further studies, the lithofacies associations of Clark et al., developed for the Durham basin (Figure 2.5.1-233), are not correlative to the Pekin, Sanford, or Cumnock formations that were described in the Sanford basin (Reference 2.5.1-204). Triassic sediments in the Durham basin have been divided into the three lithofacies associations described by Clark et al., which are represented on Figure 2.5.1-231 as unpublished geologic mapping by the NCGS and are depicted on Figure 2.5.1-233 (Reference 2.5.1-204). The lithofacies associations for Triassic sediments in the Deep River basin as described by Clark et al. and Hoffman are described in the following subsections (Reference 2.5.1-204, Reference 2.5.1-335).

2.5.1.2.3.2.2.1 Lithofacies Association I

Lithofacies Association I includes the unit Trcs/si1 (sandstone with interbedded siltstone). It consists of pinkish gray to light gray, fine- to medium-grained, micaceous arkoses and lithic arkoses; pale red, muddy, fine-grained sandstones; and reddish brown, bioturbated siltstones and mudstones. Fine-grained biotite and very fine-grained heavy minerals are distinctive to this lithofacies. The lithofacies is interpreted as originating from sandy, braided channel belts intercalated within thick sequences of heavily bioturbated siltstones, mudstones, and fine-grained sandstone lenses representing vegetated flood basin facies. Sandstone sequences occur as 1-m to more than 5-m- (3-ft. to more than 16.4-ft.-) thick cosets of trough cross beds that fine upward. Finer-grained siltstones and mudstones are bioturbated extensively and locally contain thin zones of carbonate nodules, zones of ferric concentration, and possible paleosols.

2.5.1.2.3.2.2.2 Lithofacies Association II

Lithofacies Association II consists of sandstone with interbedded siltstone (Trcs/si2) and siltstone with interbedded sandstone (Trcs/s). An arbitrary dividing point of 50 percent sandstone versus siltstone is used to differentiate these two units. These deposits are interpreted as originating from a meandering fluvial system flowing into a deltaic and lacustrine depositional environment. Trcs/si2, which underlies the HAR site (Figure 2.5.1-231, Sheet 1), consists of cyclical depositional sequences of whitish yellow to grayish pink to pale red, coarse- to very coarse-grained, trough cross-bedded lithic arkoses that fine upward through yellow to reddish brown siltstone that is burrowed and rooted (Figure 2.5.1-234). Exposures typically are highly weathered. Bioturbation usually is surrounded by greenish blue to gray reduction halos. Trcs/si2 represents lateral aggradation of a

point bar within a meandering stream surrounded by a vegetated floodplain. It contains abundant potassium feldspar and muscovite and is very coarse grained, suggesting a granitic source area. Trcsi/s consists of reddish brown, extensively bioturbated, muscovite-bearing siltstone interbedded with tan to brown, fine- to medium-grained, muscovite-bearing arkosic sandstone and is usually less than 1 m (3 ft.) thick. Siltstones contain abundant calcite nodules interpreted as caliche, which typically are surrounded by greenish blue to gray reduction halos. Trcsi/s represents fluvial deposits combined with locally developed, aerially limited, ephemeral, shallow freshwater lakes. This unit may represent a localized lowland away from the primary fluvial belt or a change to muddier conditions in later stages of deposition. The unit contains abundant vertebrate and invertebrate fossils, including plants, clams, crayfish, fish, reptile teeth, and abundant coprolites.

2.5.1.2.3.2.2.3 Lithofacies Association III

Lithofacies Association III consists of sandstone (Trcs), pebbly sandstone (Trcsc), sandstone with interbedded conglomerate (Trcs/c), and conglomerate (Trcc). In the southern Durham basin, the sandstone and pebbly sandstone are combined into Trcs. This lithofacies is interpreted as an alluvial fan complex. Interbedded sandstone and pebbly sandstone of Trcs consist of reddish brown to dark brown, irregularly bedded to massive, poorly to moderately sorted, medium- to coarse-grained, muddy lithic arkoses with occasional matrix-supported granules and pebbles. Trcs represents deposition in broad, shallow channels incised into muddy flats; the muddy, matrix-supported sandstones may represent distant debris flows or hyperconcentrated flows. Sandstone with interbedded conglomerate of unit Trcs/c consists of reddish brown to dark brown, irregularly bedded, poorly sorted, coarse-grained to pebbly, muddy lithic sandstones with interbedded pebble to cobble conglomerate. Trcs/c represents deposition in broad shallow channels by streams carrying high sediment concentrations or by debris flows. Conglomerates of unit Trcc consist of more than 50 percent reddish brown to dark brown, irregularly bedded, poorly sorted, cobble to boulder conglomerates. These conglomerates represent debris flow and channel deposits associated with alluvial fans developed along the Jonesboro fault scarp. Topography in Trcs/c and Trcc generally is steep and rugged, and in some cases the strikes of parallel ridges and first-order drainages parallel the strike of bedding.

2.5.1.2.3.2.3 Sedimentary Formations at the HAR Site

Sedimentary beds in the Triassic bedrock underlying the HNP and adjacent areas range in thickness from several centimeters (a few inches) to a maximum of about 6 m (20 ft.); the beds are commonly lenticular (Reference 2.5.1-202). Triassic sediments are covered by overburden that is partly residual and partly transported. In upland areas, the overburden consists of residual yellow, sandy clay and sandy loamy soil derived from the weathering of underlying rock. This soil is usually only 0.6 to 2 m (2 to 6 ft.) thick, but in areas of erosion, most of it has been removed by running water. In stream valleys and bottoms, yellow,

sandy, clayey alluvium often has accumulated to thicknesses of from 0.6 to 1.2 m (2 to 4 ft.) to as much as 3 to 3.7 m (10 to 12 ft.). Beneath this overburden, the Triassic sedimentary rocks usually are dense, compact, and only slightly weathered, with a variably developed, thin saprolite zone. In some areas, the Triassic rocks are overlain by younger sedimentary rocks. (Reference 2.5.1-202)

Bedrock at the HAR 2 and HAR 3 sites consists predominantly of reddish brown siltstone and sandstone, as discussed in detail in Subsection 2.5.4.1. Cross sections of rock stratigraphy at the HAR 2 site are shown on Figures 2.5.4-204A and 2.5.4-204B, and for the HAR 3 site on Figures 2.5.4-205A and 2.5.4-205B. Subordinate amounts of shale and conglomerate also were rarely encountered. Rock is typically sound and fresh below the nuclear island basemat grade (elevation 67.1 m [220 ft.]) at both HAR sites, although some isolated intervals of fractured rock and clay layers are present.

As a result of the mechanisms of deposition of the Triassic basin sediments, the continuity of individual rock strata at the site is variable both laterally and with depth. Fine-grained sandstone and siltstone strata commonly are traceable in boreholes across the extents of the HAR 2 and HAR 3 footprints. Other strata, such as coarse-grained sandstone deposits associated with stream deposition, exhibit more lateral variability. Thicknesses of individual strata or beds typically range from a few centimeters (a few inches) to several meters (tens of feet).

2.5.1.2.3.2.4 Stratigraphy in Sears No. 1 Well and Groce No. 1 Well

There are only two deep wells in the Deep River basin that can provide direct information on the deeper stratigraphy within the basin (Figure 2.5.1-230). Only one, the Chevron U.S.A., Inc., Groce No. 1 well, an exploratory oil well, extends through the entire Triassic section into the underlying basement. Basement was encountered in this well at a depth of 1550 m (5100 ft.) (Reference 2.5.1-334).

The second well, the Sears No. 1 well, was drilled in 1976 by the NCGS and USGS near New Hill, approximately 5.6 km (3.5 mi.) north-northeast of the site. This well, completed in Triassic rift sediments, was drilled to a depth of 1142 m (3746 ft.), with cuttings sampled every 1.5 m (5 ft.) (Reference 2.5.1-334). Triassic strata exposed in the borehole are, from bottom to top, a basal argillite-greywacke-conglomerate facies at least 122 m (400 ft.) thick; a 380-m- (2198-ft.-) thick sequence of massive mudstone, argillite, and quartz conglomerate facies; and a 640-m- (2100-ft.-) thick sequence of arkosic sandstone, siltstone, and mudstone (Reference 2.5.1-334). The lower 640 m (2100 ft.) of the Sears No. 1 and Groce No. 1 wells may be correlative to parts of the Pekin Formation, although neither the coal-bearing interval of the Cumnock Formation nor the Sanford Formation is seen in the Sears No. 1 well (Reference 2.5.1-334). Similarity in the Pekin Formation sediments found in the two wells suggests similar depositional environments in the Durham and Sanford areas during Pekin time. Following this time, a stable, swampy, reducing environment of low sediment volume existed in the Sanford area; the New Hill

area experienced higher energy that created channel sands and point bars in an alluvial fan setting (Reference 2.5.1-334).

2.5.1.2.3.3 Triassic-Jurassic Intrusive Rocks

Triassic sedimentary rocks in the Deep River basin were intruded by Triassic-Jurassic dikes, sills, and sill-like masses. The dikes range in width from centimeters (a fraction of an inch) to more than 90 m (300 ft.) and in length from a meter (a few feet) to more than 11 km (7 mi.). Sills and sill-like intrusives range in thickness from several centimeters (a few inches) to more than 60 m (200 ft.). These intrusives occupy about 4 percent of the total area of the Deep River basin. Sills and sill-like intrusives are almost completely confined to the Cumnock Formation in the Deep River Coal Field. Where the sills are most abundant, one-third to one-half of the Cumnock Formation is occupied by sills and sill-like intrusives that may reach a thickness of 120 m (400 ft.). The only exposed sill of any appreciable size known in the Sanford Formation crops out in western Lee County. No sills or sill-like masses have been identified in the southern part of the Durham basin. Diabase dikes generally follow northwest-southeast-trending joints and cross faults, along which displacements are less than 15 m (50 ft.), and are abundant in the Sanford basin and the Colon cross structure. Dikes in the Sanford basin trend from north 25 degrees to 40 degrees west, whereas those in the Colon cross structure and the southern end of the Durham basin trend from north 15 degrees to 20 degrees west or trend north-south. (Reference 2.5.1-202) Several north-south- to northwest-southeast-trending diabase dikes of Triassic-Jurassic age have intruded the Triassic bedrock in the site vicinity (Figure 2.5.1-230). Based on remnant magnetism, the dikes are 150 to 225 Ma; based on potassium-argon dating, the dikes are between 168 and 260 Ma (Reference 2.5.1-202). These dikes are nearly vertical and 0.3 – 5 m (1 – 15 ft.) thick. Bedrock adjacent to the dikes commonly is baked to a dark gray or black. Most dikes are deeply weathered to a mixture of clay and rounded cobbles of residual diabase. Depth of weathering in the dikes commonly is between 1.5 and 3 m (5 and 10 ft.); weathering tends to be shallower over adjacent sediments. The fault-gouge zone where dikes are displaced ranges in width from several centimeters (a few inches) to about 1 m (3 ft.). (Reference 2.5.1-202)

2.5.1.2.3.4 Post-Triassic-Jurassic Sedimentary Deposits

Cretaceous sediments within the site vicinity include sandstone and mudstone of the Middendorf and Cape Fear formations (Figure 2.5.1-230, Reference 2.5.1-336, Reference 2.5.1-208). Although these deposits generally are mapped locally south and east of the Jonesboro fault, along the southeastern margin of the Sanford basin, the Jonesboro fault is overlain and buried by undisturbed, relatively flat-lying Middendorf sediments. Isolated remnants of the Tertiary fossiliferous clays, sands, and limestones of the Yorktown, Duplin, and Castle Hayne formations, along with Tertiary terrace deposits and upland sediments, also are mapped south of the Jonesboro fault. An isolated remnant of upland gravels (Unit Tt) is observed in the vicinity of Apex, approximately 13 km

(8 mi.) northeast of the HAR site (Figure 2.5.1-230). Quaternary alluvium (Qal), including terrace and floodplain deposits, is seen along many modern drainages (Figure 2.5.1-231) (Reference 2.5.1-228).

2.5.1.2.3.5 Soils at the Site (1-km [0.6-mi.] Radius)

Detailed soils maps and studies have been performed for most counties in North Carolina. Additionally, the North Carolina Geological Survey has a generalized soils map for the entire state (Reference 2.5.1-337). Soils in the site area have been mapped in terms of both soils associations, which describe the distinctive, proportional pattern of soils in a landscape, and soils series, which describe the characteristics of a distinct soil type (Reference 2.5.1-338). Soil series are further divided into soil types and soil units, shown on Figure 2.5.1-235 for the area within 1 km (0.6 mi.) of the HAR site. The soils map predates construction of the HNP and, therefore, represents pre-construction topography and soil conditions.

The site lies within the Creedmoor-White Store association, which comprises gently sloping to hilly, deep and moderately deep, moderately well-drained soils that have very firm clayey subsoil and are derived from sandstone, shale, and mudstone (Reference 2.5.1-338, Figure 2.5.1-235). The association occurs as gently sloping soils on broad ridges and of hilly soils near drainageways in the uplands (Reference 2.5.1-338). Within 1 km (0.6 mi.) of the site, the soils are divided into the Augusta, Creedmoor, Enon, White Store, and Worsham series (Figure 2.5.1-235), as described in the following paragraphs.

The Augusta Series fine sandy loam (Au) occurs on slopes of 0 to 4 percent and on low terraces. The sandy loam has good infiltration, and surface runoff is slow to medium (Reference 2.5.1-338).

The Creedmoor Series silty loam and sandy loam, which cover much of the area within 1 km (0.6 mi.) of the site, are formed from Triassic sandstone, mudstone, and shale developed in heavily forested areas (Reference 2.5.1-338). Creedmoor sandy loam on slopes of 2 to 6 percent (CrB) occurs on broad, smooth, interstream divides (Reference 2.5.1-338). CrB has good infiltration, slow permeability, and medium runoff. Where eroded, the Creedmoor sandy loam on slopes of 2 to 6 percent (CrB2) is in many places a mixture of the remaining original surface layer and material from the subsoil. In less eroded areas, the surface layer is grayish brown to pale yellow sandy loam, but in more eroded areas the color can be strong brown and the texture clay loam. Unit CrB2 locally includes areas that are eroded down to the subsoil. (Reference 2.5.1-338) Creedmoor sandy loam on slopes of 6 to 10 percent (CrC) occurs on narrow side slopes in the uplands. This unit has good infiltration, slow permeability, rapid surface runoff, and severe erosion hazard. (Reference 2.5.1-338) Where eroded, this unit (CrC) is in many places a mixture of the original surface soil and the subsoil. In less eroded areas, the surface layer is grayish brown to pale yellow sandy loam, but in more eroded areas the color ranges to strong brown and the texture ranges to clay loam. (Reference 2.5.1-338) Creedmoor sandy loam on

slopes of 10 to 20 percent (CrE) occurs on narrow side slopes bordering upland drainageways, and is slightly to moderately eroded. This unit includes areas where the surface layer is a coarse sandy loam, a silt loam, or completely eroded. The unit has fair to good infiltration, slow permeability, and rapid runoff. (Reference 2.5.1-338) The Creedmoor silt loam on slopes of 2 to 6 percent (CtB) occurs on broad, smooth interstream divides in the uplands. It has good infiltration but slow permeability and medium runoff. (Reference 2.5.1-338) The Creedmoor silt loam on slopes of 6 to 10 percent (CtC) occurs on broad, smooth interstream divides in the uplands. Infiltration is good, permeability is slow, and surface runoff is rapid. (Reference 2.5.1-338)

The Enon fine sand loam occurs on slopes of 6 to 10 percent (EnC) on narrow side slopes in the uplands (Reference 2.5.1-338).

The White Store sandy loam on slopes of 2 to 6 percent (WsB) occurs on broad, smooth, interstream divides in the uplands (Reference 2.5.1-338). The White Store sandy loam on slopes of 6 to 10 percent (WsC) occurs on narrow side slopes in the uplands (Reference 2.5.1-338). Where the White Store sandy loam on slopes of 6 to 10 percent is eroded (WsC2), the surface layer commonly is a mixture of the remaining surface soil and the subsoil. Where the White Store sandy loam occurs on slopes of 10 to 20 percent (WsE), it is a slightly to moderately eroded soil on narrow side slopes bordering drainages (Reference 2.5.1-338).

The Worsham sandy loam (Wy) is found on slopes of 0 to 4 percent at the heads of drainageways, on foot slopes, and in slight depressions (Reference 2.5.1-338).

Included on the soils maps are units that do not describe soils. In the site area, these include gullied land (Gu), water (w), and open water (Wo). Gullied land is a miscellaneous land type consisting of areas that have eroded beyond feasible reclamation, where practically all of the surface soil and in places much of the subsoil have been removed by erosion. Some gullies have cut through the soil to the underlying rock. (Reference 2.5.1-338)

As described in Subsection 2.5.4.1, soils encountered at both the HAR 2 and HAR 3 sites consist primarily of silts, lean clays, and sand. Soils were formed in place by weathering of the parent siltstone and sandstone. The depth of soil over rock is 1.5 to 4.6 m (5 to 15 ft.) at HAR 2 and 3 to 7.6 m (10 to 25 ft.) at HAR 3. The soil profile is thinner at HAR 2 than at HAR 3 because surficial soil material at HAR 2 was removed to create the existing site grade during construction of the HNP. The ground surface at HAR 3 was not disturbed during construction of the HNP, and the soil profile is therefore thicker.

2.5.1.2.4 Structural Geology of Site Area

Pre-Mesozoic structures in the site vicinity include late Paleozoic fault zones (e.g., the Nutbush Creek fault) that crop out in the Piedmont, which are discussed in Subsection 2.5.1.1.4.2.2. These faults are characterized as late

Paleozoic ductile mylonite zones having dominantly right-lateral strike-slip displacement (Reference 2.5.1-251). The site area and much of the surrounding site vicinity lie within the Deep River basin and the most recent deformation in the site area (within an 8-km [5-mi.] radius) is primarily related to Mesozoic rifting (see Subsection 2.5.1.2.4.1). Minor faults exposed in the foundation exposures of the Main Dam occur in crystalline rock within approximately 1 km (0.6 mi.) of the Jonesboro fault, the main eastern boundary fault of the Mesozoic basin. These are described in Subsection 2.5.1.2.4.2. Evidence for local, small Cretaceous and post-Cretaceous faulting in the site vicinity beyond the site area is discussed in Subsection 2.5.1.2.4.3.

2.5.1.2.4.1 Mesozoic Structures

2.5.1.2.4.1.1 Deep River Basin Structures

The HAR site lies within the approximately 230-km- (144-mi.-) long Mesozoic Deep River basin (Figure 2.5.1-215), which traditionally is divided into three subbasins (from north to south): the Durham, Sanford, and Wadesboro basins (e.g., Reference 2.5.1-334, Reference 2.5.1-220). The Durham and Sanford basins are separated by a constriction and basement high called the Colon cross structure. Here the Deep River basin narrows appreciably (Figure 2.5.1-215). (Reference 2.5.1-220) The Colon cross structure is constrained in part by field mapping. Slightly different lithologies occur on either side of the Colon cross structure suggesting that the Colon structure may have acted as a barrier to sedimentation. (Reference 2.5.1-204) The basement high represented by the Colon cross structure likely formed from differential subsidence between the Durham and Sanford basins, but the details of the extent, as well as the structural components and evolution, of the Colon cross structure are not well known (Reference 2.5.1-204, Reference 2.5.1-254). Gravity and magnetic maps of the Durham basin provided by Bain and Brown (Reference 2.5.1-334) and Bain and Harvey (Reference 2.5.1-386), as discussed in Subsection 2.5.1.2.4.4, suggest that basement is shallower where the basin narrows. The HAR site lies in the transition between the southern end of the Durham basin and the northern end of the Colon cross structure.

Regional geologic mapping and fault investigations in the Durham basin north of the Colon cross structure conducted as part of a major study to characterize its waste storage potential provide additional data on the structural features in the Durham basin (Reference 2.5.1-334). In the Durham basin, northeast-trending faults and fracture zones are cross-faulted along north-northwest-tending faults and fracture zones to produce diamond- and triangular-shaped fault blocks in map view. The eastern border of the Durham basin is step-faulted and the basin is cut into a series of southeasterly rotated, postdepositional slices that trend parallel to the borders of the Sanford and Wadesboro basins (Reference 2.5.1-334).

The Sanford and Wadesboro basins are separated by the Pekin cross structure and Coastal Plain overlap (Figure 2.5.1-215; Reference 2.5.1-220). The Sanford

and Wadesboro subbasins have been rotated by faulting, forming horst and graben structures within the basin (Reference 2.5.1-334).

Two main sets of faults cut the Deep River basin. The major set strikes northeast (paralleling the Jonesboro fault) and includes faults both synthetic and antithetic to the Jonesboro that cut the basin into a series of largely postdepositional fault blocks that show differential subsidence (Reference 2.5.1-220). The other set of faults is roughly perpendicular to these, striking northwest. These faults are nearly vertical and are characterized by much smaller vertical displacements. Many of these northwest-striking faults have been intruded by diabase dikes. (Reference 2.5.1-220)

The most prominent faults of Mesozoic age mapped within the site vicinity (within a 40-km [25-mi.] radius) are the Jonesboro and Bonsal-Morrisville faults (Figure 2.5.1-230). Faults mapped in the site area (within an 8-km [5-mi.] radius) include the Jonesboro fault; the Harris fault (HF); the South Borrow Pit fault (SBPF); three smaller faults (faults FA, FB, and FC) at the borrow pits; the W8 and W82 faults; and two small, unnamed faults (Reference 2.5.1-275, Figure 2.5.1-230). Based on detailed studies that are discussed in the following sections, none of these faults is considered to be a capable tectonic source, as defined in Regulatory Guide 1.208, Appendix A (see discussion in Subsection 2.5.3.6).

2.5.1.2.4.1.1.1 Jonesboro Fault

The Jonesboro fault, part of which is located approximately 6.5 km (4 mi.) south of the site, is more than 160 km (100 mi.) long. It trends northeast-southwest, dips northwest, and forms the southeastern side of the Deep River basin (Figure 2.5.1-215). It marks the contact between Triassic sedimentary rocks to the west and Paleozoic volcaniclastic and crystalline rocks to the east (Figure 2.5.1-230). The Jonesboro fault and other associated faults probably were reactivated along older structural trends in the basement rock (Reference 2.5.1-202). Reactivation or initiation of tensional, normal-type movement along the Jonesboro fault was followed by deposition of Triassic sediments. The fault was last active between the intrusion of Late Triassic – Jurassic dikes and deposition of the overlying, unfaulted Cretaceous marine sediments (Reference 2.5.1-202). Southwest of the Main Dam, the fault is covered by unbroken Cretaceous sediments (Reference 2.5.1-202). The estimated total displacement along the Jonesboro fault is 3000 to 4500 m (9840 to 14,760 ft.) of dip-slip displacement (e.g., Reference 2.5.1-204); the amount of lateral displacement is unknown.

The Jonesboro fault can be divided into four segments, which are marked by abrupt, 30- to 60-degree changes in trend of the main fault surface: the Creedmoor, Durham, and Holly Springs segments, and the Sanford composite segment (Figure 2.5.1-215; Reference 2.5.1-339, Reference 2.5.1-254). Recurring displacement on the Holly Springs and Durham segments of the Jonesboro fault during the Triassic account for the development of most of the structural features in the site area (Reference 2.5.1-254). The approximately

55-km- (34-mi.-) long Durham segment extends from the abrupt (east/northeast to north/northeast) deflection point near Holly Springs to the abrupt (northeast to north/northwest) deflection point approximately 10.6 km (6.6 mi.) east of Creedmoor (Reference 2.5.1-339, Figure 2.5.1-215). Reported vertically aligned cobbles and clastic dikes and pillar structures in the Durham basin observed in road construction exposures that are no longer visible suggest that ground shaking and liquefaction accompanied some large paleoseismic events along the Jonesboro fault during deposition of Triassic sediments in the adjacent basin (Reference 2.5.1-339). In contrast, sheared cobbles and gouge zones indicate subsequent brittle fracturing of indurated rock at depth. Triassic movement on the central part of the Durham segment of the Jonesboro fault was primarily sinistral-oblique normal, as indicated by slip directions on nearby smaller faults. The overall direction of Triassic extension, inferred from these slip directions, was approximately east-west. (Reference 2.5.1-339)

As part of the HNP Unit 1 FSAR investigations, magnetic and reconnaissance surveys were conducted on diabase dikes and “cross-faults” occurring along the Jonesboro fault in an effort to constrain the timing of faulting on the Harris fault. None of the dikes mapped at these locations are continuous across the Jonesboro fault, indicating that the amount of offset between dikes varies from 400 to 130 m (1300 to 420 ft.). (Reference 2.5.1-202) The age of last movement on the Jonesboro fault is bracketed between the intrusion of Late Triassic – Jurassic dikes and the deposition of the overlying unfaulted Cretaceous marine sediments, between 180 and 135 Ma (Reference 2.5.3-202, Reference 2.5.1-275). Existing exposures of the Jonesboro fault are rare. During a period of abnormally low lake level in 1995, a normally submerged exposure of the fault was observed by Tyler Clark of the NCGS along the Shearon Harris reservoir shoreline approximately 3 km (2 mi.) southwest of where Buckhorn Creek enters the lake (Figure 2.5.1-231). At this location, very saprolitic, coarse-grained Triassic sediment was observed to be juxtaposed against a saprolitic, light-colored sandstone interpreted to be Buckhorn granite/granodiorite. The contact between the two units, the Jonesboro fault, was observed to be an approximately 0.5-m- (1.6-ft.-) wide zone of anatomosing fault surfaces and fault breccia. As observed at other locations along the Jonesboro fault, ductile deformation features overprinted by brittle deformation observed at this location suggest that the fault initiated at depth in the ductile realm and later experienced brittle deformation due to footwall uplift. A wave-cut scarp at the faulted contact was interpreted to be due to fault line erosion. (Reference 2.5.1-388)

Additionally, no geomorphic features indicative of Quaternary reactivation of faulting (e.g., scarps in post-Mesozoic deposits, offset geomorphic features, vegetation lineaments) were noted during field and aerial reconnaissance conducted for this study. Based on the above observations, the Jonesboro fault was judged not to be a capable fault (Reference 2.5.1-202).

2.5.1.2.4.1.1.2 Bonsal-Morrisville Fault

The Bonsal-Morrisville fault (Figure 2.5.1-230) is located approximately 7.4 km (4.6 mi.) northwest of the site in the Durham basin. The Bonsal-Morrisville fault is

approximately 26 km (16 mi.) long, trends approximately N37E, and has 600 to 1800 m (2000 to 6000 ft.) of vertical down-on-the-northwest displacement (Figure 2.5.1-241; Reference 2.5.3-214). Based on seismic data (Reference 2.5.1-334), the Bonsal-Morrisville fault makes the Durham basin a double half graben (Figure 2.5.1-240). Both the Jonesboro and the Bonsal-Morriville faults dip to the northwest. The Bonsal-Morrisville fault appears to die out as it heads north toward Raleigh, but to the south, it may go through the Colon cross structure to the Sanford basin to the south (Figures 2.5.1-215 and 2.5.1-230). South of the Colon cross structure in the Sanford basin, the Bonsal-Morrisville fault may become the Deep River fault, also shown to be a major basin – internal fault (Figure 2.5.1-230). The Deep River fault continues south, disappearing under the Cretaceous Coastal Plain sediments covering the Pekin cross structure, which separates the Sanford basin from the Wadesboro basin. Here the Deep River fault becomes the major bounding fault of the Wadesboro basin. (Reference 2.5.1-390)

2.5.1.2.4.1.1.3 Harris Fault (Site Fault)

Extensive geologic investigations were performed to assess surface faulting at the HNP site during the construction and licensing of the HNP facilities. The Harris fault, a minor cross-basin fault within the basin, also referred to as the “site fault” in the HNP FSAR and HNP FSER documents and in NUREG-1038, was discovered in the foundation of the HNP Waste Processing Building during excavation (Reference 2.5.3-201, Reference 2.5.3-202). Preconstruction site characterization activities, which included 3700 m (12,125 ft.) of trenching (at depths of 0.6 to 3.6 m [2 to 12 ft.]), numerous geologic borings to depths of 15 to 76 m (50 to 250 ft.), and approximately 1500 linear m (5000 linear ft.) of magnetic survey lines (Figure 2.5.3-201) failed to show evidence of this fault or other surface faulting (Reference 2.5.3-201). The two trenches appear to have intersected each other and the fault at a location along the margin of a gully. At this location the trenches do not appear to have been excavated deep enough to provide sufficient exposure of unweathered bedrock in which the fault would have been more easily identified. After the Harris fault was discovered during the waste building excavation, a comprehensive fault investigation program was completed by Ebasco Services, to evaluate the location, style of faulting, and age of the most recent movement. The results of this study are well documented in a report by Ebasco Services (Reference 2.5.3-202).

Detailed investigations conducted by Ebasco Services demonstrated that the Harris fault is a minor tensional normal fault whose last movement was prior to 150 Ma (Reference 2.5.3-202, Reference 2.5.3-201). Observations and conclusions from these studies regarding the location, style of faulting, and timing of deformation that demonstrate that the Harris fault is not a capable tectonic source are summarized in the following paragraphs.

After the Harris fault was discovered in the excavation for the HNP Waste Processing Building, it was traced some 2400 m (8000 ft.) east and west in a series of short trenches normal to the fault (Reference 2.5.3-201;

Figure 2.5.3-201). It was also exposed in excavations at the Auxiliary Dam (Reference 2.5.3-201). When exposed in sedimentary beds, the fault exhibits an approximately east-to-west strike and a southerly dip between 60 and 90 degrees; it always exhibits drag folding on the hanging wall and seldom exhibits any disturbance of bedding planes on the northern or “foot” wall (Reference 2.5.3-202). Bedding in the Triassic bedrock underlying the HNP site and adjacent areas strikes north 5 degrees to 15 degrees east and dips 9 to 17 degrees to the southeast (Reference 2.5.1-202).The fault-gouge zone varies from several centimeters (a few inches) to about 1 m (3 ft.) in width (Reference 2.5.3-201). The fault tends to become oversteepened in coarser-grained sandstones and is nearly vertical adjacent to diabase dikes offset by the fault. No additional faults were discovered in 1200 linear m (4000 linear ft.) of trenching. (Reference 2.5.3-202) Nine core borings were completed in sedimentary rocks on either side of the Harris fault to determine the vertical component of offset. Correlation of marker beds in the site area proved to be extremely difficult because sediment lithology changes radically over short lateral distances. Based on borehole data, vertical offset in Triassic sediments along the Harris fault in the vicinity of the HNP site is between 24 and 30 m (80 and 100 ft.). (Reference 2.5.3-201) Near-vertical diabase dikes proved to be the best references for estimating horizontal displacement. It was noted, however, that the offset of dikes shows only post-intrusive movement, not necessarily the total offset of the sediments. The horizontal offset of diabase dikes as exposed in the trenches ranges from 0.1 to 4 m (0.5 to 13 ft.); a large horizontal component of movement is precluded in that the fault changes strike about every 90 m (300 ft.). (Reference 2.5.3-201) Prominent joint sets at the site and their relation to the Harris fault are discussed later in this subsection.

A number of secondary minerals observed in the fault gouge at the intersections of the fault with diabase dikes were used for determining age relationships between the dikes and faulting (Reference 2.5.3-202). Zeolite mineral assemblages, including harmotome, heulandite, and laumontite, that are related to hydrothermal and “burial” metamorphic events, were observed only in association with the diabase dikes; furthermore, analyses of strontium isotope ratios ($^{87}Sr/^{86}Sr$) suggest that the diabase and zeolites are genetically related. Evidence for this association can be found in their occurrence only in the dike-fault intersections or as small veins or amygdule fillings in the dikes, and in their absence from the fault zone away from the dikes. (Reference 2.5.3-201) The minimum age of the zeolite minerals has been determined from their potassium-argon content as 35 Ma; however, these are spuriously low because zeolites tend to lose argon but not potassium. (Reference 2.5.3-202) The intact condition of some of the very brittle, delicate zeolites indicates that the zeolites were formed after faulting, suggesting that the last movement on the fault was more than 10 Ma, but most probably before the final cooling of the dike about 200 Ma or during a burial metamorphic event before 150 Ma (Reference 2.5.3-201).

One laumontite vein has been cataclastically deformed, as evidenced by shearing, and also shows mechanical disaggregation and rotation of laumontite

grains. This indicates that there was some movement on the fault after crystallization of at least some laumontite. Paleomagnetic dating shows that the diabase dikes underwent burial metamorphism about 20 million years after dike intrusion; therefore, the laumontite is likely associated with the original deuteric hydrothermal alteration of the dike shortly after its crystallization, whereas harmotome and heulandite were formed during the burial metamorphism some 20 million years later. Because the secondary minerals were emplaced prior to 150 Ma and have not been disturbed by subsequent faulting, the last movement on the fault was prior to that time. (Reference 2.5.3-202)

Soils were also used to help constrain the timing of faulting on the Harris fault. Based on trench exposures and outcrops, the fault has not moved during formation of existing soil and saprolite on Triassic sedimentary rocks. Below the uppermost soil horizon, the material is classified as saprolite, and weathering decreases with depth from 0.6 to 4.6 m (2 to 15 ft.). Clay mineralogy studies show this to be an in-place residual weathering profile. Although soils in this area have been variously estimated to be as old as Miocene, more rapid erosion has prevented the HNP site soils from developing a strong profile. (Reference 2.5.3-201) Based on analogy with depths of oxidation at a similar location, the formation of the saprolite may have started more than several million years ago (Reference 2.5.3-202), and the diabase has not been disturbed by movement on the fault for more than 500,000 years (Reference 2.5.3-201).

Based on a search of current literature and on discussions with researchers in the area, no more recent work on the Harris fault has been completed. Given the detailed level of investigation by Ebasco Services (Reference 2.5.3-202) that demonstrated that the fault is not a capable tectonic source, it was judged that no further work was needed to characterize the Harris fault.

2.5.1.2.4.1.1.4 Borrow Pit Faults

Detailed geologic mapping at scales of 1:12 to 1:3750 that was performed at both the north and south borrow pits to the west and southwest of the HNP site revealed four faults: the SBPF and faults A, B, and C (FA, FB, and FC). The Harris fault was exposed north of the north borrow pit (Figure 2.5.3-201). The borrow pits are partially located within the area of the proposed Wake/Chatham LLRW disposal facility site (Figure 2.5.3-201). (Reference 2.5.3-205) Different structural patterns were seen in the north and south borrow pits. Faulting and fault-related folding in the south borrow pit were inferred from changes in the orientation of bedding (Reference 2.5.3-205).

Faulting exposed in the borrow pits can be divided into two stages: Stage 1 faulting occurred on FA and is interpreted to be associated with longitudinal normal faulting and tilting of beds within the Triassic Deep River basin, antithetic to the Jonesboro fault; Stage 2 faulting represents a later stage of transverse faulting along the SBPF, involving both normal and dextral strike-slip components that produced map-scale folding in the hanging wall of the SBPF. Stage 2 also involved reactivation of FA and formation of east- and north-striking shear mode

fractures. Based on exposures in the borrow pits, total displacement cannot be determined on any of the faults. (Reference 2.5.3-205)

The SBPF strikes north 72 degrees west, dips 30 to 70 degrees south, has a minimum trace length of approximately 430 m (1400 ft.) based on known exposures, and exhibits a right-stepping pattern across the borrow pit (Reference 2.5.1-226). In the westernmost part of the south borrow pit, the strike of the SBPF ranges from north 80 degrees east to north 50 degrees west over 60 m (200 ft.) along strike, and dip ranges from 40 degrees to vertical. The SBPF is characterized by breccia, foliated breccia, and clay gouge (Reference 2.5.1-226). Map units in the hanging wall of the SBPF form an outcrop-scale syncline having an east/southeast-plunging axis parallel to the SBPF (Reference 2.5.1-254). The geometry of the syncline suggests it was formed by drag related to normal or right-lateral strike-slip movement along the SBPF (Reference 2.5.1-254).

Three smaller faults (FA, FB, and FC) also were mapped in the south borrow pit (Figure 2.5.1-232). FA is a northeast-striking, near-vertical to steeply east-dipping normal fault that terminates at its southern end against the SBPF. It is characterized by partly to completely decomposed, foliated breccia; random-fabric breccia; and contorted beds. The zone of contorted bedding and brecciated rock is 1 – 3 m (3 – 10 ft.) wide and can be traced approximately 35 m (110 ft.) along strike. The strike of the foliation and compositional layering within the fault zone generally is oblique to the strike of the fault, but drag-folding along the fault indicates a component of normal displacement. (Reference 2.5.1-226) Initial movement along FA is interpreted as normal displacement antithetic to the Jonesboro fault and as predating movement on the SBPF (Reference 2.5.1-226). Kinematic indicators suggest a right-lateral component of movement on FA, similar to the W8 fault (Reference 2.5.1-226). FB is a normal fault that strikes north 20 degrees to 25 degrees west, dips 85 to 90 degrees west, and can be traced approximately 24 m (80 ft.) along strike. The fault is characterized by less than 20 cm (8 in.) of foliated breccia and breccia, intense fracturing, and clay. FC generally strikes north 35 degrees east, dips 60 degrees south, and is characterized by a 25- to 33-cm- (10- to 13-in.-) wide foliated breccia. Folds in the hanging wall may indicate compressional, left-lateral, and possibly reverse components of movement on FC. (Reference 2.5.1-226)

2.5.1.2.4.1.1.5 Faults Exposed at the LLRW Disposal Facility Site

Chem-Nuclear Systems, Inc., conducted geologic and hydrogeologic investigations at the proposed LLRW disposal facility site, 300 to 450 m (1000 to 1500 ft.) west of the borrow pits (Reference 2.5.3-204). These investigations involved four trenches (GM-1 through GM-4) (Figure 2.5.3-201), totaling approximately 1219 m (4000 ft.) in length, and nine boreholes along the line of the GM-1 trench (Reference 2.5.3-204). Seismic reflection/refraction and vertical seismic profiling were performed at the site to provide correlation between boreholes, evaluate faults, and determine the thickness of weathered bedrock (Reference 2.5.3-204).

Three primary faults were identified at the LLRW disposal facility site: the W8 fault, the W82 fault, and the western extension of the SBPF (Figure 2.5.1-232). The east-southeast-/west-northwest-trending SBPF and the W82 fault were mapped approximately 760 and 975 m (2500 and 3200 ft.) south of the Harris fault, respectively (Figures 2.5.1-231 and 2.5.1-232). The W8 fault strikes north-northeast and dips approximately 55 degrees west (Reference 2.5.1-332). The W82 fault strikes east-northeast, subparallel to the Harris fault, and dips toward the south (Reference 2.5.1-226). Identification of these faults was based on three primary stratigraphic marker units: conglomeratic sandstone, sandstone, and purple sandstone and mudstone, all of which could be traced over tens of meters (hundreds of feet). Bedding dips generally from 15 to 20 degrees east (Reference 2.5.1-332). Examination of the strata adjacent to the W8 fault, regional relations, and seismic reflection data suggest that the W8 fault and other nearby, smaller north-south-trending faults are listric faults at depth that are displaced by later movement on east-west-trending faults such as the SBFP; these north-trending faults are therefore judged to have been last active in the Triassic (Reference 2.5.1-254).

Borehole W205CH1 intersected the W8 fault at a depth of approximately 100 m (330 ft.), indicating a dip of approximately 55 degrees. Borehole cores showed that the fault is expressed as a zone of numerous fractures extending approximately 3 m (10 ft.) above and approximately 9 m (30 ft.) below the fault. (Reference 2.5.1-332) Drag along the fault is revealed as a change in the orientation of bedding from easterly to westerly across the fault within the fault zone. Bedding dips increase from approximately 20 degrees east to the west of the W8 fault to approximately 40 degrees east approaching the fault (Reference 2.5.1-332). Because stratigraphy differs on either side of the fault, displacement could not be measured directly; therefore, the fault is assumed to have more than 275 m (900 ft.) of vertical displacement (Reference 2.5.1-332). Trench mapping that extended approximately 600 m (2000 ft.) east and west of the W8 fault revealed no other faults of this size (Reference 2.5.1-332). Approximately 20 smaller, low-angle, bedding-parallel faults, exhibiting little or no displacement, were observed in the trenches (Reference 2.5.1-332).

Figure 2.5.1-236 presents schematic block diagrams showing a conceptual structural model of the faults discussed previously. Wooten et al. discuss the structural development of these faults, relating them to the dominant directions of displacement on the Durham and Holly Springs segments of the Jonesboro fault (Figure 2.5.1-236; Reference 2.5.1-254). Wooten et al. note that although the dominant displacement on all three east-southeast-/west-northwest-trending faults was normal (down to the south or southeast), the steep dips (70 to 90 degrees) along parts of the fault suggest that the faults originated as strike-slip faults, accommodating displacement at the southern end of the Durham segment of the Jonesboro fault (Reference 2.5.1-254).

A comparison was made between topographic lows and faulting at the LLRW disposal facility site. Many of the topographic lows appear to be controlled by stratigraphy rather than by structure. Saddles and strike-parallel drainages tend to be underlain by fine-grained units such as mudstones, and ridges tend to be

associated with sandstone units (Reference 2.5.3-204). On a more localized scale, isolated zones of deep weathering and minor topographic lows tend to correlate with small faults or zones of increased fracturing. Where linear drainages were associated with faults, the drainages were located slightly south of the actual fault in the hanging wall block, probably due to the presence of more fractured and deformed rock compared to the relatively undeformed rock on the footwall block. (Reference 2.5.3-204) Hydrologic tests also showed that groundwater was localized within the fractured sediments adjacent to the faults, similar in some cases to the localization of groundwater along the dikes (Reference 2.5.3-204).

2.5.1.2.4.1.1.6 Other Small Intrabasinal Faults in the Site Area

Several other faults and folds have been mapped within the site area (Figures 2.5.1-230 and 2.5.1-231). The most prominent is the Bonsal-Morrisville fault, which is discussed above. A smaller, approximately N70E trending fault that crosses the Bonsal-Morrisville fault was shown on the 1985 geologic map and is retained on more recent, more detailed unpublished mapping by the NCGS (Reference 2.5.3-208). Although more recent studies could not identify the fault in the field, it was retained in more recent mapping because high-angle bedding measurements, "different"-looking strata than the rest of the basin (possibly Lithofacies Association I), and abundant fractures were seen in the area of the mapped fault. Additionally, seismic reflection studies by Bain and Harvey (Reference 2.5.1-386) indicated anomalous seismic reflectors in that area.

A north-to-northwest-trending fault that was shown on the 1985 geologic map of North Carolina within approximately 185 m (600 ft.) northeast of the HAR 3 site (Reference 2.5.3-207) has been removed on more recent unpublished maps by the NCGS. Geophysical data and field reconnaissance show no indication of a fault in this location, but rather identify the presence of a diabase dike.

Two short faults were recognized during excavation along U.S. Highway 1. The easternmost of these two faults, which lies approximately 7.2 km (4.5 mi.) northeast of the HAR site (Figure 2.5.1-231) is an approximately 460–m- [1500-ft.-] long, high-angle normal fault (Reference 2.5.3-208). The fault displays drag in Triassic sediments and is located at the northwest end of the Holly Springs anticline, part of a zone of syndepositional, fault-bend folding in the hanging wall of the Jonesboro fault (Figure 2.5.1-231). The westernmost of these two faults is a northeast-trending (approximately 630–m- [2067-ft.-] long) fault that lies approximately 2 km (1.2 mi.) northwest of the HAR site (Figure 2.5.1-231) and was identified during the widening of U.S. Highway 1 in May 1997 (Figure 2.5.1-231; Reference 2.5.3-208). The exposure was in a storm water drain excavation that is now under the highway. The excavation showed a fault zone of disrupted Triassic sediments in which at least four slickensided fault surfaces all striking northeast-southwest, dipping moderately steeply to the northwest were mapped. The fault zone could not be traced beyond the extent of the excavation. However, based on similar observations at the LLRW site to the

south, NCGS staff felt the fault zone could be significant, and speculated that it could be a northward extension of the W8 fault. (Reference 2.5.1-389)

No evidence of post-Triassic movement on any of these structures is reported in the literature or has been documented by recent mapping by the NCGS. The faults are interpreted to have formed in the same tectonic stress field as the other better-studied Mesozoic rift faults in the site area (e.g., the Jonesboro fault, the Harris fault, and the faults in the LLRW and borrow pit study areas). Based on the absence of evidence for post-Triassic deformation and the structural association with noncapable faults, these faults are judged not to be capable tectonic fault sources.

2.5.1.2.4.1.1.7 Fractures and Joints

Three prominent joint sets were observed at the HNP site; the two dominant sets are vertical. One set strikes north 40 to 50 degrees east; the other strikes north 20 to 30 degrees west. The third joint set trends north-northwest/south-southeast and dips 55 to 70 degrees to the southwest (Reference 2.5.1-202). Joints are irregularly spaced every few feet and are mostly vertical (Reference 2.5.1-202).

High-angle joints and fractures were observed in oriented acoustic soundings in boreholes BPA-5, BPA-47, and BPA-48 at HAR 2, and boreholes BPA-25, BPA-49, and BPA-50 at HAR 3, as discussed in Subsection 2.5.4.4.2.2. These features predominantly dip between 60 to 70 degrees to the west to northwest. Isolated high-angle joints are also oriented in other directions.

At the Auxiliary Dam (Figure 2.5.1-232), two dominant, high-angle fracture sets are seen in muddy, sandy conglomerate and muddy conglomeratic sandstone beds. The first strikes east-northeast and dips 75 to 90 degrees south; the second strikes south-southeast and dips 60 to 90 degrees west (Reference 2.5.1-254). The east-northeast-striking set is parallel to the Harris fault and is interpreted to represent extensional fractures developed during normal, dip-slip movement on the Harris fault. Locally, pinnate fractures show evidence for a component of left-lateral movement on the fault (Reference 2.5.1-254). South-southeast-striking fractures likely predate intrusion of dikes and are related to generally east-west extension along the Jonesboro fault and related structures (Reference 2.5.1-254). Termination relationships among the fracture sets indicate that the east-northeast-striking fractures are younger than the south-southeast-striking fractures (Reference 2.5.1-254).

Several dominant fracture orientations were seen in the north borrow pit. The oldest set of joints trend north-northwest-south-southeast, are bleached, and terminate against bedding. East-northeast-/west-southwest-trending, bleached joints terminate against the oldest north/northwest-south/southeast set; younger east-west-trending joints abut both of the older sets. The east-striking set is subparallel to and genetically related to the SBPF. The other set, which strikes generally north, is interpreted to have formed prior to movement on the SBPF. Fracture spacing increases with distance from the SBPF; close to the SBPF, fractures are 10 to 30 cm (4 to 12 in.) apart. (Reference 2.5.1-226)

East-northeast-striking fractures exposed in the southern end of the south borrow pit are interpreted to be related to the W82 fault to the south (Reference 2.5.1-254). Older joints typically are 1 to 3 m (3 to 10 ft.) long; younger joints are less than 0.3 m (1 ft.) long (Reference 2.5.1-254). East-southeast-striking, nearly vertical fractures are dominant in the south borrow pit, especially in the hanging wall near the SBPF. These fractures have been interpreted to reflect extension orthogonal to the SBPF. (Reference 2.5.1-254) Bedding-parallel fractures that occur throughout the south borrow pit likely are related to east-west extension and basin-parallel faulting (Reference 2.5.1-254).

Two dominant fracture sets were observed during trenching, drilling (using electrical and optical imaging techniques), and mapping studies at the LLRW disposal facility site. One set is parallel to the bedding, strikes roughly north, and dips to the east. The other fracture set strikes generally north and dips steeply west, is orthogonal to the first, and is not bedding parallel. (Reference 2.5.1-332; Figure 2.5.1-262). High-angle fractures are most abundant in the hanging wall of the W8 fault, and fracture density also may increase near minor faults (Reference 2.5.1-332). The bedding-parallel fractures tend to occur at the boundaries of units that have significantly different mechanical properties, such as coarse-grained sandstone in contact with mudstone. Many of these boundary areas have accommodated slip during ancient faulting and tilting episodes (Reference 2.5.1-332).

2.5.1.2.4.1.2 Mesozoic Brittle Structures East of the Jonesboro Fault

Map-scale brittle faults and linear cataclastic zones that overprint late Paleozoic structures and penetrative fabrics are recognized in the site vicinity (Reference 2.5.1-385). Both north-south- and east-west-trending structures are observed. West-northwest-/east-southeast-trending faults east of the Jonesboro fault south of Raleigh (Figure 2.5.1-230) are brittle faults that range up to 13 km (8 mi.) in length and dip moderately (50 to 60 degrees) to the north or south. These faults commonly are characterized by quartz breccia zones and offset discrete shear zones and fabrics associated with the 312 to 282 Ma Nutbush Creek fault zone (Figure 2.5.1-230). The largest structure, which is mapped for over 13 km (8 mi.), is named the Swift Creek fault after the drainage that lies subparallel to the fault zone. Displacement on the faults appear to be predominantly dip-slip. The orientations of the different fault sets suggest that west-northwest/east-southeast extension and north-south extension were components of post-Alleghanian deformation along the western edge of the eastern Piedmont. (Reference 2.5.1-385) This deformation likely was associated with extension and opening of Mesozoic basins. The faults and cataclastic zones are most commonly manifested as ridge-top float of vuggy rich quartz or breccia. Topographic expression is generally subtle to nonexistent; locally the zones are roughly parallel to steep, discontinuous bluffs having up to 25 m (82 ft.) of relief. Similarly, northwest-southeast-trending folds located west of the bend in the Jonesboro fault are either syndepositional fault-bend folds or were formed from differential movement along the Jonesboro fault.

2.5.1.2.4.2 Structures in the Vicinity of the Main Dam

Twenty-four minor faults having lengths measured in meters to tens of meters (tens of feet) and displacements measured in centimeters (inches), all judged to be noncapable, were mapped in pre-Triassic crystalline rocks in the foundation of the Main Dam (Reference 2.5.1-202, Reference 2.5.1-331). The Main Dam is located approximately 900 m (3000 ft.) southeast of the Jonesboro fault in an area of Late Proterozoic to Cambrian igneous and metamorphic rocks consisting of granite, hornblende-mica gneiss, layered quartz-feldspar gneiss, and mica schist (Reference 2.5.1-331; Figure 2.5.1-230).

The dominant joint set in the crystalline rocks at the Main Dam strikes approximately north 60 degrees to 70 degrees eastward and dips 50 to 70 degrees southeastward. Another set strikes north 20 degrees to 35 degrees west and dips 70 to 90 degrees southwest. (Reference 2.5.1-331) The foliation in this area generally strikes approximately north 55 degrees east and dips 60 degrees northwest. Joints typically are spaced approximately 1 m (2 – 3 ft.) apart and strike northeast and northwest (Reference 2.5.1-331). Folding in the site area is most common in gneisses and schists exposed in the foundation of the Main Dam. These rocks appear to have undergone several periods of folding, predominantly isoclinal folding (Reference 2.5.1-331). The magnitude of this folding could not be determined because of the limited area of exposure of the crystalline rocks.

Most of the faults have strikes between N30°E and N75°E and dip steeply to the southeast. A few strike northwest and have steep northeast or nearly vertical dips. A majority of the faults exhibit right-lateral strike separation, but those showing left-lateral separation are also common. (Reference 2.5.3-201) It was documented that the small amount of movement along these faults is typical of deformation features in metamorphic rocks and took place prior to deformation and mineralization, associated with intrusion of granitic plutons throughout the Piedmont, that occurred more than 225 Ma. Because the small amount of movement along these faults took place prior to deformation and mineralization that occurred more than 225 Ma, the faults are not considered to be capable faults. (Reference 2.5.3-201, Reference 2.5.3-203) The NRC concurred that any movement along the faults occurred prior to or during deformation-mineralization processes which terminated more than 225 million years ago, and that the faults are not capable (Reference 2.5.3-203).

2.5.1.2.4.3 Cretaceous and Cenozoic Structures

Most faults in the site vicinity are pre-Cretaceous, but there are reports of minor localized faults that cut Upper Cretaceous to Miocene (?) and Pliocene (?) sediments at five localities (Features 41, 47, 49, 57, and 150 [Table 2.5.1-201]). None of these features could be confirmed by field reconnaissance conducted for this study, likely because they were exposed in road cuts or exposures that no longer exist or they have since been obscured by vegetation, or imprecision of the reported location coordinates did not provide sufficient information to locate the specific features. The best-documented post-Cretaceous fault, which is

reported to displace Pliocene/Pleistocene terrace deposits in a formerly exposed railroad cut near the town of Banks (approximately 23 km [14 mi.] east of the HAR site), is described by Parker (Reference 2.5.1-228; Feature 150 in Table 2.5.1-201 and Figure 2.5.1-230;. At this location, which is now totally obscured by vegetation and slopewash, Parker observed a southward-dipping, low-angle thrust fault that exhibited approximately 2.5 m (8 ft.) of dip-slip displacement in pebbly sand terrace deposits of probable Pliocene or Pleistocene age (Reference 2.5.1-228; Feature 150, Figure 2.5.1-230)

With the exception of the fault identified by Parker (Reference 2.5.1-228), the reported displacement on the other faults identified in the site vicinity is less than 1 m (3 ft.) in deposits of Cretaceous to Tertiary age (Reference 2.5.1-256). None of the Cretaceous or post-Cretaceous faults identified in the site vicinity have been mapped beyond the outcrop scale. Based on the limited extent of these features combined with the apparent absence of geomorphic evidence of Quaternary surface deformation noted during the aerial reconnaissance and from examination of hillshade relief maps derived from the LIDAR data, these faults are not considered to be capable tectonic sources.

2.5.1.2.4.4 Geophysical Studies of Structural Features

Several geophysical studies that have been performed in the site vicinity, including gravity, magnetic and aeromagnetic, seismic refraction and reflection, and resistivity studies, provide information on the location and character of structural features. These studies include surveys that were conducted as part of site characterization for the HNP; regional studies to evaluate the Durham basin for waste disposal; exploratory industry investigations for oil and gas; and studies conducted for the HAR sites (Reference 2.5.1-202, Reference 2.5.1-334, Reference 2.5.1-340). Gravity and magnetic anomaly maps of the site vicinity and surrounding region based on recent regional data compilations are presented on Figures 2.5.1-237 and 2.5.1-238, respectively.

2.5.1.2.4.4.1 Site Area Gravity Data

A gravity study by Mann and Zablocki to define geologic structures in the Deep River basin is described in the HNP FSAR as follows (Reference 2.5.1-321, Reference 2.5.1-202):

> The gravity measurements were made over the Deep River basin to establish (1) locations of concealed structures within the eastern part of the Triassic basin, (2) locations of the northwestern and southeastern borders of the basin where they are covered by a post-Triassic overlap, and (3) the southeastern extent of the Sanford basin. Measurements were taken with a Worden Gravimeter at 1,200 stations spaced at about 1-mile intervals. These measurements were relative to the gravity base station at Chapel Hill established by Woollard and Mann in 1956.
>
> Assuming a specific gravity of 2.67, Bouguer anomalies were computed with an isomilligal interval of 5 milligals (mGals). A residual anomaly map

having a contour interval of 5 mGals also was made in an attempt to distinguish deep-seated features from surface and near-surface geologic effects.

Two gravity highs border the Deep River basin, one on the northwestern side of the Durham basin, the other on the southeastern side of the Wadesboro basin. Relative gravity lows exist in the Wadesboro, Sanford, and Durham basins. The only negative gravity areas are located in the southeastern part of the Durham basin and east of the Sanford basin. The gravity lows and negative areas were attributed chiefly to especially great thicknesses of Triassic rocks.

Observations and conclusions cited in the HNP FSAR are summarized as follows. Gravity studies of the Deep River basin conducted prior to the HNP FSAR revealed neither the Jonesboro nor the Harris fault, likely because of variations in the basement and near-surface lithology surrounding the basin. The Colon cross structure, the constriction between the Durham and Sanford basins, is a northwest-southeast-trending anticlinal warp. Because of this feature, the isogals surrounding this area have a saddle-like appearance. The isogals of the Bouguer and residual anomaly gravity maps cannot be correlated satisfactorily with either the outline of the basin or the entire trace of the Jonesboro fault. The Harris fault is a minor feature, also not reflected in these records. It is clear from an examination of the Bouguer map that the deepest parts of the eastern Triassic basin are located in the northwestern and southeastern parts of the Sanford basin and in the southern part of the Wadesboro basin. Although the geologic features are not defined precisely on the gravity maps, the gravity profiles clearly show the basement configuration of the Deep River-Wadesboro basin. Model calculations suggest a deep accumulation of Triassic sediments and also show the Jonesboro fault as a steeply dipping feature on the southeastern side of the basin. Longitudinal faults, cross-faults, and diabase dikes can be correlated with finger-like deviations of the isogals in the Sanford basin. The negative-anomaly area along the eastern border of the Durham basin south of Raleigh probably is associated with the large granite body in that area. The negative anomaly in the center of the Sanford basin is correlated with a network of longitudinal faults and cross-faults. These faults may have caused the basement in this area to subside more than in other parts of the Deep River basin. The station spacing was not detailed enough to locate dikes and/or faults in the site area. Recontouring and computer modeling of available gravity data revealed no information unfavorable to the plant site.

Parker makes the following observations based on gravity data of Mann, Mann and Zablocki, and the Transcontinental Geophysical Survey (Reference 2.5.1-228, Reference 2.5.1-341, Reference 2.5.1-321, Reference 2.5.1-342). The gravity contours in Wake County and vicinity generally trend northeast-southwest, conforming to the strike of the rocks. A gravity low in eastern Wake County correlates with the Rolesville batholith. The Jonesboro

fault is not apparent on the Bouguer anomaly map of Mann, but is expressed as an abrupt drop in the contours on the more detailed map of Mann and Zablocki (Reference 2.5.1-341, Reference 2.5.1-321). An east-west anomaly that crosses the metamorphic belts southwest of Raleigh into the Durham basin likely represents transverse faulting during the early stages of deposition in the Triassic basin.

Gravity data presented by Bain and Brown show variations in the depth to basement rocks and in delineation of subbasins within the Deep River basin (Figure 2.5.1-263; Reference 2.5.1-334). A gravity low near Apex is separated from the northern part of the basin by a northeast-southwest-trending fault zone in the basement that extends from approximately 3 km (1.8 mi.) north of Moncure to the area around Morrisville (Reference 2.5.1-334). This fault, referred to as the Bonsal-Morrisville fault, is also seen in magnetic, resistivity, and geologic mapping studies (Figure 2.5.1-230).

2.5.1.2.4.4.2 Magnetic Data — Site Vicinity (40-km [25-mi.] Radius)

Parker provides a detailed discussion of features observed within magnetic data for Wake County (Reference 2.5.1-228). After three northeast-southwest-trending subareas of strongly contrasting magnetic contour patterns are delimited, details within each of the subareas are described. A middle lobate area that traverses the county is characterized by strong magnetic relief having values ranging from above +100 to below -300 gammas. Much of this central belt, which is characterized by closely spaced linear ridges and trenches, corresponds generally to the belt of metamorphic rocks. The eastern part, which is underlain primarily but not totally by granite, is characterized by mostly negative values (-100 to -200 gammas), low magnetic contrast, and irregular patterns except for several prominent linear ridges marking the location of diabase dikes that extend north-northwestward. The western area shows low magnetic values (mostly -200 to -300 gammas) and widely spaced, irregular contours that in many places trend east-west. This region corresponds to the Triassic sedimentary rocks in part of the Durham basin. In places, the western edge of the central magnetic belt differs greatly from the low negative values of the western region, whereas elsewhere there is little contrast across the boundary. The Jonesboro fault, which marks this boundary, juxtaposes rocks of strongly contrasting character along some but not all segments. (Reference 2.5.1-228) There is no strong contrast across the fault in the area east and southeast of Apex, close to the HAR site. Parker attributes the lack of a strong contrast to (1) increasing depth of rock masses that are responsible for the magnetic ridges to the east and (2) the inclusion of sediment derived from the magnetic rock belt to the east into the Triassic basin sediments (Reference 2.5.1-228).

Aeromagnetic data also were reviewed as part of detailed investigations of the Durham basin to evaluate the suitability of Triassic basins for waste disposal (Reference 2.5.1-334). Aeromagnetic studies suggest the Durham basin contains alternating horsts and graben (marked by alternating higher and lower magnetic grain) that are aligned in a north 40-degree east direction and are cross-faulted, intruded by diabase dikes, rotated primarily to the southeast, and stepped up

toward the eastern boundary fault (Reference 2.5.1-334). The magnetic map of Bain and Brown was useful in identifying dikes and patterns of structural deformation (Reference 2.5.1-334). The magnetic patterns suggest shallow basement near resistivity profile B-B′ south of Sanford, as well as along the Jonesboro fault near Holly Springs and Apex and between Morrisville and the Raleigh Durham Airport (Figure 2.5.1-239, Reference 2.5.1-334). Magnetic data also indicate that the western part of the Durham basin, north of Chapel Hill, is underlain by rocks of higher magnetic intensity or by diabase sills. Additionally, the magnetic data clearly delineate an intrabasin fault, the Bonsal-Morrisville fault, which is also apparent from gravity profiles. (Reference 2.5.1-334)

Previously compiled aeromagnetic studies were examined as part of site characterization studies for the HNP. The examination revealed northwest-southeast-trending lineaments near the site, representing diabase dike swarms in this otherwise magnetically quiet area (Reference 2.5.1-202).

2.5.1.2.4.4.3 Seismic and Resistivity Data — Site Vicinity (40-km [25-mi.] Radius)

Seismic reflection and refraction studies were performed in the Durham basin by the University of North Carolina and the U.S. Geological Survey as part of the study to determine the suitability of Triassic basins for waste disposal (Reference 2.5.1-334). Seismic velocities of Triassic sediments from two lines surveyed for the waste disposal study ranged from 3400 to 4200 m/sec (11,152 – 13,776 fps) (Reference 2.5.1-334). The depth to basement along the upthrown side of the Bonsal-Morrisville fault was estimated at 610 to 1200 m (2001 – 3937 ft.); it was estimated to be 1800 m (5905 ft.) on the downthrown side. Resistivity studies (discussed below), however, suggested that the depth to basement along the eastern side of the Durham basin is approximately 500 m (1640 ft.) on the upthrown side and 1100 m (3608 ft.) on the downthrown side, and that maximum depth to basement along the east side of the basin along the Jonesboro fault is approximately 2100 m (approximately 6890 ft.) near Apex. (Reference 2.5.1-334)

Electrical resistivity studies were performed in two parts of the basin (Figure 2.5.1-239). These studies, performed in the Groce No. 1 and Sears No. 1 wells, used the known resistivity properties of >1000 ohm-meter for Piedmont crystalline rocks, <350 ohm-meter for Triassic sedimentary rocks, and 250 to 350 ohm-meter for rocks directly overlying basement (Reference 2.5.1-334). Profile A-A′ (Figure 2.5.1-240) shows approximately 600 m (1969 ft.) of apparent vertical separation on the Bonsal-Morrisville fault, shows that the southeastern edge of the basin near the Jonesboro fault is defined by a zone greater than several kilometers (a few miles) wide of stepped faults north of the Jonesboro fault, and indicates that the deepest part of the basin is approximately 548 m (approximately 1800 ft.) deep (Reference 2.5.1-334). Resistivity data are supported by geologic data from wells and surface mapping, as well by correlation with gravity and magnetic surveys (Reference 2.5.1-334). Profile B-B′ (Figure 2.5.1-240) similarly shows a 2- to 5-km- (1- to 3-mi.-) wide, stepped fault zone bounding the basin, rather than a distinct trace of the Jonesboro fault, that

has 1500 to 2200 m (4921 to 7218 ft.) of vertical displacement (Reference 2.5.1-334).

Several seismic reflection lines were performed in the Deep River basin by Texaco, Inc., in the 1980s. Texaco seismic line 85SD12 is located in the Durham basin in the vicinity of the HAR site (Reference 2.5.1-340). Line 85SD12, which extends north-south for approximately 37 km (23 mi.) across the Durham basin, crosses both the Jonesboro and Bonsal-Morrisville faults and is located approximately 3.2 km (2 mi.) east of the HNP site (Figures 2.5.1-239 and 2.5.1-240). This line illustrates well the overall architecture of the Durham basin, which consists of two asymmetric half-graben, one bordered by the Jonesboro fault and the other by the Bonsal-Morrisville fault. The deepest part of the basin lies approximately 5 km (3 mi.) northwest of the surface trace of the Jonesboro fault, where the Triassic sediment-basement contact is at a depth of 1.3 seconds two-way travel time (TWTT). The Sears No. 1 well, which is located close to Shotpoint Number (SPN) 2750, was completed in the basin sediments at a depth of 1142 m (3746 ft.).

In addition to the larger-scale structures, such as the Jonesboro fault, that define the boundaries of the Durham basin, seismic line 85SD12 images several basement-involved intrabasinal faults, broad anticlinal and synclinal features, and distributed smaller-scale synrift faulting. The locations of faults are inferred based on lateral discontinuities, disruptions, or truncations of reflectors; changes in the amount or direction of the dip of packages of reflectors; and apparent vertical separation of individual reflectors or packages of reflectors.

The Jonesboro fault, which is well imaged at the southern end of the line, is defined by strong, north-dipping reflectors that can be traced into basement to a depth of 2 seconds TWTT. The fault, which has an overall listric appearance, is subparallel to the overall fabric within the bedrock of the footwall block, suggesting that it may represent a reactivated older structure. Although the listric appearance may be due in part to velocity pull-up, folding of the synrift sediments in the hanging wall suggests the presence of a rollover anticline, which also is consistent with a listric geometry. A zone approximately 5 km (3 mi.) wide of deformation in the hanging wall of the Jonesboro fault extends north from the main trace to SPN 2920. Within approximately 3 km (2 mi.) of the main trace (SPN 3080 to 2970), the faults within the basin sediments are synthetic (north-dipping) to the Jonesboro fault (Figure 2.5.1-241). A zone of antithetic (south-dipping) faults, which appears to be associated with a gentle syncline in the basal part of the Triassic basin sediments, is present from SPN 2960 to 2920. Within this zone lie the projected trace of the Harris fault and a lineament located north of the HAR footprint (informally referred to as the fire pond lineament; see discussion in Subsection 2.5.3).

Another north-dipping, low-angle fault is apparent within the basin between SPN 2800 and 2660. This fault can be traced into basement below the Triassic sediments to a depth of approximately 1.1 second TWTT. This appears to be a reverse fault having up-on-the-north displacement. Two faults mapped by NCGS, the Bonsal-Morrisville fault (SPN 2525) and a fault that projects toward

SPN 2675, appear to be more steeply dipping structures (Figure 2.5.1-241; Reference 2.5.1-208). The structure at SPN 2675 is not well defined in the seismic data; it is marked by a steeply north-dipping zone of disrupted reflectors. The Bonsal-Morrisville fault is a zone of two to three north-dipping fault traces that form the southern boundary of a second, smaller-scale Triassic basin half-graben. This fault zone also appears to be the boundary between a south-dipping structural fabric in basement rocks south of the fault and a north-dipping fabric in basement rocks north of the fault.

Near the north end of the seismic line, a zone approximately 5 km (3 mi.) wide of south-dipping faults (SPN 2080 to 2140) coincides with a strong LIDAR lineament that appears to be a northeastward continuation of the Bush Creek fault mapped by the NCGS (Reference 2.5.1-208; Figure 2.5.1-239). The southernmost of these faults becomes listric and is associated with a prominent reflector in the basement at a depth of approximately 1 second TWTT.

2.5.1.2.4.4.4 Site-Specific Geophysical Studies — HAR Site

As part of siting studies for the HAR, magnetic and seismic refraction studies were performed to explore for diabase dikes and to characterize the compressional wave velocities of subsurface units at each of the proposed construction areas (Reference 2.5.1-343). Magnetic measurements (three 244-m- [800-ft.-] long survey lines) were collected over each proposed construction area, and three seismic refraction lines were surveyed through each area: one 244-m- (800-ft.-) long west-to-east line and two 122-m- (400-ft.-) long south-to-north lines. One magnetic survey crossed the area of a previously mapped fault north of the site. This survey revealed a diabase dike that lies on projection to previously mapped dikes to the south (Reference 2.5.1-343). Based on this study, two of the previously mapped dikes at the HNP were extended north, and one new dike may have been identified (Figure 2.5.1-232; Reference 2.5.1-343). Three stratigraphic layers were observed in the seismic refraction data. The interface between layers 1 and 2 is representative of the boundary between the loose, unconsolidated overburden and a weathered or softer bedrock surface. Seismic layer 3, which has the highest average velocity, is representative of the harder, more competent bedrock material (Reference 2.5.1-343). The results of these studies are discussed in Subsection 2.5.3.

2.5.1.2.4.5 Remote Sensing Imagery

Remote sensing imagery acquired during the fault investigation for siting of the HNP included SLAR, conventional high- and low-altitude aerial photography, Skylab imagery, and Earth Resources Technology Satellite (ERTS) composites. The locations, lengths, and alignments of several hundred linear features were identified. Imagery evaluation identified none of the features as a capable fault, nor were any of the linear features that were field-checked identified as faults. Ground-truth assessments in the field indicated no evidence of recent earth movement in the site area. The Harris fault was not detected by any imagery techniques. (Reference 2.5.1-202) SLAR data collected during the study

conducted by Bain and Brown identified lineaments in the following prominent directions: north 15 degrees west, north 3 degrees east, north 50 to 62 degrees east, and north 20 to 30 degrees west (Reference 2.5.1-334). East-west-trending lineaments could not be detected because of the direction of the flight path (Reference 2.5.1-334). Most identified lineaments could be correlated to geologic features such as dikes, faults, and bedding, and to anomalies in both the gravity and magnetic surveys. Other lineaments were associated with cultural features such as roads or power lines. (Reference 2.5.1-334) In contrast to previous remote sensing studies for the HNP site, evaluation of new LIDAR data collected as part of the study for siting the HAR shows a correlation between lineaments and identified faults, as well as bedding and dikes. Newly identified lineaments are discussed in detail in Subsection 2.5.3.

2.5.1.2.5 Site Engineering Geology Evaluation

The engineering significance of geologic and geotechnical characteristics of features and materials, including foundation materials, are addressed as follows:

- **Engineering Behavior of Soil and Rock.** Sound bedrock is encountered within 1.5 and 6 m (5 and 20 ft.) below ground surface (bgs) at HAR 2, and between 4.7 and 12.5 m (15 and 40 ft.) bgs at HAR 3. Category 1 structures will be founded on sound rock, improved with localized concrete fill as necessary, at both HAR 2 and HAR 3. The engineering behavior of rock is discussed in Subsection 2.5.4.2.

- **Zones of Alteration, Weathering, and Structural Weakness.** Below the top of sound rock, very weak rock and clay seams are encountered in isolated intervals. However, such intervals are less than 1 m (about 3 ft.) thick, are oriented along sub-horizontal bedding planes, and are surrounded by sound rock immediately above and below. Due to the limited thickness and sub-horizontal orientation of these zones under the HAR 2 and HAR 3 Category 1 structures, the rock mass is a suitable foundation bearing material for these structures. The effects of very weak rock intervals, soil-like intervals, and weathered rock intervals on Category 1 structure foundations are discussed in Subsection 2.5.4.

- **Unrelieved Residual Stresses in Bedrock.** The lack of apparent stress in underground openings for mining activities in the Deep River Coal field, some as deep as 244 m (800 ft.), provides evidence that the basic stress condition in the rocks in the site vicinity can be described as “at rest” (Reference 2.5.1-202). The site is not located in an area where recent large-scale crustal stresses related to glacial or erosional/depositional loading or unloading has occurred in recent geological time. The absence of reported shallow stress-relief features and the limited seismicity in the region also suggest minimal unrelieved residual stresses in bedrock in the site vicinity.

- **Deformation Zones.** The HAR site sits in the hanging wall of the Jonesboro fault, which has been demonstrated to be a noncapable fault (see Subsection 2.5.3.4). Interpretation of seismic reflection profile line 85SD12

(Figure 2.5.1-241) indicates that the site probably lies within a zone of antithetic (south-dipping) faults related to development of the Durham basin in Mesozoic time. The Harris fault, which lies approximately 500 m (1650 ft.) south of the footprint of the HAR 2, appears to lie near the southern limit of this zone of antithetic faults. As discussed in Subsection 2.5.4.1, interpretation of the borehole data suggests that bedding dips relatively uniformly across the footprint areas of both HAR 2 and HAR 3. At HAR 2, bedrock strata dip between 6 and 9 degrees to the east. At HAR 3, strata dip approximately 20 degrees to the east-southeast based on V_s measurements, with local strata dip based on marker bed contacts marginally less than to greater than 20 degrees in some locations. Beds are traceable across the extents of the HAR 2 and HAR 3 footprints. Based on geologic mapping, interpretation of borehole data, and interpretation of geophysical investigations, there is no indication that the footprint area for either the HAR 2 or HAR 3 is intersected by a fault comparable to the Harris fault. There may be some small-scale faulting or fracturing of bedrock beneath these sites. The presence or absence of such structures will be documented by detailed logging of the excavation walls during the construction phase.

- **Prior Earthquake Effects.** There are no reported earthquakes in the site vicinity, as described in Subsection 2.5.2.1.1 and shown on Figure 2.5.2-202. Only one earthquake has occurred within 80 km (50 mi.) of the site (to the northeast), as shown on Figure 2.5.2-202. There are no historical accounts of the behavior of the site during the few earthquakes that have been felt. There is, however, no evidence of adverse behavior and no reason to expect any.

- **Effects of Human Activities.** Previous investigations performed for the HNP site and for this study identified no activities that would adversely affect the site (Reference 2.5.1-202). The HNP FSAR specifically addressed reservoir-induced seismicity (RIS). RIS associated with the filling of reservoirs has been a recognized phenomenon since local earthquakes were felt shortly after Lake Mead began to fill in the mid-1930s (Reference 2.5.1-202). The relationship of reservoir construction to local earthquakes was recognized for many reservoirs when earthquake swarms that were previously unknown in an area began to occur in the immediate vicinity of the reservoirs. It seems evident that water impoundment triggered these local earthquakes. Alternatively, many reservoirs have been impounded with little or no association with local earthquakes (Reference 2.5.1-202). A recent review of reservoir-triggered slip indicated that the largest events have occurred in areas where there is active Quaternary faulting (Reference 2.5.1-344). No active faults are recognized in the HAR site vicinity (see Subsection 2.5.3). RIS has not been associated with reservoirs in the Piedmont province of North Carolina, despite the large sizes of some of these reservoirs (Reference 2.5.1-202). In the Piedmont of South Carolina, RIS has been reported for three reservoirs in the South Carolina – Georgia seismic zone (i.e., Lakes Jocassee, Kowee, and Monticello) (Reference 2.5.1-202). In the FSAR, the HNP site reservoirs were compared to the four reservoirs where RIS has been reported in South Carolina. No correlation in pre-existing stress, water depth, water volume, geologic or hydrogeologic conditions was

found, and it was concluded that there is no reason to expect RIS at the site (Reference 2.5.1-202). This conclusion is corroborated by the fact that there are no reported earthquakes within the HNP site vicinity (see Subsection 2.5.2.1.1 and Figure 2.5.2-202).

- **Mineral Extraction.** The primary mineral products that historically have been mined in Wake County in the site vicinity are stones for construction materials, both in the form of crushed rock and large-dimension stones (Reference 2.5.1-228). There are, however, no known mineral resources of economic significance at the HAR site. Most quarries were located near Raleigh. Materials quarried or mined in Wake County include granitic gneiss, felsic gneiss, granite, soapstone, sandstone, graphite, and coal. Sand and gravel also have been quarried, mostly along river banks and primarily for highway construction. Diabase also is quarried for crushed rock (Reference 2.5.1-345). Clays were quarried near Apex and Friendship in the early 1900s for brick making (Reference 2.5.1-228). Most of the clay is mined from beds of massive red claystone in the Pekin and Sanford formations. The most numerous and uniform claystone beds occur between 180 and 600 m (590 and 1970 ft.) above the base of the Sanford Formation in a belt that runs from west of Sanford in Lee County southwestward into Moore County (Reference 2.5.1-345). Additionally, organic-rich black shale and mudstone above and below the coal bed in the Cumnock Formation have been used for fertilizer (Reference 2.5.1-345). Sand, gravel, stone, and Portland cement account for approximately 60 percent of mineral production in North Carolina (Reference 2.5.1-345). Graphite was mined in the late 1800s and early 1900s near Raleigh (Reference 2.5.1-228). Significant coal mining began in the Deep River basin after the Civil War, with the two primary mines located in Lee County (Reference 2.5.1-345). Total reserves of unmined coal in the Deep River basin are estimated at approximately 140 million tons of mostly high- to medium-volatile bituminous coal with some semianthracite adjacent to diabase dikes (Reference 2.5.1-345). The Cumnock Formation in the Sanford basin in Chatham County contains two beds of bituminous coal, known as the Cumnock coal and the Gulf coal (approximately 2.5 – 3.7 m [8 – 12 ft.] lower in the section). The Cumnock coal, which has produced a few hundred tons of coal, is exposed over an area of about 194 km^2 (75 $mi.^2$); it is thickest (1.5 m [5 ft.]) in the north. The Gulf coal covers an area of only about 67 km^2 (26 $mi.^2$). Where the coal is intruded by dikes or sills, it has been coked for several meters. (Reference 2.5.1-334) No coal of commercial value has been identified in Wake County (Reference 2.5.1-228). Additionally, in places, limonite and siderite have been mined for iron ore since the Civil War (Reference 2.5.1-334).

  Gold, tin, uranium, pyrite, lithium (as spodumene in pegmatites), copper, molybdenum, manganese, and heavy minerals have been mined in the Piedmont province (Reference 2.5.1-345). Mineral deposits in the Piedmont are associated primarily with metavolcanic/metasedimentary sequences or with intrusive bodies (Reference 2.5.1-345). Volcanic-hosted massive sulfide deposits of the Piedmont province are identified in the Carolina slate belt; these are found mostly in the Uwharrie

Formation and Albemarle Group in metamorphosed pyroclastic rocks (Reference 2.5.1-345). Mineralization typically consists of lens- to pod-shaped masses up to 500 m (1640 ft.) long and 10 m (33 ft.) thick of pyrite-calcopyrite-sphalerite-galena with variable credits for gold and silver (Reference 2.5.1-345). Barite also has been identified in the Piedmont (Reference 2.5.1-345). In addition to cherty iron formations associated with the sulfide deposits, there are isolated occurrences of oxide-facies iron formations (Reference 2.5.1-345). Gold was produced in the Piedmont from massive stratabound-stratiform sulfide deposits and from volcanic-hosted and carbonate-hosted deposits. Most of the gold deposits are in the Carolina slate belt. The Piedmont historically also has been a major province for production of high-alumina minerals such as pyrophyllite, kaolinite, and andalusite. (Reference 2.5.1-345) The proximity of the Sanford and Wadesboro basins to areas of gold mineralization in the Piedmont raises the possibility of paleoplacers in the basal conglomeratic deposits of the Triassic basin, including a small prospect in Chatham County named the Womble Mine (Reference 2.5.1-345).

- **Hydrocarbon Potential.** There have been recent studies of the feasibility of producing methane gas from coal beds fracturing at depth in Cumnock Formation in the Sanford basin. These studies have shown sufficient quality and quantity to be commercially feasible. (Reference 2.5.1-220) Black shale containing kerogen and shows of oil and gas were encountered in coreholes drilled by Reinemund and in the Groce No. 1 test well (Reference 2.5.1-334). Oil and gas exploration in the 1980s within the Durham basin identified no oil or gas prospects.

- **Groundwater Withdrawal.** Site groundwater conditions are described in Section 2.4. Groundwater in the site area is localized in the fractured rock associated with intrusive dikes and fault zones. There is limited withdrawal of groundwater in the site area, and adverse effects from groundwater withdrawal are not expected to occur at the HAR site.

Construction Groundwater Control. Construction groundwater control is discussed in Subsection 2.5.4.6. Groundwater movement within the Triassic basin bedrock is effectively limited to secondary porosity along fractures and joints. Dewatering within bedrock can likely be achieved by intermittent pumping from within the excavation, as it proceeds. At HAR 3, the deeper extent of more permeable soil and weathered rock above sound rock will require use of more active dewatering methods. Adverse effects from construction groundwater control are not expected to occur at the HAR site.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 1 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
36	78°43’	38°03’	269 km (167 mi.)	Reverse	NA	NA	Virginia Blue Ridge Complex (Paleozoic)	High-level fluvial gravel (Miocene-Pliocene)	1.0 m (3 ft.)	Base of high-level gravel (Miocene-Pliocene)	—	—	Second small fault reported at exposure. Its vertical offset is 0.3 m (1 ft.).
37	78°47’	38°02’	266 km (165 mi.)	Reverse	N20°E	57°SE	Marshall Formation of the Virginia Blue Ridge Complex (Paleozoic)	High-level fluvial gravel (Miocene-Pliocene)	1.5 m (5 ft.)	Base of high-level gravel (Miocene-Pliocene)	—	—	Four other reverse faults reported along Rt. 250 between Greenwood and the Blue Ridge Front; none could be relocated.
38	79°48’	37°49’	254 km (157 mi.)	Vertical	N25°W	90°	Romney shale (Devonian)	High-level fluvial gravel (Miocene-Pliocene)	6 m (20 ft.)	Base of high-level gravel (Miocene-Pliocene)	—	—	One of a few Tertiary faults known in Valley and Ridge province. East block is downthrown.
39	79°58’	36°43’	150 km (93 mi.)	Normal	12°E	62°SE	Rich Acres Norite (Paleozoic)	Alluvial silt and gravel (Miocene-Pleistocene [?][(a)])	5 m (16 ft.)	Base of alluvium (Miocene-Pleistocene)	—	—	Outcrop located on nose of small ridge sloping eastward toward a tributary of the Smith River. Possible slump.
40	79°54’	36°42	145 km (90 mi.)	Vertical	N17°E	90°	Norite and biotite gneiss (Paleozoic)	Sandy colluvium (Quaternary)	1.5 m (>5 ft.)	Base of colluvium (Quaternary)	—	—	Fault can be traced for about 30 m (100 ft.) along face of exposed hillside. Possible slump. East side down.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 2 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
41	78°40'	35°32'	29 km (18 mi.)	Reverse	N89°W	34°	Piedmont schist and slate (Paleozoic)	Fluvial clayey quartz sand (Upper Cretaceous-Miocene [?][(a)])	1 m (3 ft.)	Base of fluvial strata (Upper Cretaceous-Miocene)	—	—	No geomorphic anomalies in vicinity of this locality were observed during aerial reconnais-sance conducted for this study.
42	78°27'	35°38'	45 km (28 mi.)	Reverse	N18°E	47°SE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Miocene [?][(a)])	1.2 m (4 ft.)	Base of fluvial strata (Upper Cretaceous-Miocene)	—	—	Fault trace marks sharp change in slope.
43	78°23'	35°39'	51 km (32 mi.)	Reverse	N52°E	87°SE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Miocene [?][(a)])	3 m (>10 ft.)	Base of fluvial strata (Upper Cretaceous-Miocene)	2 m (>6 ft.)	Geometry of sedimentary unconformity adjacent to fault	—
44	78°15'	35°42'	65 km (40 mi.)	Vertical	N25°E	90°	Piedmont schist and slate (Paleozoic)	NA	NA	NA	—	—	No trace of sediment except for modern alluvium in local drainages.
45	78°16'	35°44'	64 km (39 mi.)	NA	N7°W	55°NE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][(a)])	NA	Base of fluvial strata (Lower Cretaceous-Pliocene)	—	—	—

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 3 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
46	78°15’	35°40’	63 km (39 mi.)	Reverse	N7°E	63°SE	Piedmont schist, slate, and phyllite (Paleozoic)	Unconsolidated clayey sand of Coharie Formation (Pliocene-Pleistocene)	2.8 m (9 ft.)	Base of Coharie Formation (Pliocene-Pleistocene)	2 m (>6 ft.)	Geometry of sedimentary unconformity adjacent to fault	Hares Crossroads fault. Geometry of Coharie unconformity on adjacent sides of fault plane suggests that significant lateral offset may have occurred on fault.
47	78°35’	35°42’	35 km (22 mi.)	Reverse	NA	NA	Piedmont schist and slate (Paleozoic)	Unconsolidated fluvial clayey sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Cretaceous-Pliocene)	—	—	Major highway and roadcuts in the vicinity of this locality are obscured by vegetation (field re-connaissance, this study)
48	78°11’	35°44’	71 km (44 mi.)	Reverse	N35°E	78°NW	Piedmont schist, slate, and phyllite (Paleozoic)	Fluvial clayey sand and gravel (Upper Cretaceous-Pliocene [?][a])	1.5 m (5 ft.)	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	Shape of fluvial deposit suggests that it is filling an old channel, with fault located in the west bank.
49	78°43’	35°38’	22 km (14 mi.)	NA	N15°W	39°SW	Piedmont schist and slate (Paleozoic)	NA	NA	NA	—	—	Grading of area adjacent to railroad has removed all post-Paleozoic strata that would define vertical displacement.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 4 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
50	78°07’	35°42’	77 km (48 mi.)	Reverse	N51°E	48°NW	Slate of the Carolina slate belt (Precambrian)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	0.3 m (1.0 ft.)	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	—
51	78°08’	35°43’	75 km (46 mi.)	NA	N65°E	33°SE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	—
52	78°10’	35°45’	73 km (45 mi.)	Normal	N14°E	76°SE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	Fault seems to parallel the side of a small stream channel.
53	78°08’	35°47’	76 km (47 mi.)	Reverse	N39°E	87°SE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	Fault traced southward to outcrops on the Norfolk and Southern Railroad.
54	78°17’	35°47’	63 km (39 mi.)	Reverse	N8°W	65°SW	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	1.0 m (3 ft.)	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	—
55	78°03’	35°51’	85 km (53 mi.)	Reverse	N13°W	87°NE	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	—
56	78°03’	35°51’	85 km (53 mi.)	Reverse	N39°E	83°	Piedmont schist and slate (Paleozoic)	Fluvial sand and gravel (Upper Cretaceous-Pliocene [?][a])	NA	Base of fluvial strata (Upper Cretaceous-Pliocene)	—	—	—
57	78°12’	35°51’	73 km (45 mi.)	NA	N10°E	79°SE	Piedmont schist and slate (Paleozoic)	Alluvial sand and gravel (Upper Cretaceous-Miocene [?][a])	NA	Base of alluvial strata (Upper Cretaceous-Miocene)	—	—	Fault presently is obscured by vegetation.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 5 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
58	79°25'	35°20'	51 km (32 mi.)	NA	N45°W	NA	Sandstone, siltstone, and shale (Triassic)	Kaolinitic sand and gravel of Middendorf Formation (Upper Cretaceous)	NA	Base of Middendorf Formation (Upper Cretaceous)	—	—	According to data taken from geologic map compiled by Reinemund, for USGS Professional Paper 246, no outcrop reveals mapped fault.
59	79°10'	35°27'	29 km (18 mi.)	NA	NA	NA	Sandstone, siltstone, and shale (Triassic)	Fluvial sand and gravel (Upper Cretaceous-Miocene [?][(a)])	0.3 m (<1.0 ft.)	Base of fluvial strata (Upper Cretaceous-Miocene)	—	—	Triassic strata are intensely faulted in area, but younger deformation is minimal. Exposures heavily overgrown by vegetation (field reconnais-sance this study).
60	77°30'	35°19'	141 km (87 mi.)	Wrench (?)][(a)]	N20°E	NA	NA	Castle Hayne Limestone (Eocene), Beaufort Formation (Paleocene), and Peedee Formation (Upper Cretaceous)	24 m (80 ft.)	Base of Beaufort Formation (Paleozoic)	Some	Geometry of claystone fractures and experimental models	Graingers fault zone. Exact locations of faults are not given, and no outcrops containing faults have been recognized.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 6 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
61	77°19’	35°17’	137 km (85 mi.)	Reverse	N63°E	>45°SE	NA	Siliceous mudstone of Jericho Run Member of Beaufort Formation (Paleocene)	NA	Bedding in Jericho Run Member	Some	Geometry of claystone fractures	Only known fault exposure in Graingers fault zone. Jericho Run fault.
62	80°05’	35°10’	112 km (70 mi.)	Reverse	N39°E	50°NW	Slate and phyllite of the Carolina slate belt (Precambrian)	High-level terrace sand and gravel (Eocene-Miocene [?][a])	3.3 m (11 ft.)	Base of terrace deposits (Eocene-Miocene)	—	—	A second fault with orientation of N10°E-80°NW is just west of reported structure, but is truncated by terrace unconformity.
63	82°19’	35°14’	309 km (191 mi.)	Reverse Strike-Slip Tear	N80°E N20°E N6°W	11°SE 30-70°SE 15°NE	Weathered migmatite (Paleozoic)	Sandy to clayey colluvium (Quaternary)	NA	Base of colluvium (Quaternary)	—	—	Outcrop, on west end of an arcuate slump block, marks junction of dislocation surface and road.
64	82°19’	35°14’	308 km (191 mi.)	Reverse	N40°W	20°NE	Weathered migmatite (Paleozoic)	Sandy colluvium (Quaternary)	4 m (13 ft.)	Base of colluvium (Quaternary)	—	—	—
65	82°20’	35°13’	310 km (192 mi.)	Normal	N8°E	67°SE	Weathered migmatite (Paleozoic)	Sandy colluvium (Quaternary)	5 m (16 ft.)	Base of colluvium (Quaternary)	—	—	Outcrop located on nose of eastward-sloping ridge normal to direction of highway. Probable gravity slide.

HAR COL 2.5-1

Table 2.5.1-201 (Sheet 7 of 7)
Cretaceous and Post-Cretaceous Faults within 320 km (200 mi.) of the HAR Site

Fault Number	Longitude	Latitude	Distance	Fault Type	Strike	Dip and Direction	Basement Rock Affected by Faulting (age)	Sedimentary Rocks (or sediments) affected by faulting (age)	Greatest Vertical Displacement	Marker Horizon (age)	Greatest Horizontal Displacement	Marker Horizon (age)	General Information
66	79°55'	34°50'	125 km (77 mi.)	Reverse	N82°W	42°NE	Argillite (Cambrian)	Kaolinitic sand and gravel of Middendorf Formation (Upper Cretaceous)	1.2 m (>4 ft.)	Base of Middendorf Formation (Upper Cretaceous)	—	—	Middendorf sediments exposed nearby on U.S. Rt. 52 may reflect effects of tectonic warping.
67	81°10'	34°05'	266 km (165 mi.)	Reverse	N80°E	87°NW	Argillite (Cambrian)	Fluvial sand and gravel (Eocene-Pliocene [?][a])	1.5 m (5 ft.)	Base of fluvial strata (Eocene-Pliocene)	—	—	A number of reverse faults were exposed in excavation (now covered), but only one had substantial offset.
150	78°42'	35°38.4'	22 km (14 mi.)	Reverse	NA	NA	Gneiss (Cambrian)	Pebbly sandy terrace deposits (Pliocene-Pleistocene [?][a])	2.6 m (>8 ft.)	Base of terrace deposits (Pliocene-Pleistocene)	—	—	Fault not included in Prowell compilation (Reference 2.5.1-256). Fault identified by Parker (Reference 2.5.1-228).

Notes:

a) Question mark indicates uncertainty in age or type of fault.

Sources: Reference 2.5.1-256 (Fault Number 36 through 69), Reference 2.5.1-228 (Fault Number 150).

HAR COL 2.5-1

Table 2.5.1-202
Postulated Geomorphic Anomalies and Geologic Evidence Cited as Evidence for the Postulated ECFS-C

River	Age of Fluvial Surface[(a)]	Amount of Channel Incision[(b)]	Upward Displaced Fluvial Surface[(c)]	Cross-Valley Change[(d)]	Sinuosity Change[(e)]	Anastomosing Stream Pattern (u = upstream; d = downstream)	River Deflection[(f)]
Little Pee Dee	Holocene 130-70 ka	No	3 m	No	No	No	None
Lumber: floodplainterrace	Holocene 130-70 ka 240-200 ka	 3 m —	 2 m —	 T1 T1	 S1 —	 No —	 C-NNE C-NNE
Cape Fear	130-70 ka	18 m	7 m[(g)]	T1, T2	S1	No	SW C-NNE[(h)]
South	130-70 ka	No	8 m	No	No	No	SW
Neuse	Unknown	35 – 40 m	—	T3	—	No	SW
Turkey Creek	Unknown	No	—	No	—	No	SW

Notes:

a) Ages of fluvial surfaces taken from (1) Colquhoun et al. (1987) (Reference 2.5.1-369).(2) Owens (1989) (Reference 2.5.1-271), (3) Marple and Talwani (2000) (Reference 2.5.1-243).

b) Incision along the Little Pee Dee and Cape Fear rivers was measured relative to the Wando terrace.

c) Amount of upward displacement along Little Pee Dee and Cape Fear rivers was measured relative to Wando terrace.

d) T1: Down-to-the-north-northeast cross-valley tilt along zone; T2: abrupt change from a relatively symmetric cross-valley shape upstream from ZRA to a down-to-the-southwest cross-valley tilt downstream; T3: abrupt change from a V-shaped valley upstream from ZRA to a broad valley downstream with a wide floodplain.

e) S1: Decreased sinuosity along ZRA followed by a sinuosity increase downstream.

f) River deflection types: SW: deflection to the southwest; C-NNE: curves in river valleys that are convex to the north-northeast.

g) Derived for Wando terrace ~7 km downstream from ZRA-C.

h) The large north-northeast-convex mender along Cape Fear River is superimposed on the river's southwest deflection.

— = parameter not measured.

ZRA = zone of river anomalies.

Source: modified from Reference 2.5.1-243, Table 1

HAR COL 2.5-1

Table 2.5.1-203
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 and 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 2.5.1-264, Table 1.

HAR COL 2.5-1

Table 2.5.1-204
Comparison of Post-EPRI Magnitude Estimates for the 1886 Charleston Earthquake

Source Reference	Magnitude Estimation Method	Reported Magnitude Estimate	Assigned Weights	Mean Magnitude (M)
Johnston et al. (Reference 2.5.1-295)	Worldwide survey of passive-margin, extended-crust earthquakes	**M** 7.56 ± 0.35[a]	—	7.56
Martin and Clough (Reference 2.5.1-310)	Geotechnical assessment of 1886 liquefaction data	**M** 7 – 7.5	—	7.25
Johnston (Reference 2.5.1-309)	Isoseismal area regression, accounting for eastern North America anelastic attenuation	**M** 7.3 ± 0.26	—	7.3
Chapman and Talwani (Reference 2.5.1-346) (South Carolina Department of Transportation)	Consideration of available magnitude estimates	**M** 7.1 **M** 7.3 **M** 7.5	0.2 0.6 0.2	7.3
Frankel et al. (Reference 2.5.1-347) (USGS national seismic hazard mapping project)	Consideration of available magnitude estimates	**M** 6.8 **M** 7.1 **M** 7.3 **M** 7.5	0.20 0.20 0.45 0.15	7.2
Bakun and Hopper (Reference 2.5.1-294)	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 2.5.1-295, Chapter 3).

b) Ninety-five percent confidence interval estimate; M_I (intensity magnitude) is considered equivalent to **M** (Reference 2.5.1-294).

c) Bakun and Hopper’s (Reference 2.5.1-294) *preferred* estimate.

Source: Reference 2.5.1-263.

HAR COL 2.5-1

Table 2.5.1-205
Timing and Source of Liquefaction Events in Southern Atlantic Coastal Plain

Liquefaction Episode	Age, Years B.P.	Scenario 1		Scenario 2	
		Source	Magnitude[a]	Source	Magnitude[a]
1886 AD	113	Charleston	7.3	Charleston	7.3
A	546±17	Charleston	7+	Charleston	7+
B	1021±30	Charleston	7+	Charleston	7+
C	1648±74	Northern Part	~6.0	--	--
C´	1683±70	--	--	Charleston	7+
D	1966 ±212	Southern Part	~6.0	--	--
E	3548±66	Charleston	7+	Charleston	7+
F	5038± 166	Northern Part	~6.0	Charleston	7+
G	5800±500	Charleston	7+	Charleston	7+

a) Magnitude is M_w; 1886 magnitude is from Johnston (1996) (Reference 2.5.1-309).

Source: Reference 2.5.1-301

2.5.2 VIBRATORY GROUND MOTION

HAR COL 2.5-2

This subsection provides a detailed description of vibratory ground motion assessments that were carried out for the HAR site. 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.

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 (Reference 2.5.2-201, Reference 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, geophysics; and, separately, development of state-of-knowledge earthquake ground motion modeling, including epistemic and aleatory uncertainties.[a] 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 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.

1. Perform regional and site geological, seismological, and geophysical investigation in accordance with Regulatory Position 1 and Appendix C to RG 1.208.

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a. 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.

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 provided 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 CAV. The alternative approach using the CAV filter is used to develop the final GMRS for the HAR 2 and HAR 3 sites.

This subsection discusses the following aspects of vibratory ground motion:

- Seismicity (Subsection 2.5.2.1).
- Geologic structures and seismic source models (Subsection 2.5.2.2).
- Correlation of earthquake activity with seismic sources (Subsection 2.5.2.3).
- Probabilistic seismic hazard analysis and controlling earthquake (Subsection 2.5.2.4).
- Seismic wave transmission characteristics of the site (Subsection 2.5.2.5).
- Ground motion response spectra (Subsection 2.5.2.6).

2.5.2.1 Seismicity

An important component in developing a seismic hazard model for the HAR site is the seismic history of the region. The selected starting point for developing the site-specific PSHA for the HAR site is the EPRI-SOG seismic hazard model for the CEUS (Reference 2.5.2-201). 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 HAR 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 Subsection 2.5.2.1.1. Information on significant earthquakes is provided in Subsection 2.5.2.1.2.

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 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 (Reference 2.5.2-204). 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[b] 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 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 (Reference 2.5.2-209), Metzger et al. (Reference 2.5.2-210), and Munsey (Reference 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).

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b. The PSHA formulation used in this study assumes that the temporal occurrence of earthquakes conforms to a Poisson process, implying independence between the times of occurrence of earthquakes. Thus it is necessary to remove dependent events (such as foreshocks and aftershocks) from the earthquake catalog before estimating earthquake frequency rates.

The catalog for the HAR site consists of the Bellefonte GG&S earthquake catalog extended from February 2005 through December 2006 using the listing of recent earthquakes obtained from the Advanced National Seismic System (ANSS) Website (Reference 2.5.2-208, Reference 2.5.2-212).

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 HAR site. Note that only one earthquake in the project catalog has occurred within 80 km (50 mi.) of the HAR 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.

Appendix 2AA lists the earthquakes in the updated catalog that have occurred within 320 km (200 mi.) of the HAR site. The list consists of 225 events of $m_b \geq 3$ that occurred between 1774 and January 1, 2007. The size distribution of these earthquakes consists of 180 events with $3 \leq m_b < 4$; 39 events with $4 \leq m_b < 5$; 5 events with $5 \leq m_b < 6$; and only 1, the Charleston earthquake of 1886, with a magnitude exceeding m_b 6, which occurred approximately 315 km (195 mi.) from the HAR site. Estimates of the Modified Mercalli Intensity (MMI) are available for 171 earthquakes. Maximum intensities larger than MMI IV are reported for 159 events, out of which 24 have maximum intensities between VI and VII½, and 1, the 1886 Charleston earthquake, has a maximum intensity of X.

Focal depths in the range of 1 to 17 km (0.6 to 11 mi.) are reported for 61 earthquakes with m_b between 3 and 4.6; about half of the earthquakes have fixed depths of 1, 5, or 10 km (0.6, 3, or 6 mi.). Only eight events have listed depths greater than 10 km (6 mi.). 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 HAR study. Estimated seismic moments are provided for the 320-km (200-mi.) radius catalog in Appendix 2AA. The values listed for nine earthquakes were taken from Johnston et al. (Reference 2.5.2-213). For the remaining earthquakes, seismic moments were estimated by first estimating moment magnitude using the three relationships described in 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 2.5.2-214).

2.5.2.1.2 Significant Earthquakes

Ten earthquakes with estimated magnitudes of approximately m_b 4.9 or larger have been recorded within 320 km (200 mi.) of the HAR site. One additional earthquake of similar size was previously located within 320 km (200 mi.), but a revised location places it 415 km (260 mi.) from the site. The locations of these earthquakes as listed in the project catalog are shown on Figure 2.5.2-202. These earthquakes are described in chronological order as follows:

- **January 8, 1817.** The location of this intensity V event is uncertain and it was reported to have been felt in Milledgeville, Georgia; Charleston, South Carolina; New Bern and Salem, North Carolina; and Baltimore, Maryland. The EPRI-SOG catalog locates this event in North Carolina at latitude 36N, longitude 80.2W, and lists an expected m_b value of 4.9 (Reference 2.5.2-201). The NCEER-91 catalog lists the magnitude as m_b 5 based on felt area and the location as 33N, 70W (Reference 2.5.2-205). The HAR catalog uses the USGS National Hazard Mapping catalog location, which is latitude 32N, longitude 80W, near Charleston, South Carolina.

- **March 9, 1828.** This earthquake was apparently centered in southwestern Virginia and was reported felt over an area of about 565,000 km^2 (218,148 $mi.^2$), from Pennsylvania to South Carolina and the Atlantic Coastal Plain to Ohio. Reports in Virginia indicated that doors and windows rattled (MMI V). President John Quincy Adams felt this tremor in Washington, D.C.; in his diary he compared the sensation to the heaving of a ship at sea. (Reference 2.5.2-215)

- **April 29, 1852.** A moderately strong, widely felt shock occurred on April 29, 1852. At Buckingham and Wytheville, Virginia, chimneys were reportedly damaged (MMI VI). The felt area extended to Washington, D.C., Baltimore, Maryland, and Philadelphia, Pennsylvania, and included many points in North Carolina, covering approximately 420,000 km^2 (162,163 $mi.^2$). (Reference 2.5.2-215)

- **August 31, 1861.** The EPRI-SOG catalog locates this event in western North Carolina (36.2N, 81.2W) and lists the expected m_b as 5.0 (Reference 2.5.2-201). The NCEER-91 catalog revised the magnitude to m_b 5.2 based on felt area and moved the event location to the North Carolina – Tennessee border based on the analysis of felt report data (Reference 2.5.2-205, Reference 2.5.2-216). This location and magnitude were retained in the USGS National Hazard Mapping catalog.

- **December 22, 1875 (December 23, GMT).** The Central Virginia seismic zone is described in Subsection 2.5.1.1.4.2.5.1. The following description of the 1875 earthquake that occurred in this zone is from the HNP FSAR (Reference 2.5.2-217):

> The main shock of maximum intensity VII occurred at 11:45 p.m. EST on December 22, 1875. The epicenter was probably about 37.6° North, 78.5° West (near Richmond, Virginia), about 133 miles northeast of the site. The total affected area was about 50,000 square miles. This shock, preceded by a minor shock on March 10, was felt over a relatively large elliptical area extending from Baltimore, Maryland, southwest to Greensboro, North Carolina, and from the Atlantic Coast westward to Greenbrier County, West Virginia. Near the epicenter, five shocks occurred in quick succession. Bricks were shaken from chimneys in Goochland and Powhatan Counties, and shingles were shaken from a roof at Manakin, Virginia. A chimney collapsed in Wilmington, North Carolina. At Richmond, Virginia, the shock lasted 20 to 30 seconds, and deep rumbling was noted. There were no reports of the shock having been felt in the site vicinity. Numerous other small shocks have occurred in the Richmond-Charlottesville-Arvonia region; the largest possibly had intensities approaching that of the 1875 shock. Such shocks occurred in 1774, 1833, 1852, 1885, and 1907. Minor activity has occurred as recently as 1966 (intensity V near Richmond).

- **September 1, 1886.** The Charleston earthquake occurred about 315 km (195 mi.) south of the HAR site. The following summary of the effects of the 1886 Charleston earthquake is from the HNP FSAR (Reference 2.5.2-217):

> These were the strongest shocks in the southeastern United States in historical times. There were two main shocks, at 9:51 p.m. and 9:59 p.m. EST on August 31, 1886. Two epicentral tracks were identified, one near Summerville, 16 miles northwest of Charleston. The approximate epicentral coordinates are 32.9° North, 80.0° West. The shocks were preceded by explosion-like sounds near Summerville on August 27th and 28th. The main shocks were followed by aftershocks, some of rather high intensity, into the next day. The duration of the first shock was about 35 to 40 seconds. The shocks were felt in an area of about 2,000,000 square miles. The area within a distance of about 100 miles of Charleston was strongly shaken. The most serious reports came from the major population center of Charleston, where people were terrified and damage was extensive. About 60 people were killed. [Reference 2.5.2-217]

- **October 22, 1886.** This earthquake occurred at 2:45 p.m. in South Carolina. There is no depth information for this event. The shock was felt in towns that

included Charleston, South Carolina, and Atlanta and Augusta, Georgia. (Reference 2.5.2-218) In the epicentral area, the maximum intensity reached VII on the MMI scale, from which a magnitude of 5.2 m_b has been estimated. An average moment magnitude of 4.7 is computed from m_b using different conversion equations, and a seismic moment of 1.4×10^{23} dyne-cm is computed from the estimated moment magnitude using the Hanks and Kanamori formula (Reference 2.5.2-214). This earthquake is designated as an aftershock of the Charleston earthquake in the EPRI-SOG catalog.

- **May 31, 1897.** The following description of the 1897 Giles County earthquake is from the HNP FSAR (Reference 2.5.2-217):

  > The shock of maximum intensity VIII occurred at 1:58 p.m. EST on May 31, 1897. The epicenter was about 37.3° North, 80.7 ° West, about 160 miles northwest of the site; the total affected area was about 280,000 square miles. The shock was felt from Georgia on the south to Pennsylvania on the north and from the Atlantic Coast to Indiana and Kentucky, but most strongly at Pearisburg, Giles County, Virginia. Old brick houses and chimneys were cracked, and bricks were shaken from chimney tops. Fissures appeared in the ground, and small landslides were noted. At Narrows, Virginia, on the New River, near the West Virginia border, it was claimed that a motion like the ground swell of the ocean was observed. Large rocks rolled down the mountains. The shock was accompanied by loud sounds. At Raleigh, North Carolina, two shocks were felt, and a few chimneys were damaged. The shock was preceded by four shocks between May 3rd and May 31st, and was followed by aftershocks until June 6. This shock was probably felt in the site area with an intensity of about V. There have been numerous additional shocks in the Giles County area, the most recent in 1968 (intensity IV, near Narrows, Virginia). [Reference 2.5.2-217]

- **June 12, 1912.** This earthquake occurred in South Carolina. The data available do not include a focal depth estimate. The earthquake caused some damage to chimneys in Summerville, was felt at Charleston with MMI VI, and was felt in an area of about 90,000 km^2 (34,749 $mi.^2$) including locations in Georgia, and South and North Carolina. (Reference 2.5.2-218) The maximum intensity in the epicentral area is quantified as VII in the MMI scale, from which Johnston et al. (Reference 2.5.2-213) estimated a moment magnitude (**M**) of 4.5 (corresponding to a seismic moment of 6.3×10^{22} dyne-cm). The estimated m_b is 4.9.

- **January 1, 1913.** This earthquake occurred in Union County, South Carolina. The estimated m_b is listed as 4.9 in the EPRI-SOG catalog and 5.0 in the NCEER-91 and USGS National Seismic Hazard Mapping catalogs

(Reference 2.5.2-205). The HNP FSAR contains the following description of this earthquake (Reference 2.5.2-217):

> The shock occurred at 1:28 p.m. EST on January 1, 1913. The epicenter was about 34.7° North, and 81.7° West, about 175 miles southwest of the site. The maximum intensity of the shock was VII-VIII [VIII on Rossi-Forel scale, Taber (1913)]. It was felt in an elliptical area, 45 miles by 25 miles, trending north-northeast south-southwest. Wave-like motions of the ground were reported in several places. The shock was accompanied by noises like thunder. It was probably felt in the site area with an intensity on the order of I-III. The total area affected was about 43,000 square miles, with the shock felt in North Carolina, eastern Tennessee, and southeastern Georgia. On June 26, 1945, an intensity VI shock occurred in this same area near Murray Lake, South Carolina. Chimneys were reported cracked near the epicenter. Rumbling noises also preceded this shock. [Reference 2.5.2-217]

- **February 21, 1916.** The EPRI-SOG and NCEER-91 catalogs list the location for this intensity VII event as latitude 35.5N, longitude 82.5W (Reference 2.5.2-201, Reference 2.5.2-205). The estimated magnitudes from these catalogs are m_b as 4.9 and 5.2, respectively. The USGS National Seismic Hazard Mapping catalog gives the location as latitude 35.62N, longitude 83.55W and attributes the location to Dr. Martin Chapman of Virginia Tech. The estimated magnitude is m_b 5.0 in the USGS catalog. The revised location places this earthquake approximately 415 km (260 mi.) west of the HAR site (Appendix 2AA).

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. 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 HAR site. This subsection presents a description of the seismic source characterizations from the EPRI-SOG evaluation (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 (Subsection 2.5.2.2.2) (Reference 2.5.2-201). 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 the EPRI (Reference 2.5.2-201). 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 (Reference 2.5.2-201). Seismic sources and associated interpretations necessary for hazard calculations at any nuclear power plant site in the CEUS were developed (Reference 2.5.2-201). Six earth science teams (ESTs) provided input interpretations: Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical, and Woodward-Clyde Consultants (Reference 2.5.2-201). Each team produced a report (Volumes 5 through 10 of EPRI-SOG [Reference 2.5.2-201]) that provided descriptions of how the seismic sources were identified and defined.

The seismic source characterizations developed by the EPRI-SOG expert teams were used to conduct PSHAs for nuclear power plant sites in the CEUS that were reported in EPRI (Reference 2.5.2-202). The calculations performed for each site excluded the seismic sources defined by each EPRI-SOG expert team that, in combination, contributed less than one percent to the total hazard computed from all sources defined by that expert team. Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206 list the seismic sources for each of the six EPRI-SOG teams that were included in the EPRI PSHA calculations for the HNP site (Reference 2.5.2-202). These seismic sources are shown on Figures 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, and 2.5.2-208 and are described in 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.

The EPRI identification of seismic sources that are significant to assessing the seismic hazard at the HAR site was based on calculations made with the ground motion models presented in EPRI-SOG (Reference 2.5.2-202, Reference 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 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 was re-examined using updated ground motion models. This examination is presented in Subsection 2.5.2.4.3.1. The additional seismic sources defined by the EPRI-SOG expert teams in the vicinity of the site (within a radius of 320 km [200 mi.]) are also listed in Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206 and are described in the following subsections. Many of the seismic sources described by the EPRI-SOG teams are also described in Subsection 2.5.1.1.4.2.5, including the zones associated with the 1886 Charleston, South Carolina, earthquake; Giles County, Virginia; and Central Virginia.

2.5.2.2.1.1 Bechtel (BEC) Team Seismic Sources

The EPRI calculations for the HNP site included six seismic sources defined by the Bechtel team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-201 and are described briefly in the following list. Five additional seismic sources, also shown in Table 2.5.2-201, were tested in the sensitivity analysis for the current PSHA using updated ground motion models.

- **Eastern Mesozoic Basins (Source 13).** The HAR site is located within this seismic source (source 13). The eastern Mesozoic basins are generally northeast-trending, tilted, elongated, sediment-filled troughs of Late Triassic to Early Jurassic age. The basins appear to be bounded on one or both sides by major normal faults that typically follow pre-Triassic faults. The basins also may be cut by longitudinal faults parallel to the border faults and by transverse faults approximately normal to the margins (Reference 2.5.2-219). The Bechtel team stated that the evidence for association of the Mesozoic basin faults with earthquakes is ambiguous and gave it a low probability (0.1) of existence (Reference 2.5.2-219).

- **Southeast Appalachians (Source F).** This seismic source zone is located about 77 km (48 mi.) to the west of the HAR site (Reference 2.5.2-219). No specific information on this source is provided by the Bechtel team.

- **Charleston Area (Source H) and Charleston Faults (Source N3).** These two seismic source zones are located in the vicinity of Charleston, South Carolina (Reference 2.5.2-219). No specific information on these sources is provided by the Bechtel team other than that these sources represent possible source locations for the 1886 Charleston earthquake.

- **Atlantic Coastal Region (Background) (Source BZ4).** The Atlantic Coastal region background source BZ4 is a large background zone 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 2.5.2-219).

- **Southern Appalachians Region (BZ5).** The HAR site is located within the Southern Appalachians region background source (BZ5). This source zone includes the Mesozoic basins; the Giles County, Virginia, structure; the Bristol block trends; Central Virginia; and southeast Appalachians (sources 13, 19, 24, E, and F, respectively) (Reference 2.5.2-219).

- **Giles County (Source 19).** The Giles County seismic zone exhibits fairly intense small earthquake activity. The moderate-to-large 1897 Giles County earthquake occurred near this seismic zone, within the Bristol block zone (source 24). The seismic zone may have some association with deep crustal features based on geophysical data (Reference 2.5.2-219).

- **Bristol Block (Source 24).** The Bristol block geopotential trends are distributed over a fairly large area and are associated with some widely distributed moderate-to-large earthquakes. The 1897 Giles County earthquake occurred within the Bristol block (Reference 2.5.2-219). These tectonic features are associated with deep crustal features that are buried beneath younger Appalachian thrust sheets that do not appear to be affected by the deep crustal lineaments (Reference 2.5.2-219).

- **New York – Alabama Lineament (Sources 25 and 25A).** The New York – Alabama geopotential lineament seismic source zone is associated with the Tennessee segment of the New York – Alabama geopotential lineament tectonic feature. A large number of small earthquakes occur in this area and their trend could be parallel or subparallel to this tectonic feature. A moderate-to-large earthquake that occurred in 1916 could also be associated with this tectonic feature. (Reference 2.5.2-219)

- **Central Virginia (Source E).** This seismic source zone in central Virginia is located about 189 km (117 mi.) to the north of the HAR site (Reference 2.5.2-219). No specific information on this source is provided by the Bechtel team.

2.5.2.2.1.2 Dames & Moore (DAM) Team Seismic Sources

The EPRI calculations for the HNP site included four seismic sources defined by the Dames & Moore team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-202 and are described briefly in the following list. Three additional seismic sources, also shown in Table 2.5.2-202, were tested in the sensitivity analysis for the current PSHA using updated ground motion models.

- **Central Virginia (Source 40).** The Central Virginia seismic source zone (source 40) is about 170 km (105 mi.) north of the HAR site. The basis for this source zone is the clustered seismicity defined in the central Virginia area (Reference 2.5.2-220).

- **Southern Cratonic Margin (Default) (Source 41).** The Southern Cratonic Margin default zone (source 41) is a large background source that encompasses a 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 Dan River basin (source zones 42, 43, 46, respectively, which are considered mutually exclusive with the default zone, 41). This source zone contains seismicity in a diffuse pattern throughout the zone. (Reference 2.5.2-220) These are small fault-related sources located far from the HAR site and were not included in the PSHA for the HNP site.

- **Southern Appalachians Mobile Belt (Default) (Source 53).** The HAR site is located within the Southern Appalachians Mobile Belt default zone (source 53). This default source contains several Triassic basins and is composed of

crustal rocks that have undergone several periods of crustal divergence and convergence. Much of the seismicity is diffuse throughout the zone. (Reference 2.5.2-220)

- **Charleston Seismic Zone (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 are the Woodstock fault, Ashley River fault, Cooke fault, and the Helena Banks fault. (Reference 2.5.2-220)

- **Appalachian Fold Belt (Source 4) and Appalachian Fold Belt Kinks (Sources 4A, 4B, 4C, and 4D).** The Appalachian fold belt source zone is a long zone of Paleozoic folds that compose a major segment of the Appalachian Mountains from New York to Alabama. An alternative model consists of four “kinks” (sources 4A, 4B, 4C, and 4D) (Reference 2.5.2-220) located at concentrations of seismicity, including the East Tennessee seismic zone and the source area for the 1897 Giles County earthquake.

- **Mesozoic/Cenozoic Basins (Sources 47, 48, 49, 50, 51, and 53).** These sources define Mesozoic/Cenozoic Basins located in the Southern Appalachians Mobile Belt source zone (source 53). These sources represent potential earthquake sources from reactivation of basin structures. The HAR site lies within source 49, the Jonesboro basin (Reference 2.5.2-220). No earthquakes of magnitude larger than m_b 3.3 are associated with this source and it was assigned a seismicity rate of zero in the EPRI-SOG characterization (Reference 2.5.2-201).

- **Charleston Mesozoic Rift (Source 52).** This Mesozoic rift source may have some association with seismicity in the Charleston, South Carolina, area (Reference 2.5.2-220).

2.5.2.2.1.3 Law Engineering (LAW) Team Seismic Sources

The EPRI calculations for the HNP site included six seismic sources defined by the Law Engineering team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-203 and described briefly in the following list. One additional seismic source, also shown in Table 2.5.2-203, was tested in the sensitivity analysis for the current PSHA.

- **Charleston Seismic Zone (Source 35).** This source zone was defined by the Law Engineering team “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 2.5.2-221). 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 2.5.2-221)

- **Eastern Piedmont (Background Zone) (Source 107).** The HAR site is located within this large background zone. This region is characterized by a positive Bouguer gravity field and a pattern of short wavelength magnetic anomalies; the crust within this zone is believed to be thinner than the crust to the west (Reference 2.5.2-221).

- **Eastern Basement (Source 17) and Alternative Eastern Basement Background (Source 217).** The HAR site is located approximately 122 km (76 mi.) east of the Eastern Basement source zone. The Giles County and Eastern Tennessee zones of seismicity are included in this source (Reference 2.5.2-221). Source 217 is an alternative zone that encompasses the same area but has smaller maximum magnitudes.

- **Mafic Plutons (38).** Mafic plutons are considered seismic source zones by Law Engineering (Reference 2.5.2-221). Mafic plutons M27, M28, and M31 - M35 are located in the site regions (Figure 2.5.2-205).

- **Mesozoic Basins (Sources 8, C09, C10).** Mesozoic basins are recognized as potential seismic sources by the Law Engineering team (Reference 2.5.2-221). Sources C09 and C10 represent the portion of source 8 not contained in other sources.

- **Eastern Seaboard (Sources 22, C11).** This source represents reactivated normal faults located throughout the Eastern Seaboard (Reference 2.5.2-221). Source C11 represents the portion of source 22 not contained in other sources.

2.5.2.2.1.4 Rondout Associates (RND) Team Seismic Sources

The EPRI calculations for the HNP site included five seismic sources defined by the Rondout Associates team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-204 and described briefly in the following list. Four additional seismic sources, also shown in Table 2.5.2-204, were tested in the sensitivity analysis for the current PSHA.

- **Charleston (Source 24).** The Charleston seismic zone includes the Ashley River fault and Woodstock fault (Reference 2.5.2-222).

- **South Carolina Seismic 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 2.5.2-222)

- **Giles County Seismic Zone (Source 28).** The Giles County source was defined based on historical seismicity, most notably the 1897 m_b 5.8 Giles County earthquake (Reference 2.5.2-222).

- **Central Virginia Seismic Zone (Source 29).** This source was defined based on seismicity and the possible intersection of the extension of the Norfolk fault zone and the northeast-trending linear zone defined by magnetic anomalies, gravity anomalies, and volcanic-plutonic rocks (Reference 2.5.2-222).

- **Appalachian Crust (Sources 49, C01).** This crust was formed after the Precambrian and the basement is a complex accretionary terrane. The zone may not have a uniform seismic potential (Reference 2.5.2-222). Source C01 represents the portion of background source 49 not contained in other sources.

- **Southern Appalachian Seismic Zone (Source 25).** The Southern Appalachians seismic zone is defined on the basis of deep-seated seismicity lying below the décollement (Reference 2.5.2-222).

- **Tennessee/Virginia Border Zone (Source 27).** This zone is located along the New York – Alabama lineament between the more seismically active areas of Eastern Tennessee and Giles County (Reference 2.5.2-222).

- **Norfolk Fracture Zone (Source 32).** The area of underlying crustal weakness that defines this zone has a low correlation with earthquakes, but nonetheless is considered a potential earthquake source (Reference 2.5.2-222).

- **Grenville Crust (Sources 50, C02).** This area was separated from the other Precambrian crust areas because it appears to have a higher level of background seismicity (Reference 2.5.2-222). Source C02 represents the portion of background source 49 not contained in other sources.

2.5.2.2.1.5 Weston Geophysical (WGC) Team Seismic Sources

The EPRI calculations for the HNP site included six seismic sources defined by the Weston team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-205 and described briefly in the following list. Three additional seismic sources, also shown in Table 2.5.2-205, were tested in the sensitivity analysis for the current PSHA.

- **Central Virginia Seismic Zone (Source 22).** The CVSZ is a northwest-trending alignment of seismicity that extends across central Virginia. Faults that can be traced beneath this zone using regional geologic and geophysical data have a spatial correlation with recent seismicity (Reference 2.5.2-223).

- **Charleston Seismic Zone (Source 25).** Several tectonic features are included in this zone, and it is assumed by the Weston team that future seismicity 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 décollement (Reference 2.5.2-223). This zone is also part of the South Coastal Plain background (source 104) (Reference 2.5.2-223).

- **South Carolina Seismic Zone (Sources 26, C33).** This zone is also part of the South Coastal Plain background (source 104) (Reference 2.5.2-223). Source C33 consists of the portion of source 26 not contained within source 25.

- **Mesozoic Basins (Sources 28C, 28D, 28E).** The HAR site is located within basin source 28D, which corresponds to the Deep River basin (Figure 2.5.2-208). This zone is also part of the South Coastal Plain background (source 104) (Reference 2.5.2-223).

- **Southern Appalachian Background Zone (Sources 103, C17, C18, C19).** This source includes the Inner Piedmont, Blue Ridge, and Valley and Ridge physiographic belts of the southern Appalachians (Reference 2.5.2-223). Sources C17, C18, and C19 represent those portions of source 103 not contained in other sources. This zone also contains the Giles County and New York – Alabama lineament – Clingman lineament sources (sources 23 and 24).

- **South Coastal Plain Background (Sources 104, C20 through C28, C34, C35).** Several additional seismic sources are included within this zone, including the CVSZ (source 22), the Charleston, South Carolina, seismic zone (source 25), and the zones of Mesozoic basins (source 28) (Reference 2.5.2-223). This background seismicity zone shares a common boundary on the west with the Southern Appalachian background zone (source 103) (Reference 2.5.2-223). Sources C20 through C28, C34, and C35 represent those portions of source 104 not contained in other sources.

- **Giles County Seismic Zone (Source 23).** The Giles County seismic zone is a well-defined, discrete cluster of seismic activity in Giles County, Virginia; this zone is also in the northern segment of the Alabama – New York lineament and Clingman lineament zones (Reference 2.5.2-223).

- **New York – Alabama Lineament – Clingman Lineament (Source 24).** A crustal block between the New York – Alabama Lineament – Clingman lineaments is defined on the basis of instrumental seismicity data (Reference 2.5.2-223).

2.5.2.2.1.6 Woodward-Clyde Consultants (WCC) Team Seismic Sources

The EPRI calculations for the HNP site included seven seismic sources defined by the Woodward-Clyde Consultants team (Reference 2.5.2-202). These sources are listed in Table 2.5.2-206 and described briefly in the following list. Three additional seismic sources, also shown in Table 2.5.2-206, were tested in the sensitivity analysis for the current PSHA.

- **Central Virginia Gravity Saddle (Source 26).** The Central Virginia Gravity Saddle source (also known as the Central Virginia seismic zone, or CVSZ) represents a saddle in the northeast-trending gravity high associated with the Appalachians in central Virginia. Seismicity is located along the south and southwest sides of the gravity saddle (Reference 2.5.2-224).

- **State Farm Complex (Source 27).** The State Farm Complex source is defined based on Precambrian gneissic terrain located in central Virginia. It is bounded on the east by the Richmond basin and associated structures and on the west by the Goochland fault. There is a strong concentration of seismicity on either side of the feature, which is centered in the CVSZ (Reference 2.5.2-224).

- **Greater South Carolina Source Zone (Source 29).** This source zone pertains to seismicity located in South Carolina, Georgia, and western North Carolina (Reference 2.5.2-224). Alternative source zone designations (29, 29A, 29B) are used for the hazard calculations, as shown on Figure 2.5.2-208. 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 2.5.2-224).

- **Charleston Source Zone (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 2.5.2-224). 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 2.5.2-224).

- **Background Source Zone (Source B24).** The HAR site is located within a large background zone that encompasses most of North Carolina (Figure 2.5.2-208).

- **Blue Ridge Source Zone (Sources 31, 31A).** This source is an inferred crustal block, defined by magnetic data and seismicity, which is located in eastern Tennessee. The block may represent a faulted fragment of Cratonic crust or a Precambrian accreted terrane (Reference 2.5.2-224).

- **Richmond Basin (Source 28).** The source is a small Triassic basin located near Richmond, Virginia. Seismicity in the vicinity of this basin includes the Central Virginia seismic zone, which extends over a larger region (Reference 2.5.2-224).

2.5.2.2.2 Post-EPRI Seismic Source Characterizations

Seismic hazard studies conducted in the HAR site region since completion of the 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 (LLNL) Trial Implementation Program (TIP) Source Evaluations

A decade after the completion of the EPRI-SOG (Reference 2.5.2-201) evaluation, Lawrence Livermore National Laboratory (LLNL) (Reference 2.5.2-225) conducted a Trial Implementation Program (TIP) of the Senior Seismic Hazard Analysis Committee (SSHAC) guidance for a Level IV analysis (Reference 2.5.2-226). The LLNL TIP project focused on issues related to the development of seismic zonation and earthquake recurrence models. Participants in the project included a Technical/Facilitator/Integrator (TFI) team, a panel of five expert evaluators, and expert proponents and presenters. Preliminary implementations for two sites in the southeastern 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 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 HAR 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 2.5.2-209. 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 2.5.2-207 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 2.5.2-225).

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 2.5.2-227). Input for revising the source characterization used in the 1996 hazard maps was provided by researchers through a series of regional workshops (Reference 2.5.2-228). 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 source model and earthquake catalog (in m_b) developed by the USGS are shown on Figure 2.5.2-210 (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 (NMSZ), and alternative zones for a Charleston seismic source zone (Figure 2.5.2-210). 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 (SC DOT) as part of a program to develop seismic design specifications for highway bridges (Reference 2.5.2-229). The approach used in this study is similar to that used by the USGS to develop the 2002 national seismic hazard maps. The SC DOT 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 2.5.2-211.

The SC DOT 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 2.5.2-210). 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 (ZRA) (postulated ECFS) 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 SC DOT study is a mean recurrence interval of 550 years.

In the region of the HAR site, the SC DOT study defines two alternative source zone configurations. One configuration includes the site in a Piedmont – Coastal Plain zone (source area 6), which is defined based on a lack of historical seismicity, and defines 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 2.5.2-211). 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 2.5.2-230). The result was the development of an updated UCSS. The UCSS consists of the four alternative geometries for the seismic source, shown on Figure 2.5.2-212, and the seismic source logic tree, shown on Figure 2.5.2-213, 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 2.5.2-230) integrated the UCSS into the EPRI-SOG seismic source characterization by replacing each expert team'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.

2.5.2.3 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 HAR site region was developed (see discussion in 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 2.5.2-203, 2.5.2-204, 2.5.2-205, 2.5.2-206, 2.5.2-207, and 2.5.2-208. 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 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 principal sources of seismic activity are the Charleston area, the ETSZ, the area around the 1897 Giles County earthquake, and the Central Virginia seismic zone. These concentrations of seismicity were recognized and considered by the EPRI-SOG teams, as discussed in Subsection 2.5.2.2.1.

- The largest earthquake within the site region, 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 (Subsection 2.5.1.1.4.3). Paleoliquefaction studies indicate that repeated large-magnitude earthquakes have occurred in the epicentral region of the 1886 Charleston earthquake (see discussion in Subsection 2.5.1.1.4.3). Alternative source locations, maximum magnitudes,

and recurrence for repeated large-magnitude, Charleston-type earthquakes are discussed in Subsection 2.5.2.4.1.

- The updated earthquake catalog adds several m_b 3 to 5 earthquakes in the time period covered by the EPRI-SOG catalog (principally prior to 1910). The effect of these additional events on estimated seismicity rates is assessed in Subsection 2.5.2.4.1.2.

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

This subsection describes the PSHA conducted for the HAR site. Following the procedures outlined in Regulatory Guide 1.208, 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. 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 Subsection 2.5.2.4.4. The results of that analysis are used for the development of UHRS and identification of the controlling earthquakes (Subsection 2.5.2.4.4.2).

2.5.2.4.1 New Information Relative to Seismic Sources

This section 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 vicinity.

- Changes in the characterization of the rate of earthquake occurrence for one or more seismic sources.

- Changes in the characterization of the maximum magnitude for seismic sources.

2.5.2.4.1.1 Identification of Seismic Sources

Based on the review of new geological, geophysical, and seismological information that is summarized in Subsection 2.5.1, the review of seismic source characterization models developed for post-EPRI-SOG seismic hazard analyses (Subsection 2.5.2.2.2), and a comparison of the updated earthquake catalog to the EPRI-SOG evaluation (Subsection 2.5.2.3), two potential updates to the EPRI-SOG seismic sources have been identified. The first is a modification of the EPRI-SOG characterization of the Charleston region to account for the occurrence of repeated large-magnitude earthquakes. The second is consideration of the postulated ECFS as an additional seismic source. The characterization of these two sources is described in the following subsections.

2.5.2.4.1.1.1 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 2.5.2-230). The source for repeated large earthquakes at Charleston is modeled by the four alternatives shown on Figure 2.5.2-212. Earthquakes are modeled to occur as extended ruptures on closely spaced vertical faults oriented parallel to the long dimension of each source (Reference 2.5.2-230). 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 2.5.2-231).

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 2.5.2-213 (Reference 2.5.2-230). Two estimates of the frequency of the repeated large earthquakes were provided: one based on the number of earthquakes identified from paleoliquefaction data for the past 2000 years and one based on the number of earthquakes identified from paleoliquefaction data for the past 5000 years, with relative weights of 0.8 and 0.2, respectively (Figure 2.5.2-213).

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. (Reference 2.5.2-232, Reference 2.5.2-233). The magnitudes listed on Figure 2.5.2-213 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 expert teams (Reference 2.5.2-230). However, these sources are typically 250 km (150 mi.) from the HAR site (Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206). Thus the details of their geometry, as it relates to the occurrence of smaller earthquakes, are not important to the hazard assessment for the HAR site. Accordingly, a simpler approach was adopted for incorporating the updated Charleston seismic source into the PSHA for the HAR site. The alternative seismic source geometries shown on Figure 2.5.2-212 were used to model 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 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206, and the modified maximum magnitude distributions are listed in the right-hand column of the tables. Subsection 2.5.2.4.3.2.1 provides further details.

2.5.2.4.1.1.2 Postulated East Coast Fault System

The ECFS had not been postulated as a specific tectonic feature at the time of the EPRI-SOG study in the late 1980s, and therefore was not considered as a distinct source by the EPRI-SOG teams. The location of the postulated fault system is shown on Figure 2.5.2-214. Evaluation of the postulated ECFS indicates that evidence of the existence and potential activity of the source is low (see discussion in Subsection 2.5.1.1.4.2.5.2.2). Wheeler classified the postulated ECFS as a Class C feature, that is, one for which “geologic evidence is insufficient to demonstrate (1) the existence of tectonic faulting, or (2) Quaternary slip or deformation associated with the feature” (Reference 2.5.2-234). Only the southern portion of the southern segment is included in the National Seismic Hazard Mapping Project as an alternative seismic source for repeated large-magnitude events in the Charleston region (Reference 2.5.2-227). The southern ZRA, which was the extent of the postulated ECFS at the time of the TIP project, was considered a possible alternative Charleston seismic source geometry (Reference 2.5.2-225). The central and northern segments of the postulated ECFS were not included as Quaternary potentially active faults in the 2002 USGS hazard model, and they are not included in the 2007 update.

Following Regulatory Guide 1.208, an evaluation of the evidence supporting this more recent hypothesis was undertaken to assess whether this postulated fault system qualifies as a capable tectonic source, and thus a new seismic source that should be included in an updated seismic source model for the region. This assessment pertains only to the central and northern segments (ECFS-C and ECFS-N). Following the updated seismic source characterization for repeated large-magnitude earthquakes in the Charleston region that was developed by SNC, the southern segment of the postulated fault zone (ECFS-S) is included with low weight (0.1) as a possible alternative fault source for the repeated large-magnitude earthquakes near Charleston (see discussion in Subsection 2.5.2.4.1.1.1) (Reference 2.5.2-230).

The assessments for the postulated ECFS-C and postulated ECFS-N segments are based on criteria that were used by the EPRI-SOG teams to evaluate the seismogenic potential of known or suspected tectonic features. The criteria that were judged to be the most important in the EPRI-SOG evaluations included 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 are

applied to the postulated ECFS-C segment, which lies closest to the HAR site, as discussed in the following paragraphs:

- **Association with seismicity.** The postulated ECFS-C and ECFS-N are not associated with alignments of instrumental seismicity and no historical events appear to be spatially associated with these segments (Figure 2.5.2-214). As described in Subsection 2.5.1.1.4.2.5.2.2, no paleoliquefaction features have been associated with these segments of the postulated fault system.

- **Deep crustal expression.** Basement faults inferred to underlie Coastal Plain sediments in North Carolina are identified by Lawrence and Hoffman on the basis of truncated magnetic anomalies, offset anomalies, apparent drag folds, and sudden changes in magnetic anomaly shapes, patterns, or overall magnetic level along prominent lineaments (Reference 2.5.2-235). The location of the Hollister fault, which is included in the Eastern Piedmont fault system, is similar to but differs slightly from the location shown by Hatcher et al. (Reference 2.5.2-236). The postulated ECFS-C segment does not coincide with any of the basement structures identified by Lawrence and Hoffman (Figure 2.5.1-213). In addition, the postulated fault crosscuts geologic units, structure contours on the top of basement, and locally diabase dikes as mapped by Lawrence and Hoffman with no apparent offset (Reference 2.5.2-235). The postulated fault therefore does not appear to have deep crustal expression or continuity on a regional extent. The postulated fault is not expressed as a throughgoing feature in the gravity or magnetic data (Figures 2.5.1-228 and 2.5.1-229). As described in Subsection 2.5.1.1.4.2.5.2.2, an apparent down-to-the-east escarpment in the top of basement rock along the postulated ECFS-C near Bailey, North Carolina, that Marple and Talwani (Reference 2.5.2-237) describe as a fault scarp or tectonic flexure is interpreted by the original source publication (Reference 2.5.2-238) to be a shoreline erosional feature related to a Pliocene period of high sea level. There is no independent evidence to support the existence of the postulated ECFS as a throughgoing basement structure.

- **Geometry and sense of slip relative to the present stress regime.** Marple and Talwani state that the north-northeast trend of the postulated ECFS favors oblique right-lateral strike-slip movement with respect to the east-northeast-oriented maximum horizontal compressive stress field in the eastern United States. It is suggested that the southwestward deflection of the Neuse River along the ZRA-C (i.e., postulated ECFS-C) and the linear nature of its segments are evidence for strike-slip reactivation, but acknowledged that more data are needed to confirm oblique strike-slip (Reference 2.5.2-237). It is noted that a Jurassic diabase dike mapped by Lawrence and Hoffman (Reference 2.5.2-235) crosses the projected trend of the postulated ECFS-C at the southwest deflection of the Neuse River with no apparent offset, suggesting that there has not been significant right-lateral movement at that location since emplacement of the dike.

- **Evidence for brittle slip and Quaternary deformation.** Geomorphic anomalies and geologic evidence cited in support of the existence of the postulated ECFS-C as a Quaternary active tectonic structure are not definitive. Cretaceous and post-Cretaceous faults identified in the Piedmont physiographic province are not well aligned along the postulated ECFS, but rather are distributed across a 50-km (35-mi.) wide zone (Figure 2.5.1-213). As summarized in Table 2.5.1-201, the faults are generally identified at single locations, exhibit variable orientations that are generally more northerly than the inferred northeast-trending postulated ECFS-C, and have small displacements in Cretaceous to Pliocene age deposits. None of these faults has definitive evidence of Quaternary displacement (Table 2.5.1-201).

The interpretation of the inferred river anomalies as neotectonic features is questionable. Weems (Reference 2.5.2-239) does not identify anomalies in the river profiles that would coincide with the central segment of the central zone of river anomalies that Marple and Talwani cite as evidence for the existence and activity of the postulated ECFS-C (Figure 2.5.1-213) (Reference 2.5.2-237). A recent unpublished compilation map by Dr. Weems showing possible neotectonic features in the Coastal Plain includes only the postulated ECFS-S as a possible neotectonic feature.

Longitudinal profiles of drainages that cross the postulated ECFS-C developed using recently acquired LIDAR data do not indicate a consistent pattern of vertical deformation. The Cape Fear River profile exhibits an inflection at the location of the postulated ECFS-C, and apparent changes in the long-term uplift of late Quaternary terraces (e.g., the 130 to 70 ka Wando terrace) and postulated evidence for Holocene incision along the Cape Fear River are the most definitive evidence for Quaternary deformation cited by Marple and Talwani (Reference 2.5.2-237). However, comparable scale anomalies in the longitudinal profiles of rivers to the north and south are not present (Table 2.5.1-202) and the longitudinal profiles of several rivers constructed from the LIDAR data show no evidence of vertical deformation across the trend of the postulated ECFS-C (e.g., Figure 2.5.1-220, G-G′ — Neuse River, I-I′ — Buffalo Creek, J-J′ — Little River, L-L′ — Toisnot Swamp, O-O′ — Lumber River, and P-P′ — South River). Possible Quaternary deformation that may be indicated by the Cape Fear River anomalies are not clearly associated with a northeast-trending structure. Alternatively, the deformation may be related to the northwest-trending Cape Fear arch or a localized flexure on the flank of the arch (see discussion in Subsection 2.5.1.1.4.2.5.2.2).

Based on these observations, the interpretation of the postulated ECFS-C as a throughgoing basement fault is not reasonably supported by the existing data. Dominion also concluded that the postulated ECFS-N probably does not exist or has a very low probability of activity if it does exist (Reference 2.5.2-240). The NRC staff concluded based on review of the Dominion ESP Application that the geologic, seismologic, and geomorphic evidence presented by Marple and Talwani for the postulated ECFS-N is questionable (Reference 2.5.2-237).

Despite the lack of evidence for the existence or activity of these two northern segments, the postulated ECFS-N was considered as a new source in the updated PSHA for the Dominion ESP, albeit with low probability of existence (0.1) and low probability of activity (0.1) (Reference 2.5.2-240). A weight of 0.1 for the existence of this fault is not supported by the available data. However, it is acknowledged that there is uncertainty in the cause of the inferred geomorphic anomalies and, therefore, a sensitivity analysis that includes the ECFS-C and ECFS-N segments as possible seismic sources with the same probability of existence and activity that were approved by the NRC staff for the Dominion ESP site was conducted, as cited in NUREG-1835. The results, discussed in Subsection 2.5.2.4.3.2.2, indicate that the ECFS does have a small contribution to the site hazard at exceedance frequencies of interest.

Source parameters for the postulated ECFS-N and ECFS-C zones are shown on Figure 2.5.2-215. This logic tree shows four parameters: (1) probability of existence, (2) probability of activity, (3) maximum magnitude, and (4) recurrence interval. For this analysis, the postulated ECFS-N and ECFS-C segments are modeled as two fault zones that encompass the ZRA-C and ZRA-N as outlined by Marple and Talwani (Reference 2.5.2-237) (Figure 2.5.1-217). The three segments of the postulated fault show a general en-echelon, right-stepping pattern with the stepover between the southern and central segments being more pronounced than the stepover between the central and northern segments. Within each zone, the geometry is modeled as a series of parallel, vertical faults oriented northeast-southwest and parallel to the long axis of the zones.

For consistency with recent assessments, the maximum magnitude probability distribution used for the postulated ECFS-N and ECFS-C segments that was developed for the updated Charleston seismic source has been adopted: **M** 6.7 (0.1); **M** 6.9 (0.25); **M** 7.1 (0.3); **M** 7.3 (0.25); and **M** 7.5 (0.1) (Reference 2.5.2-230). This assessment follows Dominion (Reference 2.5.2-240), which also used a maximum magnitude distribution based on the magnitude distribution for the Charleston seismic source; in that case the accepted distribution was the same as the maximum magnitude distribution used for the USGS National Seismic Hazard map (Reference 2.5.2-227). It should be pointed out that there is no basis for assigning magnitudes of any size to the postulated ECFS-C and ECFS-N segments as the postulated features are not associated with definable tectonic structure and there is no associated seismicity. The earthquake repeat times shown on Figure 2.5.2-215 are applied to both the central and northern segments. The probabilities of existence and activity are assumed to be correlated for the two segments: either both exist as active sources or both do not.

The recurrence values and weights also are those used by Dominion, with the exception that the minimum recurrence interval, which was 550 years based on the recurrence of repeated large-magnitude, Charleston-type events, has been changed to 5000 years (Reference 2.5.2-240). There is no geologic or seismologic data to support a 550-year repeat time for large-magnitude events on the postulated ECFS-C or ECFS-N segments. A minimum 5000-year repeat

time is more consistent with the geologic evidence for average uplift rates of 0.02 mm/yr to a maximum uplift on the order of 0.08 to 0.2 mm/yr that may be present along the postulated ECFS-C segment (see discussion in Subsection 2.5.1.1.4.2.5.2.2). Using Wells and Coppersmith's regressions of maximum and average displacement on magnitude, expected maximum and average displacements for an **M** 7.1 earthquake are 2.3 and 1.26 m (7.6 and 4.13 ft.) (all slip types) and 1.92 m and 1.17 m (6.3 and 3.84 ft.) (strike-slip), respectively (Reference 2.5.2-231). The recurrence values used in the HAR analysis — 5000 (0.1); 25,000 (0.5); and 50,000 (0.4) — capture the range of expected repeat times for large-magnitude (~**M** 7.1) events that would be generated by the postulated ECFS based on the geologic slip rates inferred from the uplift rates and assumed horizontal to vertical slip ratios of 2 or 3 to 1. It should be noted that, although the postulated ECFS is assumed to be an oblique right-lateral strike-slip fault given its orientation in the contemporary stress field, Marple and Talwani provide no evidence other than the linear nature of its segments, a deflection of the Neuse River along one stretch of the central segment, and the presence of a low, swampy region in the postulated stepover between the southern and central segments to suggest that there has been a significant component of strike-slip movement along the postulated fault (Reference 2.5.2-237). The apparent absence of significant offset of interpreted Jurassic diabase dikes across the postulated fault in the vicinity of the Neuse River, as shown by Lawrence and Hoffman (Reference 2.5.2-235), suggests that the assumption of significant strike-slip movement may not be valid.

2.5.2.4.1.2 Earthquake Occurrence Rates

Subsection 2.5.2.1.1 describes the development of an updated earthquake catalog for the HAR 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 HAR 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 following relationship:

$$m_b{*} = m_b - \beta\sigma^2_{m_b|m_b\,instrumental}/2 \qquad (2.5.2\text{-}1)$$

when m_b is based on instrumentally recorded m_b magnitudes and by the relationship

$$m_b* = m_b + \beta\sigma^2_{m_b|X}/2 \tag{2.5.2-2}$$

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 2.5.2-1 to positive in Equation 2.5.2-2 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. (Reference 2.5.2-228, Reference 2.5.2-227). 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. The mean difference between the values of m_b* for the updated catalog and those given in the EPRI-SOG catalog for earthquakes within 320 km (200 mi.) of the HAR site is 0.04 magnitude units.

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 2.5.2-216 shows the five completeness regions (3, 6, 7, 12, and 13) that cover the area within 320 km (200 mi.) of the HAR site. Note that completeness region 12 was modified slightly to more completely encompass Charleston. 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

$$T^E_{ij} = \sum_k T_k \times P^D_{ijk} \tag{2.5.2-3}$$

where P^D_{ijk} 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 a number of 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 2.5.2-208.

Most of the newly identified earthquakes within 320 km (200 mi.) of the HAR site occurred in time periods identified in the EPRI-SOG evaluation as periods of incomplete catalog reporting (P^D < 1.0). Comparisons of the earthquake counts for these time periods suggest that inclusion of the newly identified earthquakes in the estimation of catalog completeness would likely yield values of P^D near unity for the period post-1860 for the two lowest magnitude intervals: 3.3 ≥ m_b* > 3.9 and 3.9 ≥ m_b* > 4.5. For example, in completeness region 3, the EPRI-SOG catalog contains two 3.3 ≥ m_b* > 3.9 earthquakes for the time period of 1860 to 1910, while the updated catalog without EPRI-SOG aftershocks contains 10, indicating a fivefold increase in the observed earthquake frequency for that time period. The probability of detection P^D for a given time interval is approximately proportional to the ratio of the observed earthquake frequency in that time interval to the earthquake frequency in time intervals of complete reporting. Therefore, a factor-of-five increase in the observed frequency of earthquakes suggests an equivalent increase in the level of catalog completeness for this time interval.

Reassessment of the probabilities of detection obtained in the EPRI-SOG study would require systematically updating the earthquake catalog for the entire CEUS and perhaps redefining the completeness regions (Reference 2.5.2-201). Instead, the process of using the ratio of earthquake counts in the two catalogs described in the previous paragraph was used to provide an approximate assessment for the local region around the HAR site. Updated estimates of P^D were obtained by multiplying the value of P^D reported in EPRI-SOG (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 revised assessments are presented in Table 2.5.2-208 for completeness regions 3, 6, 7, and 12. Completeness region 13 could not be analyzed because it contains only one event of m_b* ≥ 3.3 in the EPRI-SOG catalog and none in the updated catalog.

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 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 individual completeness regions are compared on Figure 2.5.2-217. Note that the observed earthquake frequencies labeled “EPRI-SOG” and “Update (no EPRI-SOG aftershocks)” are plotted slightly offset from their true location for clarity. The computed recurrence models are not offset on the figure. For completeness region 3, the updated catalogs produce somewhat lower occurrence rates. For completeness region 6, all three catalogs produce essentially the same rate. For completeness region 7, the updated catalog produces lower rates for magnitudes < m_b 4.5 and adds a single m_b 4.5 to 5.1 earthquake. As a result, a lower *b*-value is obtained for this region. However, the overall rate for this portion of the 320-km (200-mi.) region is low. For completeness region 12, which contains the HAR site, the updated catalog with all events produces slightly higher rates than the EPRI-SOG catalog. However, when the EPRI-SOG flagged aftershocks are removed, the updated catalog produces similar rates to those obtained from the EPRI-SOG catalog.

Figure 2.5.2-218 shows the combined earthquake occurrence rates calculated for the area within 320 km (200 mi.) of the HAR site. The earthquake rates were computed as a composite rate for the entire region using the catalog completeness values for each portion of the EPRI-SOG completeness regions that lie within 320 km (200 mi.) of the site. As was described previously for each completeness region, composite occurrence rates were computed using the EPRI-SOG catalog and completeness values and using the updated catalog and completeness values. The updated catalog with all events produces a slightly lower *b*-value (0.871) than the EPRI-SOG catalog (0.902), resulting in slightly higher rates for large-magnitude earthquakes. However, removal of the EPRI-SOG flagged aftershocks results in a *b*-value of 0.889 and overall rates that are similar to or slightly lower than those obtained from the EPRI-SOG catalog.

Based on comparisons shown on Figures 2.5.2-217 and 2.5.2-218, the earthquake occurrence rate parameters developed in the EPRI-SOG evaluation adequately represent the seismicity rates within 320 km (200 mi.) of the HAR site based on more recent information. The only exception is the occurrence rates for large earthquakes associated with the Charleston seismic source. Updated estimates of the occurrence rates for these earthquakes based on paleoliquefaction data are described in Subsection 2.5.2.4.1.1.1 as part of the UCSS model.

2.5.2.4.1.3 Seismic Source Maximum Magnitudes

The two major sources of seismic hazard at the HAR site are the local or host seismic source and sources related to repeated earthquakes near Charleston. The assessment of maximum magnitudes for the repeated earthquakes near Charleston was updated to represent current information as part of the UCSS model developed by SNC, and those assessments were used for the analysis of the HAR site (Reference 2.5.2-230). Information on the maximum magnitude for the local or host seismic sources is reviewed in the following paragraphs.

The EPRI-SOG source zones that contain the HAR site are indicated in Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206. These sources are either broad background or default source zones or the local Mesozoic basin in which the site lies. Figure 2.5.2-219 shows the maximum magnitude distributions for these sources. The top plot shows the composite of the distribution developed by the EPRI-SOG (Reference 2.5.2-201) expert teams in terms of the m_b magnitude scale, the magnitude scale used in the EPRI-SOG seismic hazard model. The bottom plot compares the composite EPRI-SOG maximum magnitude distribution to more recent assessments. This comparison is made in terms of the moment magnitude scale, **M**. The composite m_b distribution was converted to moment magnitude using three equally weighted m_b-**M** relationships:

1. By Atkinson and Boore (Reference 2.5.2-241):

$$\mathbf{M} = -0.39 + 0.98 m_b \qquad \text{for } m_b \le 5.5$$

$$\mathbf{M} = 2.715 - 0.277 m_b + 0.127 m_b^2 \qquad \text{for } m_b > 5.5 \qquad (2.5.2\text{-}4)$$

2. By Johnston (Reference 2.5.2-242):

$$\mathbf{M} = 1.14 + 0.24 m_b + 0.0933 m_b^2 \qquad (2.5.2\text{-}5)$$

3. By EPRI (Reference 2.5.2-243):

$$m_b = -10.23 + 6.105\mathbf{M} - 0.7632\mathbf{M}^2 + 0.03436\mathbf{M}^3 \qquad (2.5.2\text{-}6)$$

The transformed composite EPRI-SOG maximum magnitude distributions are compared to assessments by Savy et al., Frankel et al., and SC DOT (Reference 2.5.2-225, Reference 2.5.2-228, Reference 2.5.2-227, Reference 2.5.2-229).

The EPRI-SOG expert teams developed a broad uncertainty distribution for maximum magnitude for the sources that contain the HAR site. When transformed into moment magnitude, this distribution spans the same range as more recent assessments of the maximum magnitude.

In contrast, Frankel et al. assigns a single value of **M** 7.5 to the entire extended crust region shown on Figure 2.5.2-210 (Reference 2.5.2-228, Reference 2.5.2-227). The discussion presented in Frankel et al. indicates that this value was motivated by the assessed magnitude for the 1886 Charleston earthquake and the interpretation that “such a large event could not be ruled out in other areas of the extended crust” (Reference 2.5.2-228). The implication is that the single value of **M** 7.5 does not represent an assessment of the uncertainty on the upper limit of earthquake size that could occur in the vicinity of

the HAR site or within specific seismic source zones that contain the HAR site, but rather an assessment of the upper end of such a distribution. Frankel et al. do not use source zones in their PSHA calculations and are making an assessment for the entire extended crust rather than the specific region around the HAR site (Reference 2.5.2-228, Reference 2.5.2-227). As shown on Figure 2.5.2-219, the interpretations of the EPRI-SOG expert teams included the possibility that earthquakes of similar size to the 1886 Charleston earthquake could occur in the site host sources.

SC DOT also assigns a single value of **M** 7 for the maximum magnitude for seismic sources that contain the HAR site, but this report does not provide a basis for the assessment (Reference 2.5.2-229).

Savy et al. do develop an uncertainty distribution for maximum magnitude for sources representative of the HAR site region (Reference 2.5.2-225). The distribution of Savy et al. is also broad, but somewhat more bimodal than the composite EPRI-SOG distribution (Reference 2.5.2-225).

The comparisons on Figure 2.5.2-219 indicate that more recent assessments of maximum magnitude have tended to place more weight on higher magnitudes than the EPRI-SOG expert teams. However, with the exception of the repeated large earthquakes at Charleston, no large historical or prehistoric earthquakes have been identified in the EPRI-SOG seismic sources that include the site that would provide evidence for larger maximum magnitudes than the assessments of the expert teams, and the composite EPRI-SOG maximum magnitude distribution does span the range of more recent assessments. Therefore, the EPRI-SOG maximum magnitude assessments for these sources are judged to be appropriate for use in PSHA calculations for the HAR site.

The maximum magnitude distributions for the EPRI-SOG seismic sources were reviewed against the updated earthquake catalog. The minimum values for a few of these distributions (sources 107 and 217 defined by Law Engineering and sources C18 and 103 defined by Weston Geophysical) were adjusted to be consistent with the largest observed earthquake in these sources (e.g., changing the low-weighted lower value of m_b 4.9 to m_b 5.0 or limiting the minimum maximum magnitude to m_b 5.7 for sources that contain the 1897 Giles County earthquake).

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 2.5.2-244), Boore and Atkinson (Reference 2.5.2-245), and Nuttli (Reference 2.5.2-246) combined with the response spectral relationships of Newmark and Hall (Reference 2.5.2-247). 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 has completed a study to characterize the estimation of strong ground motion in the CEUS for application in PSHA for nuclear facilities (Reference 2.5.2-248). This study was conducted following the SSHAC guidelines for a Level III analysis (Reference 2.5.2-226). SSHAC provided guidance on the appropriate methods to use for quantifying uncertainty in evaluations of seismic hazard (Reference 2.5.2-226). 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 2.5.2-248). 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 2.5.2-248). Three of these ground motion clusters are appropriate for use in assessing the hazard from moderate-size 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 2.5.2-248) proposed the logic tree structure to be used with these models that is shown on the left-hand side of Figure 2.5.2-220. 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 earthquake sources. For the HAR site, these sources are Charleston-related sources (those defined in both the EPRI-SOG model, listed in Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206, 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 2.5.2-248). As shown by the third level of the logic tree (Figure 2.5.2-220), 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 2.5.2-248) median ground-motion models for clusters 1, 2, and 3 were based in large part on the CEUS ground-motion models developed by Silva et al. (Reference 2.5.2-249), Atkinson and Boore (Reference 2.5.2-241), and Campbell (Reference 2.5.2-272), respectively. Silva et al. (Reference 2.5.2-250) and Atkinson and Boore (Reference 2.5.2-251) have since developed updated versions of their models. In addition, Tavakoli and Pezeshk (Reference 2.5.2-273) present a hybrid ground-motion model for the CEUS based on the approach developed by Campbell (Reference 2.5.2-272).

These newer models are compared to the EPRI (Reference 2.5.2-248) models on Figure 2.5.2-221. The two plots on the left compare the EPRI (Reference 2.5.2-248) 5th percentile, 50th percentile, and 95th percentile 10-Hz (top) and 1-Hz (bottom) median models for ground motion cluster 1 with the three single-corner stochastic models developed by Silva et al. (Reference 2.5.2-250). The two plots in the center of Figure 2.5.2-221 compare the EPRI (Reference 2.5.2-248) 5th percentile, 50th percentile, and 95th percentile 10-Hz (top) and 1-Hz (bottom) median models for ground motion cluster 2 with the model developed by Atkinson and Boore (Reference 2.5.2-251). The Atkinson and Boore (Reference 2.5.2-251) model uses rupture distance as the distance measure, while the EPRI (Reference 2.5.2-248) cluster 2 models use Joyner-Boore distance. The comparisons shown on Figure 2.5.2-221 were made assuming that the top of rupture for the **M** 5 earthquake is at a depth of 2.5 mi. (4 km), based on a mean point-source depth of 3.7 mi. (6 km) (Reference 2.5.2-250). The median ground motions produced by the updated Atkinson and Boore (Reference 2.5.2-251) model fall within the range of the EPRI (Reference 2.5.2-248) cluster 2 medians except for distances less than about 4.3 mi. (7 km) for large-magnitude earthquakes. The two plots on the right of Figure 2.5.2-221 compare the EPRI (Reference 2.5.2-248) 5th percentile, 50th percentile, and 95th percentile 10-Hz (top) and 1-Hz (bottom) median models for ground motion cluster 3 with the model developed by Tavakoli and Pezeshk (Reference 2.5.2-273). The Tavakoli and Pezeshk (Reference 2.5.2-273) model predictions generally fall within the range of the EPRI (Reference 2.5.2-248) cluster 3 medians except for the 1-Hz estimates for small magnitudes at short rupture distances.

As presented in Subsection 2.5.2.4.4, large-magnitude earthquakes at very small distances have a very small contribution to the hazard. Also, small-magnitude earthquakes at close distances have a small contribution to the low-frequency hazard. On the basis of the comparisons shown on Figure 2.5.2-221, it is concluded that the EPRI (Reference 2.5.2-248) median ground-motion models are appropriate for use in computing the hazard for the HAR site.

2.5.2.4.2.2 Models for Ground Motion Aleatory Variability

The EPRI study also provided a characterization of the aleatory variability in CEUS ground motions based on an assessment of information available at the time (Reference 2.5.2-248). More recently, EPRI conducted a study focused in part on evaluating the appropriate aleatory variability for CEUS ground motions (Reference 2.5.2-252). 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 2.5.2-252).

The EPRI (Reference 2.5.2-252) 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 2.5.2-220. The fourth level of the logic tree addresses the overall aleatory model. Two alternatives were defined: (1) model 1A is based on western United States (WUS) aleatory variability with an additional component of intra-event variability for CEUS earthquakes and Model 1B is unmodified WUS aleatory variability, and (2) 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 2.5.2-248). EPRI (Reference 2.5.2-252) evaluated the empirical evidence for additional aleatory variability at small Joyner-Boore distances and concluded that the adjustments proposed by EPRI (Reference 2.5.2-248) 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 2.5.2-220. 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 2.5.2-248).

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 2.5.2-220 addresses the relationship between body-wave magnitude, m_b, and moment magnitude, **M**. This conversion is required because the EPRI (Reference 2.5.2-248, Reference 2.5.2-252) 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 described in Subsection 2.5.2.4.1.3. 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 Subsections 2.5.2.4.1 and 2.5.2.4.2, 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 HAR site:

- Review of the contributing EPRI-SOG seismic sources.
- Sensitivity to new data relative to the occurrence of large earthquakes in the Charleston, South Carolina, region.
- Sensitivity to the postulated ECFS as a new seismic source.

Sensitivity analyses were not conducted to address the effect of the updated ground motions models developed by EPRI (Reference 2.5.2-248, Reference 2.5.2-252) because these have become the standard set of models for the assessment of seismic hazards for proposed new power plants.

2.5.2.4.3.1 Review of EPRI-SOG Seismic Sources

The EPRI PSHA calculations did not use all of the seismic sources defined by each EPRI-SOG expert team, but rather subsets for each team that in aggregate produced 99 percent or more of the hazard that would be calculated from the complete set for that team (Reference 2.5.2-202). The selection of these subsets was made using the ground motion characterization in use at that time. As part of the evaluations for the HAR site, the selection of the appropriate subset of sources was examined using the updated ground motion models that will be used to compute the PSHA for the HAR site, EPRI (Reference 2.5.2-248, Reference 2.5.2-252). This examination was performed by calculating the mean hazard produced by each of the seismic sources listed in Tables 2.5.2-201, 2.5.2-202, 2.5.2-203, 2.5.2-204, 2.5.2-205, and 2.5.2-206, weighted by the probability that the source is included in the hazard model, P*. The sources examined include those used in the EPRI calculation (Reference 2.5.2-202) and other sources that extend to within 320 km (200 mi.) of the HAR site. 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 2.5.2-241).

2.5.2.4.3.1.1 Bechtel Team Seismic Sources

Figure 2.5.2-222 shows the mean hazard curves computed for the Bechtel team's sources listed in Table 2.5.2-201. The results indicate that the additional sources contribute slightly more than 1 percent to the 1-Hz hazard at a mean annual exceedance frequency of 10^{-4}. The additional contribution is primarily from Bechtel sources 19, 24, and E related to Giles County, the Bristol Block, and the Central Virginia seismic zone, respectively. These three sources were added to the EPRI source set for computation of the HAR site hazard (Reference 2.5.2-202).

2.5.2.4.3.1.2 Dames & Moore Team Seismic Sources

Figure 2.5.2-223 shows the mean hazard curves computed for the Dames & Moore team's sources listed in Table 2.5.2-202. The results indicate that the additional sources contribute more than 1 percent to the 10-Hz and 1-Hz hazard for mean annual exceedance frequencies in the range of 10^{-4} to 10^{-5}. The additional contribution is primarily from Dames & Moore sources 4 and 4A, 4B, 4C, and 4D related to the East Tennessee seismic zone and Giles County. These sources were added to the EPRI source set for computation of the HAR site hazard (Reference 2.5.2-202).

Review of the Dames & Moore team's source zone combinations used in the EPRI (Reference 2.5.2-202) calculations for the HAR site indicate that there is a weight of 0.74 placed on the case where there is no active seismic source in the vicinity of the site. This arises from the Dames & Moore team's specification that source 53, which contains the HAR site, is a default for source 52, the Charleston Mesozoic Rift, or the mutually exclusive alternative sources 47 through 51, the Mesozoic/Cenozoic basins (Reference 2.5.2-220). The HAR site does lie in the basin source 49. However, as there are no m_b* 3.3 or larger earthquakes contained within source 49, the computed seismicity rate was zero and it was not included in the EPRI (Reference 2.5.2-202) seismic source set.

A similar situation exists for source 41, the Southern Cratonic Margin source. This source is specified as a default for several small fault-specific sources that lie at large distances from the HAR site. The assigned probability of activity for this source, P^A, is 0.12, which, when used as P*, results in the source being present in the calculation with a weight of 0.12.

Review of the overall approach used by the Dames & Moore team suggests that an alternative interpretation is possible. Dames & Moore (Reference 2.5.2-220)

state the following in regard to their use of the probability of activity parameter, P^A:

> …our P_A estimates state our team's confidence that tectonics can explain earthquake occurrence and (1-P_A) estimates that seismicity can explain the occurrence of future events.

Dames & Moore further state (Reference 2.5.2-220):

> …it turned out that when source zones were constructed, keeping in mind our use of P_A and (1-P_A), the entire study region was covered by "source zones."

This approach is illustrated in Dames & Moore's description of the Charleston seismic source (source 54), which is assigned a P^A of 0.7 (Reference 2.5.2-220):

> This means that the Charleston seismic zone has a 70 percent chance of being the correct zone for generating a moderate to large earthquake. The remainder of 30 percent is applied to a source zone of the same geometry, but with different seismicity parameters (as described in the next section).

Given these statements, one could interpret the P^A values of 0.26 and 0.12 assigned to sources 53 and 41, respectively, as assessments of a tectonic basis for their activity as a seismic source and the sources would be included in the hazard as seismicity-based sources with weight 1-P^A. There is also the question of the location of historical earthquakes with estimated magnitudes near m_b 5, the basic level of seismic potential used to define the EPRI-SOG seismic sources (Reference 2.5.2-202). Source 41 contains the 1913 earthquake and the EPRI-SOG catalog locations for the 1817, 1861, and 1916 earthquakes (see Subsection 2.5.2.1.2 for a description of these events). Subsequent catalogs have placed these earthquakes either in the East Tennessee seismic zone (the 1861 and 1916 events) or near Charleston (the 1817 event). As discussed in Subsection 2.5.1.2, the location of the 1817 earthquake is uncertain and it was felt at widely scattered locations.

The possible alternative interpretation of the treatment of Dames & Moore's seismic source 53 is to consider it to be present as a seismic source in its entirety with P^A weight of 0.26 and to exist as a seismicity source with weight 1-P^A for those portions that are not contained in the alternative sources. A similar interpretation can be applied to source 41. These alternative interpretations result in a two- to threefold increase in the mean hazard at the HAR site computed for the Dames & Moore team's seismic sources, resulting in hazard estimates that are similar to those obtained for the other five expert teams. Because of the large effect on the computed hazard for the Dames & Moore team and to account for the uncertain location of past ~m_b events, this alternative interpretation was adopted for the HAR PSHA.

2.5.2.4.3.1.3 Law Engineering Team Seismic Sources

Figure 2.5.2-224 shows the mean hazard curves computed for the Law Engineering team's sources listed in Table 2.5.2-203. The only additional source in the site region is source 217, which is an alternative to source 17 with a lower maximum magnitude distribution. This source has slightly less than one percent contribution to the hazard at the HAR site. However, it does account for the occurrence of several m_b 5 or larger earthquakes in the 320-km (200-mi.) region around the HAR site when source 17 is not active, notably the 1897 Giles County m_b 5.7 earthquake; source 217 was included as the alternative to source17 for completeness.

Review of the Law Engineering team's seismic source combinations indicated an issue of interpretation that is somewhat analogous to that described previously for the Dames & Moore team. As described in EPRI (Reference 2.5.2-253), the Law team source 107 that contains the HAR site was treated as a background source, with the hazard from this source multiplied by the factor $P^B = 0.42$, the weighted average of the source being active. Law source 17 also contains the site and is included in the hazard with weight $P^* = 0.27$. The effect of this treatment is to assign an effective weight of $(1-0.42) \times (1-0.27) = 0.423$ to no active source in the site region and a weight of $0.42 \times 0.27 = 0.11$ to double counting of the local seismicity with sources 107 and 22 both present. Note that the third alternative Law Engineering source for the eastern seaboard, Mesozoic basins source 8, has very little overlap with source 107 and does not encompass the HAR site. In addition, source 107 contains the 1913 earthquake with an estimated magnitude of m_b 5.0 in the updated catalog.

To address the issue of the lack of a seismic source to account with weight of 1.0 for a m_b 5 earthquake within 320 km (200 mi.) of the site region, as well as the small degree of double counting of seismicity, an alternative logic structure was developed for the Law Engineering team. In this model, the seismic rates for source 107 were unmodified, but source 107 is present only when source 22 is not — in effect a modified P^* of 0.73 with the two sources treated as mutually exclusive. The effect of this modification is to increase the mean annual frequency of exceedance by about 16 percent for 10-Hz spectral acceleration and by less than 5 percent for 1-Hz spectral accelerations. Because this alternative interpretation accounts for all m_b 5 earthquakes within 320 km (200 mi.) of the HAR site and it produces an increase in the hazard, it was included in the updated PSHA for the HAR site.

2.5.2.4.3.1.4 Rondout Associates Team Seismic Sources

Figure 2.5.2-225 shows the mean hazard curves computed for the Rondout Associates team sources listed in Table 2.5.2-204. The results indicate that the additional sources contribute slightly more than 1 percent to the 10-Hz and 1-Hz hazard at mean annual exceedance frequencies in the range of 10^{-4} to 10^{-5}. This additional hazard was distributed among the four sources and all were added to

the EPRI source set for computation of the HAR site hazard (Reference 2.5.2-202).

2.5.2.4.3.1.5 Weston Geophysical Team Seismic Sources

Figure 2.5.2-226 shows the mean hazard curves computed for the Weston Geophysical team's sources listed in Table 2.5.2-205. The results indicate that the additional sources 23 and 24, Giles County and the New York – Alabama – Clingman lineaments, respectively, contribute more than 1 percent to the 10-Hz and 1-Hz hazard at mean annual exceedance frequencies in the range of 10^{-4} to 10^{-5}. This additional hazard was distributed among the four sources and all were added to the EPRI source set for computation of the HAR site hazard (Reference 2.5.2-202). These two sources are contained within Weston Geophysical's Southern Appalachian source 103, and the Weston Geophysical team created three complementary sources, C17, C18, and C19, to be used in place of source 103 when either source 23 or source 24, or both, were active. The EPRI source set for the Weston team included only source C19 and not the interior sources 23 and 24 (Reference 2.5.2-202). The full set of alternatives for these sources was included in the PSHA for the HAR site.

2.5.2.4.3.1.6 Woodward-Clyde Consultants Team Seismic Sources

Figure 2.5.2-227 shows the mean hazard curves computed for the Woodward-Clyde Consultants team's sources listed in Table 2.5.2-206. The results indicate that the additional source 31A, associated with the East Tennessee seismic zone and Giles County, contributes about 5 percent to the 1-Hz hazard, and this source was added to the EPRI source set for the Woodward-Clyde Consultants team (Reference 2.5.2-202). Portions of source 31A are contained within the Woodward-Clyde Consultants background source, but the larger maximum magnitudes for source 31A produce higher 1-Hz hazard than the seismicity from the same region within the background source. To prevent double counting of the seismicity within source 31A, an alternative version of the HAR site background was created that excludes the area within source 31A. This alternative background was used when source 31A was included in the hazard analysis.

2.5.2.4.3.1.7 Summary of Modifications to EPRI-SOG Seismic Sources

The review of the EPRI-SOG seismic sources and the sensitivity tests discussed previously resulted in the following modifications to the EPRI-SOG seismic sources as implemented in the EPRI calculations for the HAR site (Reference 2.5.2-202):

- The addition of a one or more seismic source for each team that contributed more than 1 percent to the mean annual frequency of exceedance in the range of 10^{-4} to 10^{-5} when calculations were performed using the preferred set of ground motion models for each ground motion cluster as defined by EPRI (Reference 2.5.2-248, Reference 2.5.2-252).

- Modifications to the source combinations and effective values of P* for the Dames & Moore team's source set and to a lesser extent the Law Engineering team's source set to fully account for the occurrence of m_b 5 earthquakes in the updated catalog.

- Adjustments to the minimum maximum magnitudes for a few sources to properly account for the estimated size of the largest earthquake reported in the source based on the updated earthquake catalog.

The effect of these modifications on the composite mean hazard from the six EPRI-SOG expert teams is shown on Figure 2.5.2-228. The mean exceedance frequencies produced by the revised EPRI-SOG seismic sources is approximately 12 percent greater than that computed using the EPRI source implementation in the range of 10^{-4} to 10^{-5} (Reference 2.5.2-202). Although the modifications produced larger increases in the results obtained for some teams, the original results for these teams tended to be lower than the average across all teams. After inclusion of the hazard contribution from the updated Charleston source (Subsection 2.5.2.4.3.2), the differences are less than 10 percent.

2.5.2.4.3.2 Additional Seismic Sources

The second set of sensitivity analyses test the effect of incorporating sources of repeated large-magnitude earthquakes at Charleston and the central and northern segments of the postulated East Coast fault system.

2.5.2.4.3.2.1 Charleston Source of Repeated Large Earthquakes

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 Subsection 2.5.2.4.1.1.1. The four alternative source geometries shown on Figure 2.5.2-212 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 2.5.2-231). 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 2.5.2-213. 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 2.5.2-213 were modeled by the five-point discrete approximation to a continuous distribution developed by Miller and Rice (Reference 2.5.2-254). The discrete recurrence interval values, the associated weights, and the resulting equivalent annual frequencies are listed in Table 2.5.2-209.

Figure 2.5.2-229 compares the hazard computed from the UCSS model with that obtained from the updated EPRI-SOG model (Figure 2.5.2-228) described in Subsection 2.5.2.4.3.1. The UCSS source produces exceedance frequencies for 10-Hz motions that are comparable to 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 HAR site.

As described in Subsection 2.5.2.4.1.1.1, a simplified approach is used to integrate the UCSS model with the EPRI-SOG source models. The approach is to limit the size of earthquakes produced by the Charleston-specific EPRI-SOG sources to magnitudes smaller than those produced by the UCSS. Other sources outside of the immediate Charleston source region are not altered. The effect of this integration approach on the hazard computed from the EPRI-SOG sources is also shown on Figure 2.5.2-229 by the curves labeled "Updated EPRI-SOG with modified M_{max}." The effect is very small for 10-Hz motions, producing about 1 percent or less reduction in the computed frequency of exceedance. For 1-Hz motions, eliminating the double counting of large Charleston earthquakes produces up to an 11 percent reduction in the frequency of exceedance computed from the updated EPRI-SOG sources. However, when considering the effect on the combined hazard produced by the EPRI-SOG and UCSS sources, the reduction in frequency of exceedance is only about 1 percent.

2.5.2.4.3.2.2 Postulated East Coast Fault System — Central and Northern Segments

The seismic source model for the central and northern segments of the postulated ECFS is described in Subsection 2.5.2.4.1.1.2. This source was modeled as a set of closely spaced vertical faults oriented parallel to the strike of the system in a similar fashion to the model described for the UCSS. Like the UCSS, the source is considered to produce only large earthquakes with the distribution for the expected magnitude and average repeat time defined by the logic tree shown on Figure 2.5.2-215. These values are applied individually to both segments of the postulated fault system. The source is assigned a probability of existence of 0.1 and, if it exists, a probability of being active of 0.1.

Figure 2.5.2-229 shows the combined mean hazard computed for the central and northern segments of the postulated ECFS. The postulated source has less than a 1 percent contribution to the 10-Hz hazard and a 1 to 2 percent contribution to the 1-Hz hazard for annual frequencies of exceedance in the range of 10^{-5} to 10^{-6}.

2.5.2.4.3.2.3 Summary

The sensitivity analyses clearly show that the updated characterization of repeated earthquakes near Charleston should be incorporated into updated

PSHA for the HAR site. The assessment of the postulated ECFS is somewhat speculative. On the one hand, as described in Subsection 2.5.2.4.1.1.2, the evidence for the source's existence and potential activity is very weak. On the other hand, NRC staff stated in the Final Safety Evaluation Report for the North Anna ESP site that although the evidence for the existence and activity of the northern segment of the postulated ECFS is low, the postulated ECFS-N should be included as a possible contributor to the seismic hazard for the ESP site (Reference 2.5.2-240). As modeled, postulated ECFS does have a minor contribution to the HAR site hazard in the exceedance frequency range of interest. Therefore, it was included in the updated PSHA for the HAR site.

2.5.2.4.4 PSHA for the HAR Site

The PSHA for the HAR site was conducted using the updated EPRI-SOG seismic sources described in Subsection 2.5.2.4.3.1 combined with the UCSS and postulated ECFS sources described in Subsection 2.5.2.4.3.2. Earthquake ground motions were modeled using the median ground motion models developed by EPRI and the ground motion aleatory variability models developed by EPRI (Reference 2.5.2-248, Reference 2.5.2-252). The logic tree defining the epistemic uncertainty in the ground motion characterization is shown on Figure 2.5.2-220. The hazard 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 2.5.2-220. Conversion of the moment magnitude estimates for the size of the repeated earthquakes associated with the UCSS and the postulated ECFS 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 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 (Reference 2.5.2-241).

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 2.5.2-248). 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 2.5.2-252). 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 2.5.2-248). 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 fps). The effect of the sediments overlying this generic rock condition on defining the hazard at other locations is addressed in 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 (PGA) (Reference 2.5.2-248). Figures 2.5.2-230, 2.5.2-231, 2.5.2-232, 2.5.2-233, 2.5.2-234, 2.5.2-235, and 2.5.2-236 show the resulting mean hazard curves and the 5^{th}, 16^{th}, 50^{th} (median), 84^{th}, and 95^{th} fractile hazard curves for each ground motion measure. These values are listed in Tables 2.5.2-210, 2.5.2-211, 2.5.2-212, 2.5.2-213, 2.5.2-214, 2.5.2-215, and 2.5.2-216. At low spectral frequencies (≤ 1 Hz), the mean hazard approaches or exceeds the 84^{th} 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 2.5.2-221).

Figure 2.5.2-237 shows the contribution of the three source types to the mean hazard for 10-Hz and 1-Hz spectral acceleration. As was found in the sensitivity test described in Subsection 2.5.2.4.3.2, the UCSS produces comparable hazard to that obtained from the updated EPRI-SOG sources for 10-Hz motions, and dominates the hazard for 1-Hz motions. The postulated ECFS produces a very minor contribution to the frequency of exceeding 1-Hz ground motions.

Figure 2.5.2-238 shows the effect of the alternative ground motion cluster models on the mean hazard, respectively. As described in 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 assigned 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 2.5.2-239 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 indicated that the alternative aleatory variability models developed by EPRI produced similar hazard (Reference 2.5.2-252).

Figure 2.5.2-240 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 for the 1-Hz hazard results when the results are 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 2.5.2-213 are always used for ground motion estimation.

Figure 2.5.2-241 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 Spectra for Generic CEUS Rock and Identification of Controlling Earthquakes

The mean hazard results listed in Tables 2.5.2-210, 2.5.2-211, 2.5.2-212, 2.5.2-213, 2.5.2-214, 2.5.2-215, and 2.5.2-216 were interpolated to obtain 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 2.5.2-242 and listed in Table 2.5.2-217.

Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246 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 shape 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 mean log distance, where the weights are defined by the relative contributions to the total hazard for each magnitude and distance interval. Table 2.5.2-218 lists the mean magnitudes and mean log 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 (60 mi.), consistent with the procedure outlined in Appendix D of Regulatory Guide 1.208.

The approach to be used to compute the effects of the HAR site sediments on the generic hard rock motions is Approach 2B for site response analyses described in NUREG/CR-6728 (Reference 2.5.2-255). 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. The site response uses as input ground motions appropriate for the HF and LF reference or controlling earthquakes.

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 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. For the HAR site, the DEL, DEM, and DEH magnitude-distance values were defined to represent the modes in the magnitude-distance deaggregation. As shown on Figures 2.5.2-243, 2.5.2-244, 2.5.2-245, and 2.5.2-246, 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 2.5.2-218. 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 2.5.2-218. 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 2.5.2-218. These spectra were developed using the EPRI (Reference 2.5.2-248) median ground motions models and the spectral shape functions for CEUS ground motions presented in McGuire et al. (Reference 2.5.2-255).

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 2.5.2-256). 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 84^{th} percentile ground 10-Hz spectral acceleration based on the DEL magnitude and distance and one of the EPRI ground motion models (Reference 2.5.2-248, Reference 2.5.2-252). 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 2.5.2-257). Their model covered the frequency range of 0.2 to 20 Hz. Baker and Jayaram (Reference 2.5.2-258) have extended the Baker and Cornell (Reference 2.5.2-257) 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 2.5.2-218 as an m_b 7.1 earthquake occurring at a distance of 239 km (149 mi.) 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 2.5.2-220) 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 2.5.2-258). 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 2.5.2-248, Reference 2.5.2-255). 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) (Reference 2.5.2-248). 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 Center's (PEER) Next Generation Attenuation Project (NGA) (Reference 2.5.2-252). 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 2.5.2-252) 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 2.5.2-220). A weighed average of these spectra is then computed using the weights defined on Figure 2.5.2-220. 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 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250.

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 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250.

As can be seen on Figures 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250, 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 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The uniform hazard response spectra shown on Figure 2.5.2-242 represent ground motions occurring on generic CEUS hard rock conditions. As described in

Subsection 2.5.4.1, the subsurface conditions at the HAR site consists of 1.5 to 7.6 m (5 to 25 ft.) of residual soils overlying Triassic sedimentary rock. The upper portion of the sedimentary rock is weathered. These rocks are the upper layers of the Triassic sediments that fill the Deep River basin. Based on the interpreted depth to basement inferred from resistivity data shown on Figure 2.5.1-240, the estimated thickness of the Triassic sediments at the HAR site is 1524 m (5000 ft.).

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 2.5.2-255). 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 2.5.2-255]), 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 Subsection 2.5.2.4.4.3. Steps 6 and 7 are described in Subsection 2.5.2.6. Steps 1, 2, 4, and 5 are presented in this subsection.

2.5.2.5.1 Dynamic Properties of the HAR Site

2.5.2.5.1.1 Shallow Shear Wave Velocities

The shear (V_S) and compression (V_P) wave velocity data obtained at the HAR site are described in Subsection 2.5.4.4.2. A combination of suspension logging, downhole velocity surveys, and spectral analysis of surface waves (SASW) surveys were used to measure shear wave velocities to a depth of approximately 61 m (200 ft.). Measurements were conducted in or near seven borings: BPA-5, BPA-47, and BPA-48 at the HAR 2 site; and BPA-25, BPA-49, BPA-50, and BPA-39 at the HAR 3 site. The data obtained at each boring location are shown on Figures 2.5.2-251, 2.5.2-252, 2.5.2-253, 2.5.2-254, 2.5.2-255, 2.5.2-256, and 2.5.2-257. Suspension logging interval V_S velocities were measured in all borings. Downhole velocity measurements were conducted in two borings and SASW surveys were conducted adjacent to four borings.

GeoVision used its interval velocity data to construct travel time plots that were interpreted in terms of layered velocity models (Reference 2.5.2-259, Reference 2.5.2-260). These are shown by the lines labeled "PS_Smoothed" on Figures 2.5.2-251, 2.5.2-252, 2.5.2-253, 2.5.2-254, 2.5.2-255, 2.5.2-256, and 2.5.2-257. These interpretations are very useful in smoothing the interval velocity data to define the appropriate velocity for depth intervals where the average velocity is relatively constant. Additional interpretations of layering were made for depth intervals where the suspension logging results appear to reflect relatively constant velocity. Harmonic mean velocities were computed for these layers and are plotted as vertical black lines labeled "Averaged PS." The "Averaged PS" velocities are close to the PS_Smoothed values where the averaging is done over the same depth range, indicating that the two approaches for estimating an average layer velocity produce consistent estimates.

Using the data and interpretations shown on Figures 2.5.2-251, 2.5.2-252, 2.5.2-253, 2.5.2-254, 2.5.2-255, 2.5.2-256, and 2.5.2-257, median (geometric mean) shear wave velocity profiles were constructed for each boring location. These are shown on the figures for the individual borings. Figure 2.5.2-258 compares these velocity profiles for the HAR 2 site and Figures 2.5.2-259 and 2.5.2-260 compare the individual boring velocity profiles for the HAR 3 site. The three velocity profiles for the HAR 2 site are very similar. Below the surface soils, the initial layer consists of weathered rock with V_S in the range of 914 to 1402 m/sec (3000 to 4600 fps). The velocity climbs to over 1524 m/sec (5000 fps) below elevation 70 m (230 ft.) and over 1829 m/sec (6000 fps) below elevation 61 m (200 ft.). All three profiles show the presence of a thin layer (≤1.5 m [≤5 ft.] thick) with a velocity of ~1219 m/sec (~4000 fps) that occurs between elevation 50 and 41 m (165 and 135 ft.). This layer corresponds to a clay-rich fractured zone noted in the logs for each boring, and the elevation in each boring is consistent with the general dip of the rock strata beneath HAR 2.

The four velocity profiles for the HAR 3 site show two patterns. The profiles for borings BPA-25 and BPA-49 (Figure 2.5.2-259) show either a low-velocity residual soil or very fractured and weathered rock for elevations above approximately 73 m (240 ft.). Below elevation approximately 73 m (240 ft.), moderately weathered rock is encountered with V_S near 914 m/sec (3000 fps). In these profiles, the shear wave velocity increases less rapidly with depth than at HAR 2, reaching velocities in excess of 1829 m/sec (6000 fps) below elevation 58 m (190 ft.). A thin, low-velocity layer is also observed in these two profiles, which again corresponds to a more fractured and clay-rich seam in the rock noted in the boring logs. The velocity profile for the third boring in the vicinity of the nuclear island, BPA-50 (Figure 2.5.2-260), shows somewhat different behavior. Similar to the conditions at HAR 2, the velocity quickly climbs to near 1524 m/sec (5000 fps) by elevation 73 m (238 ft.). Unlike at HAR 2, the V_S remains at 1524 m/sec (5000 fps) until elevation 46 m (150 ft.) before climbing to over 1829 m/sec (6000 fps). A fourth boring, BPA-39, located at the eastern end of the turbine building at HAR 3, shows a similar velocity profile to that observed in BPA-50.

The velocity comparisons shown on Figure 2.5.2-258 indicate that a single average velocity profile is appropriate to characterize the HAR 2 site. However, the data shown on Figures 2.5.2-259 and 2.5.2-260 indicate that conditions vary across the HAR 3 site. The data obtained at the northwestern side of the nuclear island location show lower velocities near the surface and a more gradual increase in velocity with depth. These data are consistent with the deeper extent of soil and weathered rock at HAR 3, especially in the northwestern portion of the site (Subsection 2.5.4.1). The data from the southeastern side of the nuclear island location at HAR 3 are more similar to those obtained at HAR 2, but the lower surface velocities do extend to greater depths.

Regulatory Guide 1.208 states that the site-specific GMRS is to be defined at the ground surface or at the top of the first competent layer. Given that the upper soft residual soils and the highly weathered bedrock are to be removed during construction (Subsection 2.5.4.5), the reference point for the GMRS is taken to be the top of competent rock.

Because of the difference between shallow velocities at the HAR 2 and HAR 3 site and the variable conditions at the HAR 3 site, three shallow velocity profiles are defined for use in site response analyses. These profiles are shown on Figure 2.5.2-261. A single profile is used for the HAR 2 site with a surface elevation for the GMRS at 78 m (255 ft.). Two profiles are used for HAR 3: profiles HAR 3a and HAR 3b. The surface elevations for the GMRS are 73 m (240 ft.) for HAR 3a and 75 m (245 ft.) for HAR 3b. Similar velocity measurements were obtained in all borings below elevation 46 m (150 ft.). Therefore, the three velocity profiles are merged into a common velocity at this point. Site response analyses are conducted for all three profiles in order to assess the range of site amplification representative of conditions at the site. The thin, lower-velocity layers were maintained in the base case velocity profiles because these layers represent geologically definable layers in the shallow

sedimentary rock. Sensitivity analyses indicate that including these layers produces equal or slightly higher site amplification compared to analyses of base case profiles without them.

Evaluation of the seismic response of the proposed safety-related structures requires foundation input response spectra (FIRS) at other locations. The specific requirements are for a FIRS at the foundation level of the nuclear island, elevation 67.1 m (220 ft.) and at the foundation level of the Annex Building, elevation 79 m (258 ft.). The base case profiles for the nuclear island FIRS calculations were developed by removing all of the layers above elevation 67.1 m (220 ft.) from the GMRS profiles shown on Figure 2.5.2-261.

As described in Subsection 2.5.4.5, the planned construction approach is to remove the residual soil and highly weathered rock and backfill to plant grade. The depth of planned excavation is variable due to variations in the depth of weathering. As indicated above, the proposed nuclear islands at HAR 2 and HAR 3 will be founded on sound rock at elevation 67.1 m (220 ft.). Other structures, such as the Annex Building will be founded on backfill. The planned backfill material is to be a lean concrete designed to have a shear wave velocity comparable to that of the upper layer of competent rock at each site. The advantage of this approach is that the variability in backfill depth should not introduce large variabilities in calculated site response that might occur if a variable layer of granular fill with a much lower shear wave velocity were to be used. Therefore, the site response profiles used for developing the Annex Building FIRS consist of the GMRS profiles shown on Figure 2.5.2-261 with the addition of a layer of concrete to bring the surface elevation to 79 m (258 ft.). The nominal shear wave velocities of the material are assumed to be that of the top layer of the GMRS profiles, specifically of 1219 m/sec (4000 fps) at HAR 2 and 975 m/sec (3200 fps) at HAR 3. The variability in the thickness of lean concrete backfill was accounted for as part of the profile randomization.

2.5.2.5.1.2 Deep Shear Wave Velocities

The HAR site sits in the Deep River basin containing Triassic sedimentary rocks. The estimated depth to crystalline basement rocks at the site is approximately 1500 m (5000 ft.) (Figure 2.5.1-241). A deep well, Sears No. 1, located approximately 5 km (3 mi.) north of the HNP site, was drilled to a depth of 1142 m (3746 ft.) in Triassic sediments. Bain and Brown present a compression wave velocity log for Sears No. 1 well (Reference 2.5.2-261). Figure 2.5.2-262 shows the compression wave velocities versus depth. The shallowest measurements in the Sears No. 1 well are at a depth of 152 m (500 ft.). The compression wave velocity at this depth is in the range of 3810 to 4267 m/sec (12,500 to 14,000 fps). The measured compression wave velocities at elevations below 46 m (150 ft.) at the site are also in this range. This indicates that it is reasonable to use the data from the Sears No. 1 well to extend the shallow velocity profiles shown on Figure 2.5.2-261 to the base of the Triassic sediments. The site velocity data below elevation 46 m (150 ft.) described in Subsection 2.5.4.4 indicate an average value of Poisson's ratio of 0.32. This

value was used to compute shear wave velocities from the Sears No. 1 well. These are shown on Figure 2.5.2-262. A smooth velocity profile was used to represent the shear wave velocity of the Triassic sediments, increasing from a velocity of 2027 m/sec (6650 fps) at the base of the profiles shown on Figure 2.5.2-261 to a velocity of 2316 m/sec (7600 fps) at the base of the Triassic sediments. The crystalline bedrock below the Triassic sediments was assumed to be the point at which the generic CEUS hard rock conditions are encountered.

2.5.2.5.1.3 Rock Density

The data for the density of the sedimentary rocks at the site are presented in Subsection 2.5.4.2.2. These data indicate bulk unit weights increasing from 162 to 163 pounds per cubic foot (pcf) near the surface to 168 pcf below elevation 46 m (150 ft.). The gamma log data from the Sears No 1 well indicate bulk densities in the range of 158 to 163 pcf. Because the site data are based on direct measurements of rock cores, the site data were used to define the rock density for the site response analysis. The bulk density for the concrete backfill beneath the Annex Building was assumed to be equal to a nominal value for concrete of 145 pcf.

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 15 m (50 ft.) of partly to moderately weathered rock underlain by unweathered sedimentary rocks with V_S in excess of 1829 m/sec (6000 fps). To account for the potential of nonlinear behavior in the 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 2.5.2-262), as modified by Silva (Reference 2.5.2-263) to account for depth effects. An alternative set consists of the “Peninsula Range” set, also developed by Silva et al. (Reference 2.5.2-262). The Peninsula Range set was used for the shallow rock at the HNP site in site response analyses conducted by EPRI (Reference 2.5.2-264). These two sets of relationships are shown on Figure 2.5.2-263.

The unweathered rocks 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 2.5.2-265). The energy lost in shear wave propagation is 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:

$$\xi = \frac{1}{2Q_S} \qquad (2.5.2\text{-}7)$$

Parameter Q_S is also related to the high-frequency attenuation parameter κ developed by Anderson and Hough (Reference 2.5.2-266) by the relationship:

$$\kappa = \frac{H}{Q_S V_S} \qquad (2.5.2\text{-}8)$$

where H is the thickness of the crust over which the energy loss occurs, typically taken to be 1 to 2 km (Reference 2.5.2-267). Silva and Darragh (Reference 2.5.2-267) find that Q_S is proportional to shear wave velocity:

$$Q_S = \gamma V_S \qquad (2.5.2\text{-}9)$$

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:

$$\kappa_i = \frac{H_i}{\gamma V_{Si}^2} \qquad (2.5.2\text{-}10)$$

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 2.5.2-8 and 2.5.2-10, the appropriate damping values for each layer are then obtained.

EPRI (Reference 2.5.2-264) gives the following relationship between κ and site shear wave velocity:

$$\log(\kappa) = 2.2189 - 1.0930\log(V_S) \qquad (2.5.2\text{-}11)$$

where V_S is shear wave velocity in fps and κ is in seconds. The shear wave velocity in the unweathered rock is ~6600 fps. Using this value in Equation 2.5.2-11 yields a κ value of 0.0111 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 2.5.2-264). The three-point distribution developed by Keefer and Bodily is used to represent the uncertainty distribution, leading to a three-point distribution of 0.0074 (weight 0.185), 0.0111 (weight 0.63), and 0.0166 (weight 0.185) (Reference 2.5.2-268). The attenuation models for CEUS hard rock are developed assuming a shallow crustal κ of approximately 0.006 second (Reference 2.5.2-248). The difference between the generic CEUS hard rock κ and the sedimentary rock κ is attributed to material damping in the sedimentary rocks of the Deep River basin. The κ values are reduced by an additional 0.0002 second to account for the effects of scattering due to randomization of the velocity profiles. The remaining κ values of 0.0012, 0.0049, and 0.0104 second are used to compute equivalent damping values in

the Triassic sediments. The resulting damping values for the average velocities and layer thicknesses are listed in Table 2.5.2-219.

The lean concrete backfill beneath the Annex Building was assumed to remain essentially linear until approaching strains of 0.1 percent. It is also expected to have low damping. Hasek presents modulus reduction and damping data for various soil-concrete-flyash mixtures (Reference 2.5.2-269). The low-strain damping values are in the range of 1.5 to 2.5 percent. The shear wave velocities of these materials calculated from the measured shear modulus are in the range of 274 to 427 m/sec (900 to 1400 fps). Using the concept applied above that Q_S is proportional to velocity suggests that an appropriate low-strain damping estimate for the concrete backfill is 0.5 percent. Using these concepts, modulus reduction and damping relationships were developed for the lean concrete backfill that maintain linear behavior for effective shear strains up to 0.01 percent with gradually increasing nonlinear behavior at higher strains. These relationships are also shown on Figure 2.5.2-263.

2.5.2.5.1.5 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 2.5.2-262). 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, ρ is given by:

$$\rho(h,t) = (1-\rho_d(h))\rho_t(t) + \rho_d(h) \qquad (2.5.2\text{-}12)$$

where ρ_d represents the depth-dependent correlation (generally increasing with increasing depth), and ρ_t is the thickness-dependent correlation (generally decreasing with increasing layer thickness). The factors ρ_d and ρ_t are obtained from the expressions:

$$\rho_d(h) = \begin{cases} \rho_{200}\left[\dfrac{h+h_0}{200+h_0}\right]^b & for \quad h \le 200\,m \\ \rho_{200} & for \quad h > 200\,m \end{cases} \qquad (2.5.2\text{-}13)$$

and

$$\rho_t(t) = \rho_0 \exp\left[-\left(\frac{t}{\Delta}\right)^{\alpha}\right] \qquad (2.5.2\text{-}14)$$

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 2.5.2-262). 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 HAR 2 and HAR 3 sites display low variability in velocities for individual layers about the median profiles shown on Figure 2.5.2-261. For HAR 2, the standard deviation in $\ln(V_S)$ is slightly less than 0.2 in the shallowest weathered rock, decreasing to less than 0.1 at greater depths. These values are similar to those obtained from analyses of individual firm sites (Reference 2.5.2-262), and these values were used to develop randomized velocity profiles. The location of velocity layer boundaries were randomized to vary uniformly within the range of layer thickness observed in the site borings (Figures 2.5.2-251, 2.5.2-252, 2.5.2-253, 2.5.2-254, 2.5.2-255, 2.5.2-256, and 2.5.2-257).

Sixty randomized velocity profiles were generated for each base case velocity profile. Figures 2.5.2-264, 2.5.2-265, 2.5.2-266, 2.5.2-267, 2.5.2-268, and 2.5.2-269 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 2.5.2-270, 2.5.2-271, and 2.5.2-272.

The modulus reduction and damping relationships were also randomized, as shown on Figures 2.5.2-273, 2.5.2-274, 2.5.2-275, and 2.5.2-276. The standard deviation in the modulus reduction and damping were set so that the randomized relationships fell within recommended bounds provided by Silva (Reference 2.5.2-263). 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 (Reference 2.5.2-255, Reference 2.5.2-264). 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.

2.5.2.5.2 Acceleration Time Histories for Input Rock Motions

Response spectra were developed for each deaggregation earthquake, as shown on Figures 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250. Thirty time histories were developed for each deaggregation earthquake from the time history sets given in McGuire et al. (Reference 2.5.2-255). Table 2.5.2-220 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 2.5.2-270). Figure 2.5.2-277 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 2.5.2-278 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. Figure 2.5.2-279 shows the effect of site κ on the mean amplification for the HAR 2 GMRS profile. The range in κ leads to approximately a 25 percent difference in mean amplification at 100 Hz, 40 to 50 percent differences near 40 Hz, decreasing to about 15 percent at 10 Hz. The effect of κ on site motions decreases for frequencies below 10 Hz. Figure 2.5.2-280 shows the effect of the alternative property curves on the mean amplification for the HAR 2 GMRS profile. The site amplification is insensitive to the choice of the modulus reduction and damping relationships for the shallow portion of the Triassic sediments.

Weighted mean amplification functions for each DE were computed using the weights assigned to the κ values (Subsection 2.5.2.5.1.4) and assigning equal weight to the two sets of modulus reduction and damping relationships. Figures 2.5.2-281, 2.5.2-282, and 2.5.2-283 show the resulting DEL, DEM, and DEH amplification functions for 10^{-4} ground motions for the three HAR GMRS profiles. The site amplification is insensitive to differences in the input DE

motions for frequencies less than 10 Hz and shows only small variations at higher frequencies.

The mean amplification functions for each DE were then multiplied by the weights listed in Table 2.5.2-220 to produce a mean amplification function for the HF and LF motions. Figure 2.5.2-284 shows the resulting mean amplification functions for 10^{-4} ground motions for the three GMRS profiles. Differences in the site amplification functions obtained using the three velocity profiles occur primarily for frequencies above 4 Hz and reflect the differences in the shallow portion of the three base case velocity profiles.

Figures 2.5.2-285, 2.5.2-286, and 2.5.2-287 show the mean site amplification functions computed for the HAR 2, HAR 3a, and HAR 3b GMRS profiles, respectively. Each figure shows the amplification functions obtained for ground motions corresponding to 10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. Because only the shallowest portions of the underlying sedimentary rock are modeled with nonlinear properties, variations in the site amplification functions due to the level of ground motion are small and occur primarily at frequencies greater than 10 Hz. In order to produce smooth surface spectra, the amplification functions were smoothed by eye, as shown by the heavy dashed lines on Figures 2.5.2-285 2.5.2-286, and 2.5.2-287. A single smooth amplification function is defined for frequencies less than 10 Hz for the LF motions at all levels of shaking, shown by the heavy dashed line on the figures. As shown on Figures 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250, the LF RE spectra do not affect the site ground motions for frequencies above 5 Hz, and thus are not important for defining the surface motions. Smooth amplification functions were also defined for HF motions. Variations in the amplification functions due to the level of HF ground motion were included in the amplification functions for frequencies above 12 Hz for the HAR 2 profile, for frequencies above 6 Hz for the HAR 3a profile, and for frequencies above 7.5 Hz for the HAR 3b profile.

As can be seen on Figures 2.5.2-285, 2.5.2-286, and 2.5.2-287, the site amplification is insensitive to ground motion level except at frequencies above 5 to 10 Hz, and even at these frequencies the effect is small. This small effect is a result of the fact that even the 10^{-6} levels of shaking induce only small strains in the Triassic sediments. Figures 2.5.2-288, 2.5.2-289, and 2.5.2-290 show the statistics of effective strain computed in the calculations for the 10^{-5} HF ground motion inputs. The strains for nearly all cases are below 10^{-2} percent, indicating the motions are inducing only limited nonlinearity in the site materials.

The results shown on Figure 2.5.2-284 indicate that the two HAR 3 GMRS profiles produce somewhat different amplification for frequencies above about 3 Hz, with the HAR 3a profile producing greater response for frequencies above 15 Hz and the HAR 3b profile producing slightly higher response for frequencies between 3 and 15 Hz. As the two profiles both represent characteristics of parts of the HAR 3 site, the envelope of the two smoothed amplification functions is used to develop the surface motions.

2.5.2.5.3.2 Site Amplification Functions for FIRS Profiles

The process described above for developing the GMRS profile amplification functions was repeated for the two FIRS locations. The analysis for the nuclear island FIRS was conducted using profiles with a top surface at elevation 67.1 m (220 ft.) and the analysis for the Annex Building FIRS was conducted using profiles with a top surface at elevation 79 m (258 ft.). Figure 2.5.2-291 shows the mean LF and HF amplification functions for the three nuclear island FIRS profiles for 10^{-4} exceedance level ground motions. The differences in amplification between the three profiles are smaller than those observed for the GMRS profiles (Figure 2.5.2-284) because most of the differences in the profiles occur at higher elevations.

Figure 2.5.2-292 shows the mean LF and HF amplification functions for the three Annex Building FIRS profiles for 10^{-4} exceedance level ground motions. The differences in amplification between the three profiles are somewhat larger than those observed for the GMRS profiles (Figure 2.5.2-284) because of the differences in concrete backfill thickness. As was observed for the GMRS profiles, the amplification functions for the FIRS profiles are relatively insensitive to the level shaking, especially the nuclear island FIRS profiles in which very little potentially nonlinear material remains.

Consistent with the approach used for the GMRS amplification, the FIRS amplification functions for the HAR 3 site are taken as the envelope of the values calculated for the HAR 3a and HAR 3b profiles.

2.5.2.5.3.3 Surface Spectra

Surface hazard spectra for the HAR GMRS profiles are obtained by scaling the rock RE and UHRS by the site amplification functions. Figures 2.5.2-247, 2.5.2-248, 2.5.2-249, and 2.5.2-250 show the reference (controlling) spectra for LF and HF motions developed for each annual exceedance level. These spectra are scaled by the appropriate smoothed amplification function to produce ground surface spectra. Figures 2.5.2-293 and 2.5.2-294 show these results for the 10^{-4} ground motions for the HAR 2 and HAR 3 GMRS profiles, respectively. The generic rock UHRS is also scaled using the appropriate LF and HF amplification values at the seven spectral frequencies that define the 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. show a broader peak in the spectrum in the frequency range of 10 to 100 Hz (Reference 2.5.2-255). Therefore, the approach described in 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 and 25 Hz to the appropriate value at 10 or

100 Hz. This smoothed rock UHRS was then multiplied by the HF amplification function.

A smooth envelope of all of the scaled spectra is then constructed. These smooth envelope spectra represents the 10^{-4} surface spectrum for the site defined as a free-field outcropping motion for the competent sedimentary rocks. Similar operations were performed to develop surface spectra for the 10^{-5} and 10^{-6} exceedance level motions.

2.5.2.5.3.4 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 5.0. 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 2.5.2-271).

The EPRI CAV model was implemented in a second set of PSHA calculations for the HAR site (Reference 2.5.2-271). 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 CAV model presented in EPRI 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 2.5.2-271). The magnitude conversions used in the PSHA convert a m_b 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 2.5.2-271). 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 results described in Subsection 2.5.2.5.3.1 indicate that the GMRS profile amplification is independent of amplitude for frequencies of 5 Hz or less and is weakly dependent on amplitude at higher frequencies.

Two sets of PSHA calculations with site amplification were performed for each HAR site. 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 Subsection 2.5.2.5.3.3 to produce the final hazard-consistent surface spectra.

Figures 2.5.2-295, 2.5.2-296, 2.5.2-297, 2.5.2-298, 2.5.2-299, 2.5.2-300, and 2.5.2-301 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. Also shown on these figures is the corresponding generic CEUS mean rock hazard curve from Subsection 2.5.2.4.4. The hazard curves computed using CAV all level off at an exceedance frequency of approximately 2×10^{-4}, indicating the frequency of earthquakes that produce sufficient peak ground acceleration to induce a CAV of 0.16 g-second or greater. Because the site amplification is relatively insensitive to ground motion level, the relationship of the mean surface hazard curve to fractile surface hazard curves will be very similar to that shown for the generic rock hazard curves presented on Figures 2.5.2-230, 2.5.2-231, 2.5.2-232, 2.5.2-233, 2.5.2-234, 2.5.2-235, and 2.5.2-236.

The surface mean hazard results shown on Figures 2.5.2-295, 2.5.2-296, 2.5.2-297, 2.5.2-298, 2.5.2-299, 2.5.2-300, and 2.5.2-301 are interpolated to obtain the spectral accelerations corresponding to mean annual frequencies of exceedance of 10^{-4}, 10^{-5}, and 10^{-6}. Table 2.5.2-221 lists the ratio of the surface spectra accelerations computed with CAV to those computed without CAV for the seven spectral frequencies and the three exceedance frequencies. These ratios are then used to scale the smooth surface spectra described in Subsection 2.5.2.5.3.3 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. Figures 2.5.2-302 and 2.5.2-303 show the resulting mean 10^{-4}, 10^{-5}, and 10^{-6} surface spectra for the HAR 2 and HAR 3 sites, respectively.

2.5.2.6 Ground Motion Response Spectrum

2.5.2.6.1 Horizontal GMRS

HAR COL 2.5-3 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:

$$GMRS = DF \times UHRS\ (10^{-4}) \qquad (2.5.2\text{-}15)$$

Parameter *DF* is the design factor specified by the expression:

$$DF = \text{Maximum}\ (1.0,\ 0.6(A_R)^{0.8}) \qquad (2.5.2\text{-}16)$$

In which A_R is the ratio of the UHRS ground motions for annual exceedance frequencies of 10^{-4} and 10^{-5}, specifically:

$$A_R = \frac{UHRS(10^{-5})}{UHRS(10^{-4})} \qquad (2.5.2\text{-}17)$$

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.

Figures 2.5.2-304 and 2.5.2-305 compare the horizontal GMRS calculated from the UHRS using Equations 2.5.2-15 through 2.5.2-17 to values computed as $0.45 \times SA(0.1H_D)$ for the HAR 2 and HAR 3 sites, respectively. The values of A_R do exceed 4.2 for frequencies less than about 1 Hz. The horizontal GMRS are taken as the envelope of the two GMRS values shown in the figures. These values are listed in Table 2.5.2-222. According to Westinghouse's DCD, the horizontal GMRS are compared to the Westinghouse horizontal Certified Seismic Design Response Spectrum (CSDRS) on Figures 2.5.2-306 and 2.5.2-307. For the HAR 2 site there is a very minor exceedance of the horizontal CSDRS (maximum 5.5 percent) over a narrow frequency range of approximately 32 to 36 Hz. For the HAR 3 site the exceedance of the horizontal CSDRS is larger (maximum of 19 percent) and extends over the frequency range of 29 to 42 Hz.

2.5.2.6.2 Vertical GMRS

The vertical GMRS were developed from the horizontal GMRS using vertical to horizontal (V/H) spectral ratios for CEUS rock recommended by McGuire et al. (Reference 2.5.2-255). These are given as a function of frequency for three levels of peak acceleration. The CEUS hard rock V/H ratios were used because the site shear wave velocities and estimated value of κ are close to that for CEUS generic rock and the site amplifications are relatively close to 1.0. As the horizontal GMRS peak acceleration values are all less than 0.2 g, the V/H ratios for <0.2 g were used to develop vertical GMRS. These are listed in Table 2.5.2-222 and are compared to the Westinghouse vertical CSDRS spectra on Figures 2.5.2-306 and 2.5.2-307. The vertical GMRS are enveloped by the CSDRS.

HAR COL 2.5-2

Table 2.5.2-201
Bechtel Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
BZ4 Atlantic Coast BG[(a)]	1	145	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 BG[(a,b)]	1	0	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[(a,c)]	0.42	267	6.8 [0.2], 7.1 [0.4], 7.4 [0.4]	6.6 [1.0]
N3 Charleston Faults[(a,c)]	0.53	300	6.8 [0.2], 7.1 [0.4], 7.4 [0.4]	6.6 [1.0]
F Southeast Appalachians[(a)]	0.35	77	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]
13 Eastern Mesozoic Basins[(a,b)]	0.1	0	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]
Additional Sources				
24 Bristol Block[(a,d)]	0.25	222	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]
25 New York – Alabama Lineament[(d)]	0.3	350	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]	Not Included
25A Alternate for 25[(d)]	0.45	320	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]	Not Included
E Central Virginia[(a)]	0.35	189	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]	5.4 [0.1], 5.7 [0.4] 6.0 [0.4], 6.6 [0.1]
19 Giles County[(a)]	0.35	234	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]

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-202
Dames & Moore Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
40 Central Virginia[a]	1	170	6.6 [0.8], 7.2 [0.2]	6.6 [0.8], 7.2 [0.2]
54 Charleston[a,c]	1	256	6.6 [0.75], 7.2 [0.25]	6.6 [1.0]
41 Southern Cratonic Margin[a]	0.12	66	6.1 [0.8], 7.2 [0.2]	6.1 [0.8], 7.2 [0.2]
53 Southern Appalachians Mobile Belt[a,b]	0.26	0	5.6 [0.8], 7.2 [0.2]	5.6 [0.8], 7.2 [0.2]
Additional Sources				
4 Appalachian Fold Belt[a,d]	0.35	214	6.0 [0.8], 7.2 [0.2]	6.0 [0.8], 7.2 [0.2]
4A, 4B, 4C, 4D Appalachian Fold Belt Knots[a,d]	0.65	420 for 4A 218 for 4B 464 for 4C 660 for 4D	6.6 [0.75], 7.2 [0.25] for 4A 6.2 [0.75], 7.2 [0.25] for 4B 5.0 [0.75], 7.2 [0.25] for 4C 5.6 [0.75], 7.2 [0.25] for 4D	6.6 [0.75], 7.2 [0.25] for 4A 6.2 [0.75], 7.2 [0.25] for 4B 5.0 [0.75], 7.2 [0.25] for 4C 5.6 [0.75], 7.2 [0.25] for 4D
47 to 51 Mesozoic/Cenozoic Basins [49[b]]	0.28	274 for 47 135 for 48 0 for 49 44 for 50 163 for 51	6.0 [0.75], 7.2 [0.25]	Not Included
52 Charleston Mesozoic Rift	0.46	265	4.7 [0.75], 7.2 [0.25]	Not Included

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-203
Law Engineering Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
107 Eastern Piedmont[a,b]	0.42	0	4.9 [0.3], 5.5 [0.4], 5.7 [0.3]	5.0 [0.3], 5.5 [0.4], 5.7 [0.3]
17 Eastern Basement[a,d]	0.62	122	5.7 [0.2], 6.8 [0.8]	5.7 [0.2], 6.8 [0.8]
35 Charleston[a,c]	0.6	284	6.8 [1.0]	6.6 [1.0]
M Mafic Plutons[a] (38)	0.43	109 to 195	6.8 [1.0]	6.8 [1.0]
8 Mesozoic Basins[a] (C09 & C10)	0.27	83	6.8 [1.0]	6.8 [1.0]
22 Eastern Seaboard Reactivated Normal Faults[a,b] (C11)	0.27	0	6.8 [1.0]	6.8 [1.0]
Additional Source				
217 Eastern Basement[a,d]	0.38	122	5.2 [0.5], 5.7 [0.5]	5.7 [1.0]

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-204
Rondout Associates Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
49 (C01) Appalachian Crust[a,b]	1	0	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]
24 Charleston[a,c]	1	249	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]	6.6 [1.0]
26 South Carolina[a]	1	164	5.8 [0.2], 6.5 [0.6], 6.8 [0.2]	5.8 [0.2], 6.5 [0.6], 6.8 [0.2]
29 Central Virginia[a]	1	177	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]
28 Giles County[a]	1	190	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]
Additional Sources				
25 Southern Appalachian[a,d]	0.99	344	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]	6.6 [0.2], 6.8 [0.6], 7.0 [0.2]
27 Tennessee-Virginia Border[a]	0.99	224	5.2 [0.2], 6.3 [0.6], 6.5 [0.2]	5.2 [0.2], 6.3 [0.6], 6.5 [0.2]
32 Norfolk Fracture[a]	0.67	190	5.8 [0.2], 6.5 [0.6], 6.8 [0.2]	5.8 [0.2], 6.5 [0.6], 6.8 [0.2]
50 (C02) Grenville Crust[a]	1	101	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]	4.8 [0.2], 5.5 [0.6], 5.8 [0.2]

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-205
Weston Geophysical Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
104 Southern Coastal Plain[a,b]	1	0	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[a,c]	0.99	256	6.6 [0.9], 7.2 [0.1]	6.6 [1.0]
26 South Carolina[a,c]	0.86	94	6.0 [0.67], 6.6 [0.27], 7.2 [0.06]	6.0 [0.67], 6.6 [0.27], 7.2 [0.06]
22 Central Virginia[a]	0.82	165	5.4 [0.19], 6.0 [0.65], 6.6 [0.16]	5.4 [0.19], 6.0 [0.65], 6.6 [0.16]
28D Mesozoic Basin[a,b]	0.26	0	5.4 [0.65], 6.0 [0.25], 6.6 [0.1]	5.4 [0.65], 6.0 [0.25], 6.6 [0.1]
103 Southern Appalachian (C17, C18, C19)[a,d]	1	124	5.4 [0.26], 6.0 [0.58], 6.6 [0.16]	5.7 [0.26], 6.0 [0.58], 6.6 [0.16] for 103 and C18 5.4 [0.26], 6.0 [0.58], 6.6 [0.16] for C17 and C19
Additional Sources				
23 Giles County[a]	0.9	212	6.0 [0.81], 6.6 [0.19]	6.0 [0.81], 6.6 [0.19]
24 New York – Alabama – Clingman Lineaments[a,d]	0.9	250	5.4 [0.26], 6.0 [0.58], 6.6 [0.16]	5.4 [0.26], 6.0 [0.58], 6.6 [0.16]
28C and 28 E Mesozoic Basins	0.26	111	5.4 [0.65], 6.0 [0.25], 6.6 [0.1]	Not Included

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-206
Woodward-Clyde Consultants Team Seismic Sources

Source	P*	Closest Distance to HAR Site (km)	EPRI (1989a) Maximum Magnitude Distribution for HNP Site (m_b)	Maximum Magnitude Distribution Used in PSHA for HAR Site (m_b)
EPRI (Reference 2.5.2-202) Source Set				
B24 Background[a,b]	1	0	5.8 [0.33], 6.2 [0.34], 6.6 [0.33]	5.8 [0.33], 6.2 [0.34], 6.6 [0.33]
26 Central Virginia Gravity Saddle[a]	0.434	160	5.4 [0.33], 6.5 [0.34], 7.0 [0.33]	5.4 [0.33], 6.5 [0.34], 7.0 [0.33]
27 State Farm Complex[a]	0.474	162	5.6 [0.33], 6.3 [0.34], 6.9 [0.33]	5.6 [0.33], 6.3 [0.34], 6.9 [0.33]
29 Greater South Carolina[a,c]	0.122	161	6.7 [0.33], 7.0 [0.34], 7.4 [0.33]	6.6 [1.0]
29A Greater South Carolina #2[a,c]	0.305	214	6.7 [0.33], 7.0 [0.34], 7.4 [0.33]	6.6 [1.0]
29B Greater South Carolina #3[a]	0.105	161	5.4 [0.33], 6.0 [0.34], 7.0 [0.33]	5.4 [0.33], 6.0 [0.34], 7.0 [0.33]
30 Charleston[a,c]	0.573	272	6.8 [0.33], 7.3 [0.34], 7.5 [0.33]	6.6 [1.0]
Additional Sources				
31 Blue Ridge[d]	0.024	202	5.9 [0.33], 6.3 [0.34], 7.0 [0.33]	Not Included
31A Blue Ridge alternative[a,d]	0.211	202	5.9 [0.33], 6.3 [0.34], 7.0 [0.33]	5.9 [0.33], 6.3 [0.34], 7.0 [0.33]
28 Richmond Basin	0.092	204	5.8 [0.33], 6.2 [0.34], 6.6 [0.33]	Not Included

a) Included in HAR PSHA.

b) Host/background sources.

c) Charleston sources.

d) East Tennessee seismic zone sources.

Notes:

P* = the probability that the source is included in the hazard model.

HAR COL 2.5-2

Table 2.5.2-207 (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 2.5.2-225). 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 2.5.2-225). A summary map showing the major source zone alternative boundaries is presented on Figure 2.5.2-209.
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.

HAR COL 2.5-2

Table 2.5.2-207 (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-I) + (4A-2) + (4A-3), but has a graded boundary defined by three cylindrical sources.

HAR COL 2.5-2

Table 2.5.2-207 (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).

Source: Reference 2.5.2-225.

Table 2.5.2-208 (Sheet 1 of 3)
Earthquake Counts and Assessed Catalog Completeness within 320 km (200 mi.) of HAR Site

	Assessed Probability of Detection for Time Period*								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	TE[a]	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 Catalog
	Corresponding Time Length (years)																
m_b*	155	80	50	40	25	10.16	21.84	(years)									
Completeness Region 3[b]																	
3.3 to 3.9			0.182	0.489	0.76	1	0	57.82			2	6	8	1	0	17	EPRI-SOG[c]
			1	1	1	1	1	147.00			12	6	7	3	4	32	Update[d]
			0.91	0.91	0.91	1	1	136.65			10	6	7	3	4	30	Update-EPRI AS[e]
3.9 to 4.5			0.524	1	1	1		101.36			2	1	4	0	0	7	EPRI-SOG
			1	1	1	1	1	147.00			6	1	2	0	2	11	Update
			1	1	1	1	1	147.00			5	1	2	0	2	10	Update-EPRI AS
4.5 to 5.1		0.233	0.721	1	1	1		129.85		2	0	2	1	1	0	6	EPRI-SOG
		0.233	1	1	1	1	1	165.64		1	5	0	0	0	0	6	Update
		0.233	1	1	1	1	1	165.64		1	3	0	0	0	0	4	Update-EPRI AS
5.1 to 5.7		0.233	0.964	1	1	1		142.00		0	1	2	0	0	0	3	EPRI-SOG
		0.233	1	1	1	1	1	165.64		0	1	1	0	0	0	2	Update
		0.233	1	1	1	1	1	165.64		0	1	1	0	0	0	2	Update-EPRI AS
5.7 to 6.3		0.436	0.981	1	1	1		159.09		0	0	0	0	0	0	0	EPRI-SOG
		0.436	1	1	1	1	1	181.88		0	0	0	0	0	0	0	Update
		0.436	1	1	1	1	1	181.88		0	0	0	0	0	0	0	Update-EPRI AS
6.3 to 6.9		0.588	1	1	1	1		172.20		0	0	0	0	0	0	0	EPRI-SOG
		0.588	1	1	1	1	1	194.04		0	0	0	0	0	0	0	Update
		0.588	1	1	1	1	1	194.04		0	0	0	0	0	0	0	Update-EPRI AS
Completeness Region 6																	
3.3 to 3.9		0.289	0.861	0.987	1	1		140.81		2	2	4	4	0	0	12	EPRI-SOG
		0.723	0.861	0.987	1	1	1	197.33		5	2	4	5	0	1	17	Update
		0.578	0.861	0.987	1	1	1	185.77		4	2	4	5	0	1	16	Update-EPRI AS
3.9 to 4.5		0.652	1	1	1	1		177.32		0	2	2	1	0	0	5	EPRI-SOG
		0.652	1	1	1	1	1	199.16		0	3	1	0	0	0	4	Update
		0.652	1	1	1	1	1	199.16		0	3	1	0	0	0	4	Update-EPRI AS
4.5 to 5.1	0.412	0.937	1	1	1	1		263.98	1	1	0	0	0	0	0	2	EPRI-SOG
	0.412	0.937	1	1	1	1	1	285.82	1	1	1	0	0	0	0	3	Update
	0.412	0.937	1	1	1	1	1	285.82	1	1	1	0	0	0	0	3	Update-EPRI AS
5.1 to 5.7	0.812	0.961	1	1	1	1		327.90	0	0	0	0	0	0	0	0	EPRI-SOG
	0.812	0.961	1	1	1	1	1	349.74	0	0	0	0	0	0	0	0	Update
	0.812	0.961	1	1	1	1	1	349.74	0	0	0	0	0	0	0	0	Update-EPRI AS
5.7 to 6.3	0.962	0.991	1	1	1	1		353.55	0	0	0	0	0	0	0	0	EPRI-SOG
	0.962	0.991	1	1	1	1	1	375.39	0	0	0	0	0	0	0	0	Update
	0.962	0.991	1	1	1	1	1	375.39	0	0	0	0	0	0	0	0	Update-EPRI AS

HAR COL 2.5-2

Table 2.5.2-208 (Sheet 2 of 3)
Earthquake Counts and Assessed Catalog Completeness within 320 km (200 mi.) of HAR Site

	Assessed Probability of Detection for Time Period*								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 for	Earthquake
m_b*	155	80	50	40	25	10.16	21.84	(years)	1780	1860	1910	1950	1975	3/1985	1/2007	TE	Catalog
6.3 to 6.9	0.988	1	1	1	1	1		358.30	0	0	0	0	0	0	0	0	EPRI-SOG
	0.988	1	1	1	1	1	1	380.14	0	0	0	0	0	0	0	0	Update
	0.988	1	1	1	1	1	1	380.14	0	0	0	0	0	0	0	0	Update-EPRI AS
Completeness Region 7																	
3.3 to 3.9			0.307	0.532	0.951	1		70.57			2	0	1	0	0	3	EPRI-SOG
			0.307	0.532	1	1	1	93.63			0	0	2	0	1	3	Update
			0.307	0.532	1	1	1	93.63			0	0	2	0	1	3	Update-EPRI AS
3.9 to 4.5			0.546	0.918	1	1		99.18			2	0	1	0	0	3	EPRI-SOG
			0.546	1	1	1	1	124.30			1	0	0	0	0	1	Update
			0.546	1	1	1	1	124.30			1	0	0	0	0	1	Update-EPRI AS
4.5 to 5.1		0.475	0.925	0.984	1	1		158.77		0	0	0	0	0	0	0	EPRI-SOG
		0.475	1	1	1	1	1	185.00		0	1	0	0	0	0	1	Update
		0.475	1	1	1	1	1	185.00		0	1	0	0	0	0	1	Update-EPRI AS
5.1 to 5.7		0.957	0.991	1	1	1		201.27		0	0	0	0	0	0	0	EPRI-SOG
		0.957	1	1	1	1	1	223.56		0	0	0	0	0	0	0	Update
		0.957	1	1	1	1	1	223.56		0	0	0	0	0	0	0	Update-EPRI AS
5.7 to 6.3		0.996	1	1	1	1		204.84		0	0	0	0	0	0	0	EPRI-SOG
		0.996	1	1	1	1	1	226.68		0	0	0	0	0	0	0	Update
		0.996	1	1	1	1	1	226.68		0	0	0	0	0	0	0	Update-EPRI AS
6.3 to 6.9		1	1	1	1	1		205.16		0	0	0	0	0	0	0	EPRI-SOG
		1	1	1	1	1	1	227.00		0	0	0	0	0	0	0	Update
		1	1	1	1	1	1	227.00		0	0	0	0	0	0	0	Update-EPRI AS
Completeness Region 12																	
3.3 to 3.9			0.303	0.754	1	1		80.47			12	12	10	3	0	37	EPRI-SOG
			0.581	1	1	1	1	126.04			23	17	17	3	4	64	Update
			0.328	0.754	1	1	1	103.57			13	12	13	3	4	45	Update-EPRI AS
3.9 to 4.5		0.435	0.881	0.995	1	1		153.81		7	6	5	5	1	0	24	EPRI-SOG
		0.435	1	1	1	1	1	181.80		1	13	6	4	2	2	28	Update
		0.435	1	1	1	1	1	181.80		1	13	6	1	2	2	25	Update-EPRI AS
4.5 to 5.1	0.327	0.82	0.99	1	1	1		240.95	0	3	2	2	1	0	0	8	EPRI-SOG
	0.327	1	1	1	1	1	1	277.69	0	4	4	1	2	0	0	11	Update
	0.327	0.82	1	1	1	1	1	263.29	0	3	4	0	2	0	0	9	Update-EPRI AS

HAR COL 2.5-2

Table 2.5.2-208 (Sheet 3 of 3)
Earthquake Counts and Assessed Catalog Completeness within 320 km (200 mi.) of HAR Site

	Assessed Probability of Detection for Time Period*								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)																
m_b*	155	80	50	40	25	10.16	21.84	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 Catalog
5.1 to 5.7	0.763	0.935	0.99	1	1	1		317.73	0	0	0	1	0	0	0	1	EPRI-SOG
	0.763	0.935	1	1	1	1	1	340.07	0	1	1	1	0	0	0	3	Update
	0.763	0.935	1	1	1	1	1	340.07	0	1	0	1	0	0	0	2	Update-EPRI AS
5.7 to 6.3	0.952	0.987	1	1	1	1		351.68	0	0	1	0	0	0	0	1	EPRI-SOG
	0.952	0.987	1	1	1	1	1	373.52	0	0	1	0	0	0	0	1	Update
	0.952	0.987	1	1	1	1	1	373.52	0	0	1	0	0	0	0	1	Update-EPRI AS
6.3 to 6.9	0.984	1	1	1	1	1		357.68	0	0	1	0	0	0	0	1	EPRI-SOG
	0.984	1	1	1	1	1	1	379.52	0	0	1	0	0	0	0	1	Update
	0.984	1	1	1	1	1	1	379.52	0	0	1	0	0	0	0	1	Update-EPRI AS

Notes:
a) Equivalent period of completeness.

b) Completeness regions shown on Figure 2.5.2-216.

c) EPRI-SOG (Reference 2.5.2-201).

d) Updated catalog (Appendix 2AA).

e) Updated catalog with events flagged as aftershocks by EPRI-SOG (Reference 2.5.2-201) removed.

HAR COL 2.5-2

Table 2.5.2-209
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

HAR COL 2.5-2

Table 2.5.2-210
PSHA Results for 0.5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

0.5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-04	3.29E-02	1.18E-02	1.78E-02	2.95E-02	4.90E-02	6.17E-02
3.00E-04	1.55E-02	4.68E-03	6.92E-03	1.29E-02	2.46E-02	3.39E-02
1.00E-03	5.44E-03	1.91E-03	2.57E-03	4.37E-03	8.32E-03	1.26E-02
2.00E-03	3.05E-03	1.07E-03	1.55E-03	2.57E-03	4.47E-03	6.61E-03
5.00E-03	1.42E-03	3.31E-04	5.89E-04	1.23E-03	2.24E-03	3.16E-03
7.00E-03	1.04E-03	1.74E-04	3.47E-04	8.51E-04	1.74E-03	2.46E-03
1.00E-02	7.06E-04	7.76E-05	1.74E-04	5.25E-04	1.29E-03	1.91E-03
2.00E-02	2.80E-04	1.05E-05	3.02E-05	1.45E-04	5.50E-04	9.77E-04
3.00E-02	1.43E-04	2.57E-06	8.51E-06	5.25E-05	2.69E-04	5.75E-04
5.00E-02	5.21E-05	3.24E-07	1.32E-06	1.15E-05	8.71E-05	2.34E-04
7.00E-02	2.42E-05	6.76E-08	3.24E-07	3.55E-06	3.47E-05	1.07E-04
1.00E-01	9.79E-06	1.18E-08	6.31E-08	8.91E-07	1.10E-05	4.07E-05
2.00E-01	1.25E-06	1.02E-09	2.75E-09	4.37E-08	8.91E-07	3.98E-06
3.00E-01	3.06E-07	5.13E-10	8.71E-10	7.08E-09	1.78E-07	8.71E-07
5.00E-01	4.20E-08	< 1E-10*	< 1E-10*	1.32E-09	1.86E-08	1.26E-07
1.00E+00	2.14E-09	< 1E-10*	< 1E-10*	< 1E-10*	1.59E-09	9.12E-09

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-211
PSHA Results for 1-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

1-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	1.28E-02	3.89E-03	5.75E-03	1.05E-02	2.04E-02	2.88E-02
2.00E-03	6.67E-03	2.24E-03	3.09E-03	5.37E-03	1.05E-02	1.51E-02
5.00E-03	2.83E-03	1.02E-03	1.48E-03	2.40E-03	4.17E-03	6.03E-03
7.00E-03	2.08E-03	6.92E-04	1.07E-03	1.82E-03	3.02E-03	4.37E-03
1.00E-02	1.47E-03	4.17E-04	6.92E-04	1.29E-03	2.24E-03	3.09E-03
2.00E-02	6.61E-04	1.07E-04	2.14E-04	5.25E-04	1.10E-03	1.66E-03
3.00E-02	3.65E-04	3.80E-05	8.51E-05	2.57E-04	6.46E-04	1.07E-03
5.00E-02	1.46E-04	7.76E-06	2.04E-05	7.94E-05	2.63E-04	5.13E-04
7.00E-02	7.09E-05	2.29E-06	6.76E-06	3.09E-05	1.23E-04	2.75E-04
1.00E-01	2.97E-05	5.37E-07	1.82E-06	1.00E-05	4.79E-05	1.23E-04
2.00E-01	3.95E-06	2.04E-08	9.12E-08	7.76E-07	5.13E-06	1.59E-05
3.00E-01	9.97E-07	3.39E-09	1.29E-08	1.41E-07	1.15E-06	3.80E-06
5.00E-01	1.49E-07	8.32E-10	1.78E-09	1.32E-08	1.62E-07	6.03E-07
7.00E-01	3.99E-08	5.01E-10	7.76E-10	3.31E-09	4.17E-08	1.86E-07
1.00E+00	9.84E-09	< 1E-10*	5.01E-10	1.26E-09	9.55E-09	5.13E-08
2.00E+00	6.74E-10	< 1E-10*	< 1E-10*	< 1E-10*	1.29E-09	4.07E-09

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-212
PSHA Results for 2.5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

2.5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	3.24E-02	1.35E-02	1.86E-02	3.02E-02	4.57E-02	5.75E-02
2.00E-03	1.99E-02	6.92E-03	9.55E-03	1.74E-02	3.02E-02	4.07E-02
5.00E-03	8.32E-03	2.82E-03	3.89E-03	6.76E-03	1.32E-02	1.86E-02
1.00E-02	3.96E-03	1.48E-03	2.00E-03	3.31E-03	5.89E-03	8.51E-03
2.00E-02	1.85E-03	6.03E-04	9.12E-04	1.62E-03	2.75E-03	3.80E-03
5.00E-02	5.50E-04	9.55E-05	1.82E-04	4.47E-04	9.33E-04	1.38E-03
1.00E-01	1.54E-04	1.35E-05	2.95E-05	9.55E-05	2.75E-04	5.01E-04
2.00E-01	2.89E-05	1.20E-06	3.16E-06	1.32E-05	4.90E-05	1.12E-04
3.00E-01	8.97E-06	2.34E-07	6.92E-07	3.47E-06	1.41E-05	3.55E-05
4.00E-01	3.62E-06	7.08E-08	2.14E-07	1.26E-06	5.50E-06	1.41E-05
5.00E-01	1.74E-06	2.63E-08	8.32E-08	5.50E-07	2.63E-06	6.76E-06
6.00E-01	9.36E-07	1.18E-08	3.63E-08	2.63E-07	1.45E-06	3.80E-06
8.00E-01	3.47E-07	3.47E-09	1.00E-08	7.94E-08	5.62E-07	1.48E-06
1.00E+00	1.59E-07	1.66E-09	3.89E-09	2.95E-08	2.57E-07	6.92E-07
2.00E+00	1.39E-08	5.13E-10	8.13E-10	2.00E-09	2.09E-08	7.24E-08
3.00E+00	3.12E-09	< 1E-10*	< 1E-10*	9.55E-10	4.68E-09	1.82E-08

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-213
PSHA Results for 5-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

5-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
2.00E-03	3.04E-02	1.29E-02	1.70E-02	2.82E-02	4.27E-02	5.50E-02
5.00E-03	1.51E-02	5.13E-03	6.92E-03	1.29E-02	2.34E-02	3.24E-02
1.00E-02	7.47E-03	2.46E-03	3.39E-03	6.17E-03	1.18E-02	1.66E-02
2.00E-02	3.34E-03	1.10E-03	1.59E-03	2.82E-03	5.13E-03	7.41E-03
5.00E-02	9.89E-04	2.24E-04	3.89E-04	8.32E-04	1.55E-03	2.29E-03
1.00E-01	3.08E-04	4.27E-05	8.32E-05	2.24E-04	5.25E-04	8.71E-04
2.00E-01	7.03E-05	6.03E-06	1.32E-05	4.17E-05	1.20E-04	2.34E-04
3.00E-01	2.56E-05	1.78E-06	4.07E-06	1.35E-05	4.27E-05	8.91E-05
4.00E-01	1.18E-05	7.24E-07	1.70E-06	6.03E-06	2.00E-05	4.17E-05
5.00E-01	6.37E-06	3.47E-07	8.32E-07	3.16E-06	1.07E-05	2.29E-05
6.00E-01	3.80E-06	1.74E-07	4.57E-07	1.82E-06	6.46E-06	1.41E-05
8.00E-01	1.66E-06	5.13E-08	1.66E-07	7.24E-07	2.88E-06	6.31E-06
1.00E+00	8.70E-07	1.82E-08	7.24E-08	3.47E-07	1.55E-06	3.47E-06
1.50E+00	2.61E-07	3.24E-09	1.38E-08	8.13E-08	4.79E-07	1.10E-06
2.00E+00	1.08E-07	1.29E-09	4.47E-09	2.69E-08	1.91E-07	4.79E-07
4.00E+00	1.05E-08	< 1E-10*	8.32E-10	2.04E-09	1.70E-08	5.37E-08

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-214
PSHA Results for 10-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

10-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
3.00E-03	2.65E-02	1.15E-02	1.48E-02	2.34E-02	3.80E-02	5.01E-02
1.00E-02	9.82E-03	3.31E-03	4.47E-03	7.94E-03	1.48E-02	2.29E-02
2.00E-02	4.61E-03	1.45E-03	2.09E-03	3.72E-03	6.92E-03	1.10E-02
5.00E-02	1.37E-03	3.16E-04	5.25E-04	1.10E-03	2.09E-03	3.39E-03
1.00E-01	4.43E-04	6.76E-05	1.29E-04	3.24E-04	7.24E-04	1.23E-03
2.00E-01	1.14E-04	1.23E-05	2.51E-05	6.92E-05	1.91E-04	3.72E-04
3.00E-01	4.66E-05	4.47E-06	9.33E-06	2.63E-05	7.76E-05	1.59E-04
4.00E-01	2.41E-05	2.04E-06	4.57E-06	1.35E-05	4.07E-05	8.32E-05
5.00E-01	1.43E-05	1.05E-06	2.57E-06	7.94E-06	2.40E-05	4.90E-05
6.00E-01	9.26E-06	5.25E-07	1.62E-06	5.13E-06	1.62E-05	3.24E-05
8.00E-01	4.66E-06	1.78E-07	7.08E-07	2.51E-06	8.13E-06	1.70E-05
1.00E+00	2.71E-06	7.59E-08	3.55E-07	1.38E-06	4.79E-06	1.02E-05
1.50E+00	9.81E-07	1.07E-08	8.51E-08	4.47E-07	1.74E-06	3.72E-06
2.00E+00	4.55E-07	2.75E-09	2.82E-08	1.78E-07	8.13E-07	1.82E-06
3.00E+00	1.40E-07	9.12E-10	5.89E-09	4.27E-08	2.40E-07	5.75E-07
5.00E+00	2.61E-08	< 1E-10*	1.20E-09	6.17E-09	4.27E-08	1.18E-07

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-215
PSHA Results for 25-Hz Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

25-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	4.14E-02	2.24E-02	2.88E-02	3.80E-02	5.50E-02	6.76E-02
3.00E-03	2.48E-02	1.00E-02	1.35E-02	2.04E-02	3.72E-02	5.37E-02
1.00E-02	1.05E-02	3.02E-03	4.17E-03	7.24E-03	1.62E-02	3.02E-02
2.00E-02	5.54E-03	1.29E-03	1.91E-03	3.55E-03	8.32E-03	1.70E-02
5.00E-02	1.88E-03	2.63E-04	4.79E-04	1.12E-03	2.75E-03	6.03E-03
1.00E-01	6.66E-04	6.03E-05	1.20E-04	3.47E-04	9.77E-04	2.19E-03
2.00E-01	1.98E-04	1.35E-05	2.82E-05	8.71E-05	2.75E-04	7.08E-04
3.00E-01	9.12E-05	5.37E-06	1.23E-05	3.80E-05	1.23E-04	3.24E-04
4.00E-01	5.14E-05	2.69E-06	6.61E-06	2.14E-05	6.76E-05	1.82E-04
5.00E-01	3.25E-05	1.35E-06	3.89E-06	1.35E-05	4.37E-05	1.18E-04
7.50E-01	1.39E-05	3.63E-07	1.45E-06	5.75E-06	2.04E-05	5.13E-05
1.00E+00	7.54E-06	1.29E-07	6.76E-07	3.16E-06	1.18E-05	2.82E-05
1.50E+00	3.15E-06	1.86E-08	1.86E-07	1.23E-06	5.13E-06	1.26E-05
2.00E+00	1.68E-06	4.27E-09	6.76E-08	5.89E-07	2.75E-06	7.08E-06
5.00E+00	1.90E-07	< 1E-10*	2.04E-09	3.39E-08	2.46E-07	9.33E-07
7.00E+00	7.53E-08	< 1E-10*	1.00E-09	8.91E-09	7.94E-08	3.72E-07

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-216
PSHA Results for the 100-Hz (PGA) Spectral Acceleration on CEUS Generic Hard Rock for the HAR Site

100-Hz Spectral Acceleration (g)	Annual Exceedance Frequency					
	Mean	5th%	16th%	50th%	84th%	95th%
1.00E-03	3.30E-02	1.51E-02	1.91E-02	2.95E-02	4.79E-02	6.03E-02
2.00E-03	2.15E-02	7.76E-03	1.02E-02	1.78E-02	3.39E-02	4.79E-02
5.00E-03	9.94E-03	2.95E-03	3.98E-03	7.24E-03	1.59E-02	2.63E-02
1.00E-02	4.73E-03	1.32E-03	1.91E-03	3.39E-03	7.41E-03	1.32E-02
2.00E-02	1.95E-03	4.17E-04	6.92E-04	1.41E-03	3.02E-03	5.37E-03
5.00E-02	4.49E-04	5.25E-05	1.02E-04	2.88E-04	7.59E-04	1.38E-03
1.00E-01	1.13E-04	9.77E-06	2.00E-05	6.03E-05	1.86E-04	3.89E-04
2.00E-0	2.48E-05	1.41E-06	4.07E-06	1.32E-05	3.98E-05	8.51E-05
3.00E-01	1.03E-05	3.39E-07	1.41E-06	5.25E-06	1.74E-05	3.63E-05
4.00E-01	5.57E-06	1.02E-07	5.89E-07	2.75E-06	9.77E-06	2.09E-05
5.00E-01	3.48E-06	2.69E-08	2.75E-07	1.62E-06	6.03E-06	1.32E-05
6.00E-01	2.36E-06	9.33E-09	1.45E-07	1.02E-06	4.07E-06	9.12E-06
8.00E-01	1.25E-06	2.24E-09	4.79E-08	4.68E-07	2.14E-06	5.01E-06
1.00E+00	7.42E-07	1.12E-09	1.86E-08	2.34E-07	1.26E-06	3.24E-06
2.00E+00	1.14E-07	< 1E-10*	1.41E-09	1.82E-08	1.62E-07	5.50E-07
7.00E+00	7.53E-08	< 1E-10*	1.00E-09	8.91E-09	7.94E-08	3.72E-07

Notes:
*PSHA software assigns a value of 0 to exceedance frequencies $<10^{-10}$

HAR COL 2.5-2

Table 2.5.2-217
Equal-Hazard Generic CEUS Hard Rock Spectra for the HAR Site

Spectral Frequency (Hz)	Spectral Acceleration (g) for Exceedance Frequency of:			
	Mean 10^{-3}	Mean 10^{-4}	Mean 10^{-5}	Mean 10^{-6}
100 (PGA)	0.0303	0.106	0.304	0.880
25	0.0762	0.286	0.876	2.487
10	0.0615	0.212	0.581	1.488
5	0.0496	0.172	0.425	0.953
2.5	0.0318	0.122	0.289	0.588
1	0.0143	0.060	0.148	0.300
0.5	0.0072	0.036	0.099	0.213

HAR COL 2.5-2

Table 2.5.2-218
Reference and Deaggregation Earthquakes Developed from Deaggregation of the CEUS Generic Hard Rock Hazard Results

Annual Frequency of Exceedance and Spectra Frequencies	Reference (Controlling) Earthquake		Deaggregation Earthquakes		
	Magnitude (m_b)	Distance (km)	Magnitude (m_b)	Distance (km)	Weight
Mean 10^{-3} 5 and 10 Hz	6.5	173	5.4	52	0.214
			6.0	182	0.195
			7.0	262	0.591
Mean 10^{-3} 1 and 2.5 Hz	6.8 6.8[a]	227 263[a]	5.5	34	0.064
			6.1	203	0.157
			7.0	270	0.779
Mean 10^{-4} 5 and 10 Hz	6.6	108	5.4	17.8	0.271
			6.2	88	0.089
			7.1	239	0.640
Mean 10^{-4} 1 and 2.5 Hz	7.0 7.1[a]	203 251[a]	5.5	14.6	0.054
			6.2	88	0.051
			7.1	250	0.895
Mean 10^{-5} 5 and 10 Hz	6.3	40	5.4	9.5	0.472
			6.3	24	0.117
			7.2	217	0.411
Mean 10^{-5} 1 and 2.5 Hz	7.0 7.2[a]	160 239[a]	5.6	10.0	0.076
			6.3	25	0.067
			7.2	236	0.857
Mean 10^{-6} 5 and 10 Hz	6.0	15.1	5.5	8.4	0.607
			6.3	13.2	0.241
			7.2	190	0.152
Mean 10^{-6} 1 and 2.5 Hz	6.9 7.2[a]	101 229[a]	5.6	8.7	0.109
			6.4	14.5	0.157
			7.2	221	0.734

Notes:
a) Computed using earthquakes with distances >100 km (60 mi.).

HAR COL 2.5-2

Table 2.5.2-219
Damping Ratios for Sedimentary Rock

Layer Thickness (ft.)	Shear Wave Velocity (fps)	Equivalent Damping Ratio (%) for:		
		κ = 0.0012 sec	κ = 0.0049 sec	κ = 0.0104 sec
Profile HAR 2				
50	6095	0.10%	0.39%	0.83%
4	4145	0.14%	0.58%	1.22%
1050	6650	0.09%	0.36%	0.76%
2100	7000	0.08%	0.34%	0.72%
2000	7600	0.08%	0.31%	0.67%
Profile HAR 3a				
1094	6650	0.09%	0.36%	0.77%
2100	7000	0.08%	0.34%	0.73%
2000	7600	0.08%	0.32%	0.67%
Profile HAR 3b				
18	5500	0.11%	0.44%	0.93%
1045	6650	0.09%	0.36%	0.77%
2100	7000	0.08%	0.34%	0.73%
2000	7600	0.08%	0.32%	0.67%

HAR COL 2.5-2

Table 2.5.2-220 (Sheet 1 of 2)
Time History Data Sets Used for Each Deaggregation Earthquake

Hazard	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	5.4	52	0.214	**M** 4.5 – 6, D 0-50 km
	HF DEM	6.0	182	0.195	**M** 6 – 7, D 100-200 km
	HF DEH	7.0	262	0.591	**M** > 7, D 100-200 km
Mean 10^{-4} 1 and 2.5 Hz	LF DEL	5.5	34	0.064	**M** 4.5 – 6, D 0-50 km
	LF DEM	6.1	203	0.157	**M** 6 – 7, D 100-200 km
	LF DEH	7.0	270	0.779	**M** > 7, D 100-200 km
Mean 10^{-4} 5 and 10 Hz	HF DEL	5.4	17.8	0.271	**M** 4.5 – 6, D 0-50 km
	HF DEM	6.2	88	0.089	**M** 6 – 7, D 50-100 km
	HF DEH	7.1	239	0.640	**M** > 7, D 100-200 km
Mean 10^{-4} 1 and 2.5 Hz	LF DEL	5.5	14.6	0.054	**M** 4.5 – 6, D 0-50 km
	LF DEM	6.2	88	0.051	**M** 6 – 7, D 50-100 km
	LF DEH	7.1	250	0.895	**M** > 7, D 100-200 km
Mean 10^{-5} 5 and 10 Hz	HF DEL	5.4	9.5	0.472	**M** 4.5 – 6, D 0-50 km
	HF DEM	6.3	24	0.117	**M** 6 – 7, D 10-50 km
	HF DEH	7.2	217	0.411	**M** > 7, D 100-200 km
Mean 10^{-5} 1 and 2.5 Hz	LF DEL	5.6	10.0	0.076	**M** 4.5 – 6, D 0-50 km
	LF DEM	6.3	25	0.067	**M** 6 – 7, D 10-50 km
	LF DEH	7.2	236	0.857	**M** > 7, D 100-200 km

HAR COL 2.5-2

Table 2.5.2-220 (Sheet 2 of 2)
Time History Data Sets Used for Each Deaggregation Earthquake

	Deaggregation Earthquakes (DE)				
Hazard	Designation	Magnitude (m_b)	Distance (km)	Weight	NUREG/CR-6728 CEUS Data Set
Mean 10^{-6} 5 and 10 Hz	HF DEL	5.5	8.4	0.607	**M** 4.5 – 6, D 0-50 km
	HF DEM	6.3	13.2	0.241	**M** 6 – 7, D 10-50 km
	HF DEH	7.2	190	0.152	**M** > 7, D 100-200 km
Mean 10^{-6} 1 and 2.5 Hz	LF DEL	5.6	8.7	0.109	**M** 4.5 – 6, D 0-50 km
	LF DEM	6.4	14.5	0.157	**M** 6 – 7, D 10-50 km
	LF DEH	7.2	221	0.734	**M** > 7, D 100-200 km

Notes:
Source: Reference 2.5.2-255.

HAR COL 2.5-2

Table 2.5.2-221
Effect of CAV on Surface Spectra

Spectral Frequency (Hz)	Spectral Accelerations Computed with CAV / Spectral Accelerations Computed without CAV					
	HAR 2			HAR 3		
	10^{-4}	10^{-5}	10^{-6}	10^{-4}	10^{-5}	10^{-6}
100	0.7692	0.8690	1.0847	0.7920	0.8882	1.0911
25	0.5796	0.8612	1.0600	0.6399	0.8859	1.0712
10	0.6740	0.8922	1.0033	0.7519	0.9086	1.0153
5	0.6614	0.9330	0.9806	0.7122	0.9440	0.9846
2.5	0.6699	0.9395	0.9836	0.7254	0.9496	0.9867
1	0.5832	0.9128	0.9816	0.6593	0.9276	0.9853
0.5	0.4597	0.8880	0.9711	0.5367	0.9081	0.9765

HAR COL 2.5-3

Table 2.5.2-222 (Sheet 1 of 3)
5 Percent Damped GMRS Spectra for HAR Site

Spectral Frequency (Hz)	GMRS Spectral Acceleration (g)			
	HAR 2		HAR 3	
	Horizontal	Vertical	Horizontal	Vertical
100.000	0.1228	0.0958	0.1374	0.1071
60.241	0.1980	0.1769	0.2165	0.1934
50.000	0.2351	0.2024	0.2598	0.2236
40.000	0.2830	0.2344	0.3120	0.2585
33.333	0.3165	0.2481	0.3569	0.2798
30.303	0.3242	0.2487	0.3787	0.2906
25.000	0.3337	0.2503	0.4231	0.3174
23.810	0.3327	0.2469	0.4248	0.3152
22.727	0.3317	0.2437	0.4262	0.3131
21.739	0.3308	0.2405	0.4264	0.3100
20.833	0.3299	0.2373	0.4222	0.3036
20.000	0.3290	0.2342	0.4208	0.2995
18.182	0.3231	0.2257	0.4061	0.2837
16.667	0.3191	0.2216	0.3935	0.2732
15.385	0.3155	0.2178	0.3809	0.2630
14.286	0.3122	0.2144	0.3696	0.2539
13.333	0.3091	0.2113	0.3594	0.2456
12.500	0.3063	0.2084	0.3500	0.2382
11.765	0.3003	0.2035	0.3415	0.2314
11.111	0.2948	0.1989	0.3337	0.2252
10.526	0.2896	0.1947	0.3264	0.2195
10.000	0.2848	0.1908	0.3196	0.2142
9.091	0.2738	0.1834	0.3056	0.2048
8.333	0.2640	0.1769	0.2933	0.1965
7.692	0.2554	0.1711	0.2825	0.1893
7.143	0.2476	0.1659	0.2728	0.1828
6.667	0.2406	0.1612	0.2641	0.1769
6.250	0.2342	0.1569	0.2561	0.1716
5.882	0.2284	0.1530	0.2489	0.1668
5.556	0.2230	0.1494	0.2423	0.1623
5.263	0.2180	0.1461	0.2362	0.1583

HAR COL 2.5-3

Table 2.5.2-222 (Sheet 2 of 3)
5 Percent Damped GMRS Spectra for HAR Site

Spectral Frequency (Hz)	GMRS Spectral Acceleration (g)			
	HAR 2		HAR 3	
	Horizontal	Vertical	Horizontal	Vertical
5.000	0.2134	0.1430	0.2306	0.1545
4.545	0.2036	0.1364	0.2189	0.1467
4.167	0.1949	0.1306	0.2088	0.1399
3.846	0.1873	0.1255	0.1999	0.1340
3.571	0.1806	0.1210	0.1920	0.1287
3.333	0.1745	0.1169	0.1850	0.1239
3.125	0.1690	0.1132	0.1786	0.1197
2.941	0.1639	0.1098	0.1728	0.1158
2.778	0.1593	0.1068	0.1675	0.1122
2.632	0.1551	0.1039	0.1627	0.1090
2.500	0.1512	0.1013	0.1582	0.1060
2.381	0.1473	0.0987	0.1540	0.1032
2.273	0.1437	0.0963	0.1501	0.1006
2.174	0.1403	0.0940	0.1465	0.0982
2.083	0.1372	0.0919	0.1432	0.0959
2.000	0.1342	0.0899	0.1400	0.0938
1.818	0.1254	0.0840	0.1309	0.0877
1.667	0.1179	0.0790	0.1232	0.0825
1.538	0.1114	0.0747	0.1164	0.0780
1.429	0.1043	0.0699	0.1089	0.0729
1.333	0.0982	0.0658	0.1023	0.0685
1.250	0.0927	0.0621	0.0965	0.0647
1.176	0.0879	0.0589	0.0913	0.0612
1.111	0.0835	0.0560	0.0867	0.0581
1.053	0.0790	0.0529	0.0819	0.0549
1.000	0.0749	0.0502	0.0776	0.0520
0.909	0.0702	0.0470	0.0730	0.0489
0.833	0.0662	0.0443	0.0690	0.0462
0.769	0.0631	0.0423	0.0655	0.0439
0.714	0.0605	0.0405	0.0625	0.0418

HAR COL 2.5-3

Table 2.5.2-222 (Sheet 3 of 3)
5 Percent Damped GMRS Spectra for HAR Site

Spectral Frequency (Hz)	GMRS Spectral Acceleration (g)			
	HAR 2		HAR 3	
	Horizontal	Vertical	Horizontal	Vertical
0.667	0.0581	0.0389	0.0597	0.0400
0.625	0.0559	0.0375	0.0574	0.0384
0.588	0.0540	0.0362	0.0555	0.0372
0.556	0.0522	0.0350	0.0537	0.0360
0.526	0.0506	0.0339	0.0521	0.0349
0.500	0.0491	0.0329	0.0506	0.0339
0.455	0.0434	0.0291	0.0448	0.0300
0.417	0.0387	0.0259	0.0400	0.0268
0.385	0.0349	0.0234	0.0361	0.0242
0.357	0.0317	0.0212	0.0328	0.0220
0.333	0.0290	0.0194	0.0300	0.0201
0.313	0.0263	0.0177	0.0273	0.0183
0.294	0.0241	0.0161	0.0250	0.0167
0.278	0.0222	0.0148	0.0229	0.0154
0.263	0.0205	0.0137	0.0212	0.0142
0.250	0.0190	0.0127	0.0197	0.0132
0.238	0.0176	0.0118	0.0183	0.0123
0.227	0.0165	0.0110	0.0171	0.0114
0.217	0.0154	0.0103	0.0160	0.0107
0.208	0.0145	0.0097	0.0150	0.0101
0.200	0.0136	0.0091	0.0141	0.0095
0.182	0.0118	0.0079	0.0123	0.0082
0.167	0.0104	0.0069	0.0108	0.0072
0.154	0.0092	0.0061	0.0095	0.0064
0.143	0.0082	0.0055	0.0085	0.0057
0.133	0.0074	0.0049	0.0077	0.0051
0.125	0.0066	0.0045	0.0069	0.0046
0.118	0.0060	0.0040	0.0063	0.0042
0.111	0.0055	0.0037	0.0057	0.0038
0.100	0.0046	0.0031	0.0048	0.0032

2.5.3 SURFACE FAULTING

HAR COL 2.5-4 This subsection describes the evidence used to evaluate the potential for future surface faulting and related capable tectonic deformation at the HAR site and surrounding site area. The following aspects of the geology and seismicity of the site region are discussed:

- Geological, seismological, and geophysical investigations (Subsection 2.5.3.1).
- Geological evidence, or lack thereof, of surface deformation (Subsection 2.5.3.2).
- Earthquakes associated with capable tectonic sources (Subsection 2.5.3.3).
- Ages of most recent deformation (Subsection 2.5.3.4).
- Relationship between tectonic structures in the site area and regional tectonic structures (Subsection 2.5.3.5).
- Characterization of identified capable tectonic sources (Subsection 2.5.3.6).
- Designation of zones of Quaternary deformation in the site region (Subsection 2.5.3.7).
- Potential for surface tectonic deformation at the site (Subsection 2.5.3.8).

Results of the surface faulting study indicate that there is no evidence of Quaternary tectonic surface faulting or fold deformation at the HAR site, and no capable tectonic sources have been identified within 40 km (25 mi.) of the site. In accordance with Regulatory Guide 1.208, a capable tectonic source is defined as a tectonic structure that can generate both vibratory ground motion and tectonic surface deformation, such as faulting or folding at or near the earth’s surface in the present seismotectonic regime.

2.5.3.1 Geological, Seismological, and Geophysical Investigations

Investigations that have been performed to evaluate the potential for surface fault rupture at the HAR site, as well as the surrounding HAR site area, include the following:

- Compilation and review of existing data and literature.
- Lineament analyses.
- Discussions with current researchers in the area.

- Field reconnaissance.
- Review of seismicity data.

2.5.3.1.1 Compilation and Review of Existing Data and Literature

An extensive body of existing information is available regarding faulting in the HAR site area. A map showing the locations of fault investigation trenches and geophysical surveys completed near the HAR site is shown on Figure 2.5.3-201. The following principal sources of data were used:

- Documents developed in support of licensing of the HNP. These include the HNP FSAR (Reference 2.5.3-201), a two-volume report entitled "Fault Investigation: Shearon Harris Nuclear Power Plant, Units 1, 2, 3, 4" (Reference 2.5.3-202), and reports summarizing faulting investigations in the Main Dam and auxiliary areas (Reference 2.5.3-203).
- U.S. Geological Survey open-file report by Bain and Brown (Reference 2.5.3-214) that summarizes detailed geophysical and remote sensing studies conducted by the USGS to evaluate the potential waste storage potential of the Durham basin.
- Site characterization reports and data collected in support of a proposed LLRW disposal facility site in Wake County adjacent to and west of the existing HNP site (Reference 2.5.3-204).
- Investigations of faults exposed at two borrow pits approximately 2.4 km (1.5 mi.) west of the HAR site (Reference 2.5.3-205, Reference 2.5.3-206).
- Published geologic maps and unpublished maps and data made available by the NCGS (Reference 2.5.3-207, Reference 2.5.3-208).
- Seismicity data from published literature, recent analysis of historical seismicity in the region (Reference 2.5.3-209), and analysis completed for this study.

2.5.3.1.2 Lineament Analyses

Investigations for the HNP (Reference 2.5.3-201, Reference 2.5.3-202) involved extensive interpretation of aerial photographs and other remote sensing imagery, including conventional high- and low-altitude aerial photography (including false color enhancement), and Skylab and Landsat imagery. Several hundred lineaments were identified by these investigations. Field checking included at least one checkpoint along each significant lineament, with greater emphasis on those lineaments closest to the site. No linear features were identified as capable faults based on the imagery analysis, and none of the lineaments that were field checked were identified as faults. (Reference 2.5.3-201)

LIDAR data released by the North Carolina Department of Transportation in March 2005 provide improved, more detailed images of surface topography than was available during the HNP FSAR study (Reference 2.5.3-201). LIDAR data are available for most of the state of North Carolina at a grid size of 46 m (150 ft.), and at the county level at a grid size of 6 m (20 ft.). The more detailed county-level data were used in this study to identify lineaments within the site area. Lineaments were mapped using hillshade models of the 6 m (20 ft.) grid elevation data for Wake, Chatham, Lee, and Harnett counties. Each elevation location has been rounded to the nearest foot and represents the average elevation for the entire grid cell. The vertical accuracy of the LIDAR data is about 25 cm (10 in.). (Reference 2.5.3-210) Hillshade models were generated from county elevation data using ArcGIS Spatial Analyst (azimuth = 315°, altitude = 45, z factor = 1) with an output grid cell size of 6 m (20 ft.) Lineaments were identified on the hillshade models in ArcGIS for the site area's 8-km (5-mi.) radius, and are discussed in Subsection 2.5.3.2.1.

2.5.3.1.3 Discussions with Current Researchers in the Area

Researchers were contacted who were familiar with the structural and tectonic framework of the region, Coastal Plain stratigraphy, and post-Cretaceous faulting in the Coastal Plain; these researchers provided recently published and in-press publications for review. Mr. Timothy (Tyler) Clark, former head geologist for the NCGS, led a field trip to provide an overview of Mesozoic rift basin stratigraphy in the site area and participated in field and helicopter reconnaissance to investigate faults in the site vicinity.

2.5.3.1.4 Field Reconnaissance

Field reconnaissance was conducted as part of the HAR site characterization activities. The field investigations focused on (1) a review of the geology of the site location (within approximately 1 km [0.6 mi.] of the HAR 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 vicinity and surrounding region (e.g., the postulated ECFS [Reference 2.5.3-211] and "fall lines" of Weems [Reference 2.5.3-212]). An aerial reconnaissance was conducted to further evaluate these features.

2.5.3.1.5 Review of Seismicity Data

A comprehensive review of both instrumental and historical earthquakes was completed for the HAR study (see Subsection 2.5.2.1). A map showing seismicity within an 80-km (50-mi.) radius of the site is shown on Figure 2.5.1-255.

2.5.3.2 Geological Evidence, or Absence of Evidence, for Surface Deformation

The HAR site area (an 8-km [5-mi.] radius) sits largely within the Deep River basin, a north- to northeast-trending half-graben. The HAR site is located approximately 6.4 km (4 mi.) north of the Jonesboro fault, the major west-dipping, high-angle, normal boundary fault that separates the Triassic sedimentary rocks from the Raleigh metamorphic belt and the Carolina zone metavolcanic and metasedimentary rocks (Figures 2.5.1-230, 2.5.1-231, and 2.5.1-232). As illustrated on Figure 2.5.1-240, the site sits in the hanging wall of the Jonesboro fault, which locally is disrupted by distributed synthetic and antithetic faults. Small-scale intrabasin and cross-basin faults in the site area also have been identified based on surface geological investigations, subsurface trenching and drilling, and interpretation of seismic reflection data. The closest, well-documented faults to the HAR site include the Harris fault, the SBPF, and the W8 and W82 faults (Figure 2.5.1-232). Additional minor faults were exposed within igneous and metamorphic rocks in the foundations of the Main Dam structures approximately 8 km (5 mi.) south of the HAR 2 site (Figure 2.5.1-230, Reference 2.5.3-203). These structures are described in detail in Subsection 2.5.1.2.4.

Investigations conducted for the HNP after the Harris fault was identified also included extensive use of remote sensing techniques to seek other linear features in the site region and area (Reference 2.5.3-202). The lineaments identified in the site area by these investigations are shown on Figure 2.5.3-202. Although several hundred lineaments were identified using these aerial photograph and remote sensing techniques, no linear features were identified as capable faults on the basis of imagery evaluation, nor were any that were field checked identified as faults. The Site fault was undetected by any imagery technique. (Reference 2.5.3-201)

A more recent technique known as LIDAR (light detection and ranging) provides improved images of surface topography. In contrast to the previous conclusions based on the lineament analysis completed as part of the HNP FSAR (Reference 2.5.3-201), a possible extension of the Harris fault and other east-southeast-trending faults identified in the LLRW disposal facility area do appear to coincide locally with lineaments identified in the LIDAR data. The Harris fault, the SBPF, and the W8 and W82 faults characterized in the LLRW disposal facility site to the west all appear to be associated with a discontinuous set of east-to-west-trending lineaments that extend across the site area (Figure 2.5.3-203). The lineaments that are closely associated with mapped faults appear to reflect differential erosion along zones of more fractured bedrock. No evidence to indicate that the faults are capable tectonic sources was identified from the lineament analysis. The results of the LIDAR lineament analysis are discussed in Subsection 2.5.3.2.1.

2.5.3.2.1 Results of Lineament Analysis

As described in Subsection 2.5.3.1, a lineament analysis was undertaken as part of the HAR study to identify and characterize lineaments in the site area that might intersect the HAR site. Previous investigations for the HNP (Reference 2.5.3-201, Reference 2.5.3-202) involved extensive aerial photograph interpretation and other remote sensing techniques including conventional high- and low-altitude aerial photograph, SLAR, including false color enhancement, and Skylab and Landsat imagery (Reference 2.5.3-201). The HNP FSAR stated, “No linear features were identified as capable faults on the basis of imagery evaluation, nor were any that were field checked identified as faults. The Harris fault was undetected by any imagery technique” (Reference 2.5.3-201). Figure 2.5.3-202 shows the previously interpreted lineaments, mapped faults, and dikes in the site area superimposed on the LIDAR image. Although some of the previously identified lineaments correspond to lineaments that can be identified using the new LIDAR data, many of them do not correspond to topographic lineaments and may have been representative of cultural features. Additionally, lineaments observed from the NCGS SLAR and Landsat imagery were identified in the field or on maps as diabase dikes, faults, lithologic contacts, streams, roads, power lines, and pipelines (Reference 2.5.3-214). In the SLAR imagery, faults and fractures appeared as linear depressions (Reference 2.5.3-214).

Several trends of lineaments are apparent in the LIDAR data (Figures 2.5.3-203 and 2.5.3-207). The main trends are between N30W to N30E and east to west to N60E or N60W (Figure 2.5.3-207). Many of these, which are defined by small drainages, appear to reflect the dominant joint and fracture sets that are recognized in the region. As reported in the HNP FSAR (Reference 2.5.3-201), three joint sets are present in the Triassic sedimentary rocks: the two dominant sets are approximately vertical, one striking N40 to 50E and the other N20 to 30W; a third set strikes north to northwest and dips 55 to 70 degrees to the southwest. Bedding in the Triassic in the site area primarily is between north to N35E, and it appears that many of the small drainages are parallel to bedding and probably formed by differential weathering of the more easily eroded beds. The dikes generally are aligned subparallel to the N20 to 30W trend, but shorter east-west-trending dikes have been mapped in the New Hill 7.5-minute quadrangle (Figure 2.5.1-231). (Reference 2.5.3-208) East-to-west-oriented lineaments also are apparent in the LIDAR data. Lineament analysis by Bain and Brown (Reference 2.5.3-214) from SLAR data could not identify east-to-west lineaments because of the flight direction. Bain and Brown (Figure 2.5.3-207) also identified more distinct preferred directions of lineaments compared to the LIDAR analysis in this study (Reference 2.5.3-214). This may be due in part to cultural features identified in the SLAR imagery.

In contrast to the previous conclusions based on the lineament analysis completed as part of the HNP FSAR, what appears to be an easterly extension of the Harris fault and other east-to-southeast-trending faults identified in the LLRW disposal facility area, as well as portions of the Jonesboro fault, do appear to

coincide with lineaments identified in the LIDAR data (Reference 2.5.3-201). The possible eastward extension of the Harris fault, as well as the SBPF and W82 faults, characterized in the LLRW disposal facility site to the west all appear to coincide with a set of generally east-to-west-trending lineaments that can be traced across the site area (Figure 2.5.3-203). Although the faults do not directly correspond to lineaments defined by the drainage bottoms, it is likely that the drainages are localized in the more fractured and more easily eroded areas on the hanging wall, as seen at the LLRW disposal facility (Reference 2.5.3-204).

One of the longer of these east-west-trending lineaments extends across the northernmost part of the HAR site location, approximately 150 m (500 ft.) north of the HAR 3 site (Figure 2.5.3-206). The lineament, which trends east to southeast, is identified primarily by a series of aligned drainages, including the drainage currently impounded to form the fire pond. The HNP, HAR 2, and HAR 3 sites appear to lie between the Harris fault and this lineament, which is referred to as the fire pond lineament (FPL).

The origin of the FPL is unknown. There are no surface exposures of bedrock where the lineament crosses Old Highway 1 approximately 2.9 km (1.8 mi.) southwest of Bonsal (point FPL-1), where it crosses Highway 1 (point FPL-2), along the railroad and stream cuts northwest of HAR 3 (FPL-3), or along the projected trend of the lineament where it crosses a spur off SR 1134 (FPL-4), and in the stream approximately 100 m (300 ft.) west of the main access road into the HNP site (SR 1134 [FPL-5]) (Figure 2.5.3-206).

The closest exposure of bedrock to the FPL is located along the outflow channel to the fire pond northeast of the HAR 3 site. Exposures of interbedded siltstones and sandstones of Lithofacies I Unit Trcs/si2 (Figure 2.5.3-208) in the outflow channel for the fire pond north of HAR 3 display four prominent fracture sets: N5E, N30 to 50W, N35E, and N85W. The N85W trending fracture is open and parallels the trend of the FPL (Figure 2.5.3-209). Bedding is oriented approximately N15E, and dips approximately 17 to 30 degrees southeast at this location. No evidence of faulting or disruption of the soils formed in these deposits was observed.

East-west-trending lineaments along the projected trends of the FPL and the Harris faults intersect the Texaco seismic line 85SD12 (Figure 2.5.3-203). These lineaments appear to be associated with an approximately 5-km- (3-mi.-) wide zone of secondary faults in the hanging wall of the Jonesboro fault (Figure 2.5.1-241). Based on the possible alignments of the FPL and Harris fault with the lineaments associated with the faults identified in the Texaco seismic line, it is reasonable to assume that the HNP, HAR 2, and HAR 3 sites lie within a similar zone of secondary, small normal faults in the hanging wall of the Jonesboro fault. It is noted, however, that the FPL lineament cannot be directly linked to the east-west-trending lineament that crosses the seismic line.

The FPL may be related to a secondary fault of similar age to the Harris fault, based on similarities in orientation and geomorphic expression. If the FPL is

coincident with a fault, it is likely that the lineament represents differential erosion of more highly fractured or deformed rock. No scarps were observed across the projected trends of the FPL during field reconnaissance conducted for this study. There is no evidence to indicate that the FPL is the surface expression of a capable tectonic source.

Another well-expressed LIDAR lineament approximately 21 km (13 mi.) northwest of the site trends N60E and corresponds to the Bush Creek fault mapped by the NCGS (Reference 2.5.3-208). Where the lineament that corresponds to the Bush Creek fault crosses seismic line 85SD12, a zone of primarily down-to-the-south normal faulting is observed in the seismic profile (Figures 2.5.1-236 and 2.5.1-241). There is no reported evidence of Quaternary deformation along this feature.

2.5.3.3 Correlation of Earthquakes with Capable Tectonic Sources

There have been no historically reported earthquakes or alignments of earthquakes within 40 km (25 mi.) of the site that can be associated with a mapped bedrock fault (see Subsection 2.5.2.1). No earthquakes greater than m_b = 3.0 are identified in the site vicinity, and the largest and only earthquake within an 80-km (50-mi.) radius of the site was an m_b = 3.3 event that occurred in 1896.

2.5.3.4 Ages of Most Recent Deformations

Detailed studies to determine the age of mapped faults in the study area were performed as part of the HNP fault investigations (Reference 2.5.3-202, Reference 2.5.3-201). The age of last movement on the Jonesboro fault is bracketed between the intrusion of Late Triassic – Jurassic dikes and the deposition of the overlying unfaulted Cretaceous marine sediments, between 180 and 135 Ma (Reference 2.5.3-202).

Diabase dikes, secondary minerals in fault gouge adjacent to the dikes, and soils were used to constrain the age of the last movement on the Harris fault. Detailed studies to determine the age of most recent deformation on the Harris fault are described in Subsection 2.5.3.2.1. Movement along the Harris fault occurred after deposition and lithification of several thousand feet of Triassic basin sediments and ended shortly after intrusion of the latest of the Jurassic dikes (Reference 2.5.3-201). Field observations of undisturbed soil, saprolite, and in one place, a post-Triassic sedimentary deposit overlying the fault, indicated that the most recent movement was probably more than one million years ago (Reference 2.5.3-201). Secondary zeolite minerals that formed in the fault zone along the Harris fault during thermal conditions that were last present in the region over 150 Ma are undeformed, indicating that the most recent movement predated this time (Reference 2.5.3-201). According to NUREG-1038, the NRC staff concurred that all the faults in the HNP site and Main Dam areas predate the mineralization that formed after regional deformation — at least 2.5 Ma, and likely more than 136 to 190 Ma.

No detailed geochronologic data exist for the borrow pit faults (SBPF, FA, FB, FC), fault W8, or fault W82; however, geologic relations suggest these faults were last active in the Triassic period (see Subsection 2.5.3.1.4 and Subsection 2.5.3.1.5 for a more detailed discussion).

2.5.3.5 Relationship of Tectonic Sources in the Site Area to Regional Tectonic Structures

Mapped surface bedrock faults within the site area (8-km [5-mi.] radius) are primarily related to the formation of the Mesozoic Deep River basin. There is no new information to suggest that the faults associated with the Mesozoic basins in the site region are capable tectonic sources as defined by Regulatory Guide 1.208 (Appendix A).

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 bullet item 1 above, such that movement on one could reasonably be expected to be accompanied by movement on the other.

None of the mapped bedrock faults within a 40-km (25-mi.) radius or possible fault-related lineaments (e.g., the FPL) within an 8-km (5-mi.) radius of the HAR site is assessed to be a capable tectonic source. This conclusion is based on the following lines of evidence as discussed in the previous subsections:

- The Jonesboro fault is overlain by undeformed Cretaceous sediments (Reference 2.5.3-202).
- Detailed investigations to evaluate the Harris fault, including trenching, drilling, and mapping, have provided detailed geochronologic data that demonstrate faulting is likely older than 150 Ma.
- Structural analysis of the development of faults and fractures exposed at the borrow pit and LLRW areas suggest that these structures predate or formed during the same stress regime as the Harris fault.

- Evidence observed in trenches shows that the faults identified at the LLRW disposal facility site, approximately 1.5 mi. west of the HAR site, predate significant weathering and soil formation or exhibit no evidence of recent faulting (Reference 2.5.3-206).

- No evidence of Quaternary deformation is reported in the literature or was observed during field and aerial reconnaissance conducted for this study.

- There are no associated historical earthquakes or alignments of seismicity to suggest the presence of a capable tectonic source in the site area.

- East-west-trending lineaments such as the FPL and the lineament along the eastward projection of the Harris fault if related to faults can be explained as having been caused by differential erosion of fractured rock associated with secondary faults in the hanging wall of the Jonesboro fault. There is no surface expression of recent faulting along the FPL.

- Based on structural association with the Harris fault, it is expected that faults or folds that may be expressed as lineaments in the rift basin sediments in the site area likely formed during the same period of deformation and, therefore, are not capable tectonic sources. Excavation exposures for HAR safety-related facilities will be mapped in detail, and the surface rupture and ground-motion-generating potential of any deformation features identified will be assessed.

2.5.3.7 Designation of Zones of Quaternary Deformation in the Site Region

No zones of Quaternary deformation that would require additional investigation are identified within the HAR site area. Investigations of the Harris fault and other more minor faults observed in the Main Dam area showed that these faults are not capable, in accordance with 10 CFR 100, Appendix A (Reference 2.5.3-201). Review of existing HNP documents, mapping, and subsurface investigations conducted for this study identified no evidence for surface deformation at either the HAR 2 or the HAR 3 sites.

2.5.3.8 Potential for Surface Tectonic Deformation at the Site

The potential for tectonic deformation at the HAR site is assessed to be negligible. This conclusion is based on the following:

- The results of comprehensive fault investigations that have demonstrated that mapped faults within the site area are not capable faults (Reference 2.5.3-202, Reference 2.5.3-203).

- Evidence observed in trenches that shows that the faults identified at the LLRW disposal facility site, approximately 1.5 mi. west of the HAR site,

predate significant weathering and soil formation or exhibit no evidence of recent faulting (Reference 2.5.3-206).

- Bedrock geologic mapping in the site vicinity (Reference 2.5.3-208, Figure 2.5.1-231) that identified no evidence for surface faulting or deformation that would suggest capable faults in the HAR site area.

- The absence of geomorphic features indicative of Quaternary deformation as reported in the previous HNP reports and literature and inferred from observations made during the field reconnaissance conducted for this study.

The floors and walls of excavations for all safety-related structures for the HAR 2 and HAR 3 facilities will be mapped in detail, and the NRC will be notified immediately if previously unknown geologic features that could represent a hazard to the proposed facilities are identified. 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.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

HAR COL 2.5-1
HAR COL 2.5-2
HAR COL 2.5-3
HAR COL 2.5-5
HAR COL 2.5-6
HAR COL 2.5-7
HAR COL 2.5-8
HAR COL 2.5-9
HAR COL 2.5-10
HAR COL 2.5-11
HAR COL 2.5-12
HAR COL 2.5-13
HAR COL 2.5-16

This subsection presents information on the properties and stability of all soils and rock that may affect the nuclear power plant facilities, under both static and dynamic conditions, including vibratory ground motions associated with the GMRS. The discussion focuses on the stability of the materials as they influence the safety of Seismic Category 1 facilities (i.e., nuclear islands) and presents an evaluation of the site conditions and geologic features that may affect the power plant structures or their foundations.

This subsection is organized into subsubsections as presented in Regulatory Guide 1.206. These include:

- Geologic Features (Subsection 2.5.4.1).

- Properties of Subsurface Materials (Subsection 2.5.4.2).

- Foundation Interfaces (Subsection 2.5.4.3).

- Geophysical Surveys (Subsection 2.5.4.4).

- Excavations and Backfill (Subsection 2.5.4.5).

- Groundwater Conditions (Subsection 2.5.4.6).

- Response of Soil and Rock to Dynamic Loading (Subsection 2.5.4.7).

- Liquefaction Potential (Subsection 2.5.4.8).
- Earthquake Site Characteristics (Subsection 2.5.4.9).
- Static Stability (Subsection 2.5.4.10).
- Design Criteria (Subsection 2.5.4.11).
- Techniques to Improve Subsurface Conditions (Subsection 2.5.4.12).

Subsection headings and heading numbers follow Regulatory Guide 1.206 for Subsection 2.5.4, instead of the DCD headings. The combined license information section is included as Subsection 2.5.6.

2.5.4.1 Geologic Features

HAR COL 2.5-1

HAR 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 deformation or foundation instability at the HAR 2 and HAR 3 safety-related facilities. A summary of subsurface conditions at HAR 2 and HAR 3, based on the subsurface investigation, is first presented. 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; history of deposition and erosion; and estimates of preconsolidation pressures.

This discussion is based on the site geology summarized in Subsection 2.5.1, surface faulting described in Subsection 2.5.3, and the results of site-specific subsurface investigation activities presented in Subsection 2.5.4.2. Subsurface investigation locations are shown on Figures 2.5.4-201, 2.5.4-202, 2.5.4-203A, and 2.5.4-203B.

2.5.4.1.1 Summary of Subsurface Conditions at HAR 2 and HAR 3

The HAR 2 and HAR 3 nuclear islands will be founded at subgrade elevation 67.1 m (220 ft.), at locations shown on Figure 2.5.4-202. Table 2.5.4-201 presents a summary of subsurface conditions encountered at boreholes advanced near the AP1000 structures. Soil boring and rock coring logs are presented in Appendix 2BB. Figures 2.5.4-204A and 2.5.4-204B present geologic cross sections through HAR 2 based on these boreholes, and Figure 2.5.4-205A and 2.5.4-205B present geologic cross sections through HAR 3. The cross section locations are indicated on Figure 2.5.4-202.

The depth of soil and top of sound rock are indicated on the cross sections and in Table 2.5.4-201. The definitions used to characterize the top of weathered rock

(i.e., soil depth) and top of sound rock are presented in Subsection 2.5.4.2.2.7. The interpreted top of sound rock elevations and contours are shown on Figures 2.5.4-206A and 2.5.4-206B.

2.5.4.1.1.1 Description of Soil and Rock at HAR 2 and HAR 3

Surficial soils encountered at the HAR consist primarily of lean clays, sand, and silt that were formed in place by weathering of the parent bedrock. The depth of soil over rock typically is 1.5 to 4.6 m (5 to 15 ft.) at HAR 2, and 3.0 to 7.6 m (10 to 25 ft.) at HAR 3. The transitions from soil to weathered rock and to sound rock are gradual.

2.5.4.1.1.1.1 Rock Core Descriptions

Field descriptions of rock lithology were consistent with rock types reported for the HNP and with descriptions by the American Geological Institute (AGI) (Reference 2.5.4-201, Reference 2.5.4-202).

The following criteria were used by CH2M HILL field geologists and engineers to field-classify rock types during the HAR site investigations:

- Sandstone: A coarse-grained rock with greater than 50 percent apparently coarser than 0.05 millimeter (mm) (0.002 inch [in.]) diameter.

- Siltstone: A fine-grained rock with greater than 50 percent apparently finer than 0.05 mm (0.002 in.) diameter. Grains are predominantly visible by the unaided eye or with a hand lens. Siltstone may have visible bedding planes, but is not fissile (i.e., does not preferentially cleave along bedding planes).

- Claystone: A fine-grained rock with greater than 50 percent apparently finer than 0.05 mm (0.002 in.) diameter. Individual grains are not clearly visible with the unaided eye or with a hand lens. Claystone may have visible bedding planes, but is not fissile. Claystone was rarely encountered in the HAR site boreholes.

- Shale: A fine-grained rock that is fissile (i.e., cleaves predominantly along bedding planes). Shale is further described as either silty or clayey based on grain visibility. Shale was rarely encountered in the HAR site boreholes.

- Conglomerate: A coarse-grained rock with appreciable pebble- or gravel-size particles, was also rarely encountered in the HAR site boreholes.

As described in Subsection 2.5.4.2.3.1, 16 rock samples were submitted for petrographic laboratory examination. Based on particle counts of microscopic images of thin rock sections, four of the samples were classified as “mudstone” by the sedimentary rock classification system by Picard (Reference 2.5.4-203). This system identifies mudstone as containing an intermediate amount of both silt and clay-size particles, as an intermediate material between siltstone and

claystone. By contrast, the AGI describes mudstone as a fine-grained rock containing both siltstone and claystone (Reference 2.5.4-202).

Mudstone was not used as a lithologic term during field classification at the HAR sites, for several reasons. Field classification of the clay and silt fractions based on visual methods is not considered reliable enough to accurately identify mudstone compared with siltstone or claystone. Further, the term mudstone was not used for the HNP investigation; rock descriptions in the HNP are consistent with the AGI definitions. Siltstone, as described in this FSAR, therefore includes both siltstone and mudstone as classified using the Picard system. This is consistent with nomenclature conventions used by other recent researchers in the Durham basin, and is consistent with rock type descriptions presented in the HNP FSAR (Reference 2.5.4-204, Reference 2.5.4-203).

The following key observations were made regarding HAR site soil and rock conditions, as indicated on the borehole logs in Appendix 2BB; on cross sections on Figures 2.5.4-204A, 2.5.4-204B, 2.5.4-205A, and 2.5.4-205B; on top of sound rock elevations on Figures 2.5.4-206A and 2.5.4-206B; and in Table 2.5.4-201:

- The depth to sound rock is shallow at HAR 2, in part because surficial soil and weathered rock at HAR 2 were removed in the 1970s to create the existing site grade during construction of the HNP. At some boreholes north of HAR 2 (BPA-2 and BPA-6), depth to sound rock is greater than under the HAR 2 nuclear island.

- Surficial soils at HAR 3 were not removed nor disturbed during construction of the HNP, and the top of sound rock is therefore deeper than at HAR 2.

- Bedrock at the HAR sites consists predominantly of reddish-brown siltstone and reddish-brown to gray sandstone. Subordinate amounts of shale, conglomerate, and claystone were also occasionally encountered.

- Rock is typically sound and fresh below the nuclear island subgrade elevation of 67.1 m (220 ft.) at the HAR, as described in Subsection 2.5.4.2. Isolated intervals of altered rock and soil-like layers are present within otherwise sound rock in various locations, but such intervals are less than 1 m (about 3 ft.) thick except at Borehole BPA-6, as described in Subsection 2.5.4.1.3.

The continuity of individual rock strata varies both laterally and with depth as a result of the mechanisms of deposition of the Triassic basin. As observed on the cross sections of Figures 2.5.4-204A, 2.5.4-204B, 2.5.4-205A, and 2.5.4-205B, fine-grained sandstone and siltstone strata are often traceable across multiple boreholes at HAR 2 and HAR 3. Coarse-grained sandstone deposits associated with stream deposition exhibit more lateral variability than the finer grained deposits. Thicknesses of individual strata range from a few centimeters (few inches) to several meters (tens of feet). As observed during rock coring, individual strata typically transition compactly into one another and do not exhibit

structural weakness. Exceptions are present where thin clay seams are observed along bedding planes.

2.5.4.1.1.1.2 Clay Seams within Sound Rock

Thin clay seams encountered within otherwise sound rock were generally found intact, were bounded by surrounding bedrock, usually siltstone, and exhibited evidence of bedding. This indicates that these clay seams were formed through depositional processes, including fluvial and lacustrine deposits (Reference 2.5.4-204). The results of the petrographic examinations described in Subsection 2.5.4.2.3.1 indicate that compaction, and not cementation, was the primary cause of lithification for the fine-grained rock samples from the HAR site. It is considered likely that most of the clay seams encountered within otherwise sound rock have never fully lithified.

Other clay seams present near the top of sound rock, or along water bearing fracture intervals, were likely formed by weathering of the parent rock. Other thin clay intervals observed within rock cores were likely formed by the coring process itself, where relatively weak rock mechanically fractured during coring and formed clay coatings at the interfaces of the fractures. Such intervals are identified on the rock core logs as fracture zones with multiple sub-horizontal fractures alternating with thin clay seams or clay coatings. If mechanical formation was not definitively identified in the field, these features were logged as observed in the rock cores.

Slickensides were observed within siltstone and clay seams in the rock cores at numerous locations. These slickensides were typically apparent as oriented grains on bedding surfaces or within mechanical rock core breaks, and likely formed prior to lithification due to compaction and settlement of the parent sediments. The presence of such slickensides is not considered an indication of tectonic movement.

2.5.4.1.1.1.3 Correlation with the Site Geologic Setting

As discussed in Subsection 2.5.1, the HAR site is within the Durham basin of the Deep River basin. The Lithofacies Association (LA) II of the Durham basin is characterized by siltstone with interbedded sandstone, which are products of the depositional processes associated with fluvial, deltaic, and lacustrine deposits. LA II is characterized by trough cross-bedded sandstones, fining upward sequences, and fine-grained bioturbated siltstones. (Reference 2.5.4-204)

The rock core observations presented in Subsection 2.5.4.1.1.1.1 and 2.5.4.1.1.1.2 indicate that the HAR site bedrock is consistent with the LA II of the Durham basin. The results of petrographic examinations of rock core samples, as described in Subsection 2.5.4.2.3.1, are also consistent with bedrock of the LA II. LA II includes arkosic sandstones, conglomerates, siltstones, and claystones. Thirteen of the 16 petrographic examination samples were identified as arkose, subarkose, or lithic arkose.

2.5.4.1.1.2 Dip of Rock Strata

Rock strata are typically continuous across multiple boreholes at HAR 2 and HAR 3, as shown on the cross sections on Figures 2.5.4-204A, 2.5.4-204B, 2.5.4-205A, and 2.5.4-205B. The dip magnitude and direction of rock strata at HAR 2 and HAR 3 were evaluated using three methods:

- Identification of representative, continuous marker beds in boreholes at HAR 2 and HAR 3, and calculation of dip by triangulation of the contact elevations of these marker beds. Representative marker beds are shown on Figures 2.5.4-204A, 2.5.4-204B, 2.5.4-205A, and 2.5.4-205B, and summarized in Table 2.5.4-202.

- Identification of common shear wave velocity (V_s) patterns (i.e., fingerprints) among the suspension logging profiles at boreholes at HAR 2 and HAR 3 (see Subsection 2.5.4.4.2.1), and calculation of dip based on elevations of the common patterns among the boreholes.

- Evaluation of bedding features identified in oriented acoustic televiewer logs at boreholes at HAR 2 and HAR 3 (Subsection 2.5.4.4.2.2).

Table 2.5.4-202 summarizes the results of the rock strata dip calculations from these three methods. Results from the three methods are generally consistent with each other. The following observations are based on the rock strata dip calculations summarized in Table 2.5.4-202:

- Variations in dip under each HAR 2 and HAR 3 nuclear island footprint are interpreted to be due to lateral deposition changes. This is indicated by the relatively greater variation in dip of the coarser sandstone strata, such as HAR 2 Marker Bed A. Conversely, the finer-grained strata, such as HAR 2 Marker Bed B, which were deposited in more quiescent environments, exhibit less variation in dip across the nuclear island footprints.

- Variations in dip between the HAR 2 and HAR 3 footprints are interpreted to be primarily the result of folding or warping. The dip decreases in magnitude to the southeast, from HAR 3 to HAR 2, as indicated in Table 2.5.4-202. This is also observed in the contact elevation of HAR 2 Marker Bed B at Borehole BGA-3 (between HAR 2 and HAR 3), compared with boreholes under the HAR 2 nuclear island, as shown on Figures 2.5.4-204A and 2.5.4-205A. As shown, the apparent dip between BGA-3 and the HAR 2 nuclear island is steeper than at HAR 2 and shallower than at HAR 3, indicating a transition consistent with gradual folding or warping of the strata between the two sites.

- At HAR 2, the dip magnitude ranges from 6 to 9 degrees, directed generally to the east.

- At HAR 3, the dip ranges between approximately 19 and 23 degrees to the east-southeast. Local variations in dip appear to be related to small depositional variations. The dip calculated by matching the V_s profiles is marginally less than 20 degrees (19.9 degrees), as summarized in Subsection 2.5.4.4.2.1.2.

Subsection 2.5.1 provides additional descriptions of on-site and regional bedrock structures. As stated previously, the borehole data do not indicate faulting beneath the HAR 2 or HAR 3 nuclear island footprints. However, the presence or absence of small-scale faulting in the areas between or near HAR 2 and HAR 3 cannot be confirmed with the available data. Previous investigations have demonstrated that the Harris fault and other faults observed within the HAR site area are not capable tectonic structures, as described in Subsection 2.5.1.

2.5.4.1.2 Subsidence, Solution Activity, Uplift, or Collapse

As described in 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 HAR site, and hence there is no associated risk of subsidence or collapse. The rocks present at the HAR site are not susceptible to solution activity because carbonates or evaporates are not present in the subsurface there, as documented in Subsection 2.5.1.

2.5.4.1.3 Zones of Alteration, Irregular Weathering, or Structural Weakness

The following information describes zones of alteration, irregular weathering, and structural weakness encountered at the HAR site.

- Very weak rock and clay seams are encountered in isolated, thin intervals below the top of sound rock, as described in Subsection 2.5.4.1.1.1. These intervals are less than 1 m (about 3 ft.) thick, oriented along sub-horizontal bedding planes, with sound rock immediately above and below. Due to the limited thickness and sub-horizontal orientation of these zones under the HAR safety-related structures, these features do not compromise the suitability of the rock mass as foundation-bearing material for these structures, as discussed in Subsection 2.5.4.10.

- High-angle joints and fractures that cross bedding planes are present in rock at the HAR site, as described in Subsection 2.5.1.2.4 and Subsection 2.5.4.4.2.2. Where encountered, these features are typically tight, without significant evidence of weathering or infilling.

- Borehole observations at BPA-6, located approximately 17 m (55 ft.) north of the HAR 2 nuclear island, indicate that an approximate 2.7-m (9-ft.) total thickness of clay and silt intervals are present within rock between elevations 48.8 and 53.3 m (160 and 175 ft.).

- As presented in Subsection 2.5.4.2.3.1.1, UCS results from intact rock core samples from BPA-6 were lower than UCS results from samples from boreholes located under the HAR 2 or HAR 3 safety-related structures. For example, at Borehole BPA-7 (located near the north edge of the HAR 2 nuclear island closest to BPA-6), rock core was intact with no evidence of clay or silt intervals below elevation 67.1 m (220 ft.). UCS results from BPA-7 were also consistent with other results at HAR 2, which are higher than UCS results from Borehole BPA-6.

- The continuity of bedding planes between boreholes and the integrity of rock core under the HAR sites indicate that deformation features similar to the Harris fault are not present under the proposed safety-related facilities, as described in Subsection 2.5.1.2.5. However, because fault features are present in the HAR site vicinity, the presence of such a feature between or near the HAR sites cannot be precluded with existing information. It is possible that the borehole conditions observed at BPA-6 are indicative of such a feature north of HAR 2. Detailed exploration and mapping of the nuclear island excavations, and at appropriate locations near the excavations, will be performed after excavation and prior to construction as discussed in Section C.I.2.5.4 of Regulatory Guide 1.206, and in accordance with Regulatory Guide 1.208.

2.5.4.1.4 Unrelieved Stresses in Bedrock

There is no evidence of unrelieved stresses in bedrock, as described in Subsection 2.5.1.2.5.

2.5.4.1.5 Rocks or Soils that May Become Unstable

The potential hazard from rocks or soils that become unstable was determined to be low for the following reasons:

- As discussed in Subsection 2.5.4.1.1 and 2.5.4.2.4, rock strength does vary with depth and between boreholes, and occasional intervals of very weak rock and clay are present within otherwise sound rock. However, these very weak rock and clay intervals are less than 1 m (about 3 ft.) thick and oriented sub-horizontally along bedding planes. As described in Subsection 2.5.4.10.3, laboratory tested samples of clay intervals within rock have low water content (less than the plastic limit) which indicates low compressibility. The presence and orientation of the isolated very weak rock and clay intervals have been incorporated in the stability and settlement analyses presented in Subsection 2.5.4.10.1 and 2.5.4.10.3.

- Liquefaction of soil above the rock is not a concern for safety-related structures at HAR 2 or HAR 3, as discussed in Subsection 2.5.4.8. The nuclear islands and the Annex Buildings will be founded on sound rock or concrete fill over sound rock. Any imported granular fill or cohesive fill

adjacent to nuclear island sidewalls will be compacted as described in Subsection 2.5.4.5.3, such that its liquefaction potential will be eliminated.

- High angle joints and fractures in bedrock are typically tight without significant evidence of weathering or infilling, as described in Subsection 2.5.1.2.4 and Subsection 2.5.4.4.2.2. As shown on Figure 2.4.1-205, the planned site grade is essentially flat within 152 m (500 ft.) of the AP1000 structures (at approximate site grade 79.2 m [260 ft.]). Since surface slopes will not be present near the safety-related structures, high angle joints and fractures are not expected to adversely affect safety-related structure foundation stability.

2.5.4.1.6 History of Deposition and Erosion

As described in Subsection 2.5.1.2, the rocks near the surface at the HNP were deposited in a half graben formed in Triassic time. The half graben resulted from a large fault — the Jonesboro fault — in the underlying metamorphic rock basement. The current near-surface rocks were deposited in a fluvial or floodplain area that developed within the half graben.

The depression in land surface caused by the formation of the half graben in the underlying bedrock created a topographic low area into which area rivers and streams began to flow, carrying with them the sediment eroded from the adjacent highlands. Over time, these deposits filled the depression. The diabase dikes were intruded at or near the time when infilling of the half graben had occurred. Later, the area eroded to its present topography.

2.5.4.1.7 Estimates of Preconsolidation Pressures

Weathering processes that have formed the residual soils present over bedrock have also reduced the apparent preconsolidation pressure of these soils below the maximum past geologic loading pressure. Index test results for residual soil samples indicate moisture content significantly lower than the plastic limit, as presented in Subsection 2.5.4.2.3.2. This indicates that the soils are highly overconsolidated, as discussed in Subsection 2.5.4.10.3. This is consistent with consolidation test results for residual soil samples presented in Subsection 2.5.4.2.3.2, which also indicate high preconsolidation pressures.

2.5.4.2 Properties of Subsurface Materials

HAR COL 2.5-6

A program of field investigations and laboratory testing was performed to characterize the subsurface material properties underlying and adjacent to the HAR site, in accordance with Regulatory Guide 1.132 and Regulatory Guide 1.138. This subsection presents detailed discussions of the type, quantity, extent, purposes, and results of the investigation activities. Plot plans and profiles of information from site explorations are also provided. Properties of soil and rock used in design are summarized.

2.5.4.2.1 Description of Investigation Activities

Field investigation activities were performed at the HAR site from April through December 2006. 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 and laboratory tests that were conducted. Detailed discussions of the criteria used to develop the scope of the site investigation activities are presented throughout this subsection.

CH2M HILL managed the site investigation program. CH2M HILL geologists and engineers performed soil and rock logging in accordance with the Site Investigation Work Plan (SIWP) (Reference 2.5.4-205). The as-built borehole coordinates and ground elevations were surveyed by Smith and Smith Surveyors of Apex, North Carolina.

2.5.4.2.1.1 Soil Boring and Rock Coring

Eighty-three boreholes were advanced to obtain soil and rock samples for geologic characterization and for laboratory testing. The program included 50 boreholes at or near the planned AP1000 structures (BPA-series boreholes), 18 general characterization boreholes around the HAR site area (BGA-series), 8 boreholes at planned cooling tower locations (BCTA-series), and 7 boreholes at conveyance lines and at the intake structure (BCA-series). Figures 2.5.4-201 and 2.5.4-202 show the borehole locations.

The investigation activities at and near the HAR site were performed to develop a comprehensive characterization of subsurface conditions that may influence foundation performance of safety-related structures. Types, quantities, and depths of boreholes and in situ tests were selected to conform to guidance in NRC Regulatory Guide 1.132, and laboratory tests were performed to conform to guidance in NRC Regulatory Guide 1.138. Investigations were performed in accordance with the SIWP for the project (Reference 2.5.4-205). The investigations are consistent with the guidance in NRC Regulatory Guide 1.132 to address safety-related structure performance for a uniform site. The criteria that formed the basis for borehole locations and depths are presented in the following subsections. Changes to the planned activities made to address observations during the investigation, and their rationale, are also described.

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 spacing of principal boreholes for safety-related structures for favorable, uniform geologic conditions, as follows:

- At least one boring beneath every safety-related structure.

- For larger, heavier structures, such as the containment building, at least one boring per 900 square meters (m^2) (10,000 square feet [$ft.^2$]), approximately 30-m (100-ft.) spacing.

- In addition, a number of borings along the perimeter, at corners, and other selected locations.

- One boring per 30 m (100 ft.) for essentially linear structures.

The initial site investigation activities were selected to satisfy these criteria. The initial issue of the SIWP (Revision 0) specified 20 boreholes each at the HAR 2 and HAR 3 AP1000 structures, which provided coverage of the nuclear islands (Seismic Category 1 structures) as well as adjacent structures. These included five boreholes under the middle and sides of each nuclear island (BPA-3, BPA-5, BPA-7, BPA-8, and BPA-10 at HAR 2, and BPA-23, BPA-25, BPA-27, BPA-28, and BPA-30 at HAR 3), and eight boreholes outside the corners and sides of each nuclear island (BPA-1, BPA-2, BPA-4, BPA-6, BPA-9, BPA-11, BPA-12, and BPA-13 at HAR 2, and BPA-21, BPA-22, BPA-24, BPA-26, BPA-29, BPA-31, BPA-32, and BPA-33 at HAR 3). This also included four boreholes at the sides of the Seismic Category 2 portion of each Annex Building (BPA-4, BPA-8, BPA-9, and BPA-13 at HAR 2, and BPA-24, BPA-28, BPA-29, and BPA-33 at HAR 3).

Shortly after initiation of the field investigation activities, two additional boreholes were advanced at each HAR unit and incorporated into the SIWP (Revision 2). These boreholes provided coverage at the east wall of each nuclear island, and an additional borehole under each containment building (BPA-41 and BPA-45 at HAR 2, and BPA-43 and BPA-46 at HAR 3).

In November and December 2006, two additional boreholes were advanced at each nuclear island. These included boreholes near the northeast and southeast corners of each nuclear island (BPA-47 and BPA-48 at HAR 2, and BPA-49 and BPA-50 at HAR 3). Locations of BPA-47 and BPA-49 were field-modified to their final locations from the locations shown in Revision 3 of the SIWP to provide better distribution of the suspension logging holes at the HAR sites. These four new boreholes served two primary purposes:

- First, they provided deep boreholes for additional suspension logging profiling. This resulted in a roughly equilateral triangle of three boreholes at each unit (with BPA-5 and BPA-25) for evaluating the variation of V_s with depth and distance.

- Second, they allowed performance of rock point-load tests on rock core specimens at close spacing with depth, as a semi-quantitative field-measure of rock UCS.

Two of these boreholes (BPA-48 and BPA-50) were also co-located near the northeast corner of the HAR 2 and HAR 3 Annex Buildings, respectively.

In total, these boreholes provide coverage of the safety-related structures that satisfies and exceeds the criteria listed in NRC Regulatory Guide 1.132. The boreholes have average spacing less than 30 m (100 ft.) on center (more than one borehole per 900 m^2 [10,000 $ft.^2$] under safety-related structures).

The locations of the boreholes at the HAR sites are considered sufficient to characterize 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 borings for safety-related structures, as follows:

- Where soils are thick, the maximum required depth for engineering purposes, denoted 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.

- Borings should extend at least 10 m (33 ft.) below the lowest part of the foundation.

- If competent rock is encountered at lesser depths than those given, borings 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 borings and a minimum of one boring 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.

The depths of principal borings at safety-related structures were selected to satisfy these criteria. As specified in the initial submittal of the SIWP (Revision 0), principal boreholes were planned to satisfy the following criteria:

- Each of the principal boreholes were advanced below elevation 55.5 m (182 ft.), which is 11.6 m (38 ft.) below the nuclear island subgrade elevation of 67.1 m (220 ft.) (see Table 2.5.4-201).

- In addition, principal boreholes beneath safety-related structures were advanced to penetrate at least 6.1 m (20 ft.) below the top of sound rock.

- Initially, one borehole at each unit was advanced to a depth necessary to characterize soil/rock properties for ground motion parameters (including V_s) to be used for ground motion calculations and soil/rock-structure interaction studies (BPA-5 at HAR 2 and BPA-25 at HAR 3). These were advanced to at least elevation 18.3 m (60 ft.), which is the depth equivalent to the maximum width of the nuclear island of 48.8 m (160 ft.) below the subgrade elevation.

The top of sound bedrock was encountered above the nuclear island subgrade elevation at each of the principal boreholes under safety-related structures, as listed in Table 2.5.4-201. The depth requirements in NRC Regulatory Guide 1.132 for principal boreholes and uniform site conditions were satisfied by the criteria in the SIWP (Revision 0).

After initiation of the field investigation activities, several of the principal boreholes were extended to lower elevations to characterize isolated intervals of very weak rock and clay encountered below the top of sound rock. These changes were incorporated in SIWP Revisions 1, 2, and 3, and the resulting final borehole depths are listed in Table 2.5.4-201. As listed, several of the boreholes were advanced to elevation 36.6 m (120 ft.) or below, which is deeper than the initially planned bottom borehole elevation of 55.5 m (182 ft.) for those boreholes. All were terminated below the top of sound rock.

The three boreholes per unit at which suspension logging surveys were performed (BPA-5, BPA-47, and BPA-48 at HAR 2, and BPA-25, BPA-49, and BPA-50 at HAR 3) were advanced to at least elevation 18.3 m (60 ft.). As presented in Subsection 2.5.2, the depths of each of these boreholes characterized the transition from surficial rock conditions to deeper rock properties, which were characterized by other HAR site vicinity and regional data sources.

2.5.4.2.1.1.3 Drilling and Sampling Methods

Soil boreholes were advanced using mud rotary drilling. The positive hydrostatic head produced by this method aids maintenance of borehole sidewall integrity while drilling through loose or soft soils. This method also allows collection of reliable standard penetration test (SPT) blow counts in these materials. Mud rotary is the recommended drilling method in NRC Regulatory Guide 1.132.

Rock coring was performed using NQ- or HQ-size double-tube rotary wireline core barrels, in accordance with American Society for Testing and Materials (ASTM) D2113 (Reference 2.5.4-206). The stationary inner core barrel used in double-tube coring helps to reduce rock core damage compared with single-tube coring, and is a recommended coring method in NRC Regulatory Guide 1.132. NQ was the predominant size (nominal sample diameter of 47.6 mm [1.875 in.]) used for most of the boreholes. Boreholes in which suspension logging soundings were performed, as described in Subsection 2.5.4.4.1.1 (BPA-5, BPA-25, BPA-39, BPA-47, BPA-48, BPA-49, and BPA-50), were advanced using HQ-size (nominal sample diameter of 63.5 mm [2.5 in.]) wireline coring tools. Drilling activities were performed primarily by S&ME, Inc. of Knoxville, Tennessee, with additional work

performed by Parratt-Wolff of Hillsborough, North Carolina, and Trigon Engineering of Raleigh, North Carolina.

Disturbed soil samples were collected by SPT sampling at regular depth intervals in accordance with NRC Regulatory Guide 1.132. “Undisturbed” soil samples were collected by Pitcher tube sampling methods at targeted depth intervals, typically in a separate borehole adjacent to the borehole in which SPT samples were collected. Pitcher tube sampling is described in Appendix C of NRC Regulatory Guide 1.132. In this method, a thin-walled steel tube (which collects the sample) is spring-connected to a rotating outer cutting bit. The spring allows the tube to recess deeper within the cutting bit in stiffer soils, which better protects the sample tube from damage. A few “undisturbed” samples were collected using Shelby tubes.

Rock core samples were collected directly from the NQ- or HQ-size rock core, and managed as either “routine-care” or “special-care” cores, as described in Subsection 2.5.4.2.1.5.

Upon completion of drilling and coring, boreholes greater than 100 feet total depth were surveyed for verticality deviation. Boreholes in which geophysical surveys were performed were surveyed for verticality deviation by GeoVision, and other boreholes were surveyed by CH2M HILL. All surveyed boreholes were found to be within approximately one degree of vertical. Boreholes were abandoned with concrete grout in accordance with North Carolina state requirements.

The soil and rock 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 HAR, and to provide reliable data to characterize 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 soil and rock types, soil consistency, and rock soundness.

- 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 2.5.4-205). Appendix 2BB presents soil boring and rock core logs. As-built survey coordinates and elevations are also included on the borehole logs (Reference 2.5.4-207).

- SPTs were performed at regular intervals in soil in accordance with ASTM D1586 (Reference 2.5.4-208). The SPT blow count criteria used to define soil boring refusal was at least 50 blows per 7.6 cm (3 in.).

- Field indicators of rock soundness and strength were established based on the rock quality designation (RQD) (ASTM D6032), the R-value indicator of

strength (International Society of Rock Mechanics [ISRM], 1981), and field point-load tests (ASTM D5731) (Reference 2.5.4-209, Reference 2.5.4-210, Reference 2.5.4-211). Rock PMTs were also performed in selected boreholes as described in Subsection 2.5.4.2.1.3.

2.5.4.2.1.1.5 Basis for Selection of Field Rock Hardness and Strength Tests

Rock consistency at the HAR site was initially characterized using a field hardness test as described in the SIWP (Reference 2.5.4-205). This common test of rock hardness is based on response of the rock core to scratching with a steel blade. During the course of field activities, it was observed that most of the rock core collected on-site was soft based on the field hardness test (i.e., could be scratched with a steel blade), but that the rock typically exhibited significant strength.

As a result of these observations, a semi-quantitative "R-scale" field measure of rock core strength based on ISRM was then implemented (Reference 2.5.4-210). The R-scale strength value is determined based on the observed response of the rock to impact with a geologic hammer. Starting in mid-July 2006, R-scale strength values were determined for previously collected rock cores stored as routine-care samples within plastic liners at the sample storage area. Samples were often stored for several weeks prior to determination of R-scale values. R-scale values were determined upon collection for rock cores collected in August, November, and December 2006. The resulting R-scale values provide a semi-quantitative field indication of rock strength and a basis to identify sound rock, but are not considered appropriate for use as specific UCS values for design.

In addition to the hardness and R-scale values, field point-load tests (PLTs) were performed in accordance with ASTM D5731 at regular depth intervals on rock core collected from Boreholes BPA-47, BPA-48, BPA-49, and BPA-50 (in November and December 2006). 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 UCS (Reference 2.5.4-211). The field PLT results supported collection of rock UCS samples from Boreholes BPA-47 through BPA-50, to demonstrate that UCS samples were collected from rock spanning the range of PLT rock strengths.

These field hardness and strength tests, combined with the 80 laboratory UCS tests on rock core specimens, provide data to characterize rock strength at the HAR sites.

2.5.4.2.1.2 Geophysical Surveys

A program of borehole and surface geophysical surveys was implemented at HAR 2 and HAR 3. In-hole survey methods consisted of suspension logging, downhole, and acoustic televiewer surveys. Surface geophysical surveys consisted of seismic refraction, spectral analysis of surface waves (SASW), and

magnetometer surveys. Multi-channel analysis of surface waves (MASW) surveys were also attempted.

Detailed descriptions of the geophysical survey activities are presented in Subsection 2.5.4.4.1.

2.5.4.2.1.3 Rock Pressuremeter Testing

In-Situ Soil Testing, LC, of Lancaster, Virginia, performed rock pressuremeter tests at two boreholes each at HAR 2 and HAR 3 (BPA-3, BPA-23, BPA-41, and BPA-43). These tests were performed at multiple depths per borehole, to provide information on the rock mass modulus of in situ rock (Reference 2.5.4-212).

2.5.4.2.1.4 Groundwater Monitoring Wells and Hydraulic Conductivity Tests

Twenty monitoring wells were installed at the HAR site to monitor seasonal fluctuations in groundwater elevations and to evaluate hydraulic conductivity of soil and rock. Monitoring well locations were installed in accordance with the SIWP (Reference 2.5.4-205). Figures 2.4.12-203 and 2.4.12-204 show the locations of these wells. Table 2.4.12-205 provides a summary of monitoring well construction details. Tables 2.4.12-206 and 2.4.12-207 summarize groundwater elevations and gradients in on-site groundwater monitoring wells, and Table 2.4.12-208 provides a summary of in-well slug test results.

Hydraulic pressure (i.e., packer) tests were performed in two boreholes, one each near the centers of HAR 2 and HAR 3 (BPA-45 and BPA-46, respectively). These tests were performed over 3-m (10-foot) depth intervals within rock at each borehole. Monitoring wells MWA-3D and MWA-8D were installed in these boreholes after completion of the packer testing. Descriptions of the methods and results, including comparison with hydraulic conductivity results from in-well slug tests at these boreholes, are presented in Subsection 2.5.4.2.2.6.

2.5.4.2.1.5 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 detail in the SIWP (Reference 2.5.4-205) and summarized as follows:

- Soil samples recovered by SPT methods were stored in jars. These samples were retained for visual-manual field classification, but were not submitted for laboratory testing.

- Pitcher tube and Shelby tube samples were sealed at both ends with wax and plastic end caps. These samples were managed for protection from shock and extremes in temperature or humidity. They were maintained in a vertical orientation after collection. Pitcher tube and Shelby tube samples were stored

in a separate temperature- and humidity-monitored storage area prior to shipment to the testing laboratory.

- 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 2.5.4-213).
  - 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 and plastic sleeve liners, and long-term storage in access-controlled storage areas. These samples will be available for inspection by designers, contractors, or regulatory staff during design, construction, and operation.
  - 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.

Foam cushioning material was used to protect soil and rock core specimens from shock when they were transported to the testing laboratory. Upon receipt at the testing laboratory, 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 the HAR site for long-term storage with the routine-care samples.

2.5.4.2.1.6 Laboratory Testing of Soil and Rock

Laboratory tests were performed on more than 100 rock and soil samples recovered during the drilling and sampling program. S&ME, Inc., performed the geotechnical laboratory tests at their laboratory in Knoxville, Tennessee, except as stated below for petrographic examinations of rock core samples. The S&ME laboratory performed work under its own quality program, which CH2M HILL

audited and approved for work on this project. Laboratory tests were performed in accordance with testing methods recommended in NRC Regulatory Guide 1.138.

2.5.4.2.1.6.1 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 80 intact rock core samples in accordance with ASTM D7012 (Reference 2.5.4-214). Note that ASTM D7012 is a new standard that includes ASTM D2664, D5407, D2938, and D3148 (standards referenced in NRC Regulatory Guide 1.138) as methods. Bulk density was reported with the UCS results for all but six of these samples.

- Forty-seven of these UCS tests were performed with axial and radial strain measurements, which allowed characterization of the elastic modulus and Poisson's ratio (Methods C and D of ASTM D7012) (Reference 2.5.4-214).

- Sixteen rock samples were submitted for petrographic examination to provide a detailed assessment of the lithology and mineralogy of the rocks at the HAR site in accordance with Rock Testing Handbook (RTH) method 102-93 (Reference 2.5.4-215). GeoSystems of Kingwood, Texas performed these examinations as an approved subcontractor to S&ME, Inc.

- Soil index tests were performed on 26 soil samples collected by Pitcher tube and Shelby tube sampling above bedrock, as well as 13 samples collected from soil-like seams within rock. Index tests included moisture content (ASTM D2216), Atterberg Limits (ASTM D4318), gradation (ASTM D422), and specific gravity (ASTM D854) (Reference 2.5.4-216, Reference 2.5.4-217, Reference 2.5.4-218, and Reference 2.5.4-219).

- Soil consolidation tests were performed on six samples, in accordance with ASTM D2435 (Reference 2.5.4-220).

- Soil strength tests were performed using three different methods:

    - Unconfined compression (UC) tests were performed on 13 samples in accordance with ASTM D2166 (Reference 2.5.4-221).

    - Unconsolidated-undrained (UU) triaxial compression tests were performed on nine samples in accordance with ASTM D2850 (Reference 2.5.4-222).

    - Consolidated-undrained (CU) triaxial compression tests were performed on three samples, in accordance with ASTM D4767 (Reference 2.5.4-223).

Of the laboratory tests, the rock UCS tests are considered most critical to the safe performance of the Seismic Category I structures. The soil strength tests provide design data for foundations of nonsafety-related structures and construction slope stability, and have been used as an indicator of the shear strength of clay seams within rock. No special sample preparation or test procedure modifications were required or applied for the rock UCS tests.

Relative density tests were not conducted because these tests are applicable only for granular soils, whereas soils are predominantly fine-grained at the HAR sites.

Results of the laboratory tests are presented in Subsection 2.5.4.2.3.

2.5.4.2.1.6.2 Criteria for Selection of Rock Core Samples for Laboratory Testing

Rock core specimens were selected as "special-care" samples and laboratory test samples based on the following criteria:

- Samples were collected at targeted elevation ranges, including near and slightly below the nuclear island subgrade elevation (elevation 67.1 m [220 ft.]).
- Samples were collected to characterize different rock types within or between boreholes, as determined by the CH2M HILL field representatives.
- Samples were collected to span the range of apparent rock core soundness, based on observations of hardness or R-scale value as determined by the CH2M HILL field representatives.
- Samples were targeted for collection at locations of relatively weak rock as indicated by the PLT point load index results (at Boreholes BPA-47 through BPA-50 only).

In total, UCS tests were performed on 80 special-care rock core samples, 70 of which were collected at or near the HAR structures.

2.5.4.2.1.6.3 Criteria for Selection of Soil Samples for Laboratory Testing

The top of rock is shallow at the HAR structure locations. The existing soil profile will be removed for construction of the nuclear islands (seismic Category I structures) and the adjacent Annex Buildings and the first bay of the Turbine Buildings (seismic Category II structures), as discussed in Subsection 2.5.4.5. Therefore, the soil profile has minimal effect on the performance of these structures. Laboratory samples of soils were collected primarily to provide data on soils that may be left in-place under other structures and to support construction slope stability of the upper slopes of the nuclear island excavations.

Isolated soil seams within bedrock were tested for index properties to allow comparison with literature correlations for preconsolidation pressure and compression indices, as described in Subsection 2.5.4.10.3.2. Representative clay seams were selected for testing that were large enough to collect as "special-care" core specimens, which also appeared relatively undisturbed by the coring process. None of the soil-like seams collected were considered sufficiently undisturbed for use as reliable consolidation or shear strength test specimens. Therefore, correlation with index tests and with surficial soil shear strength test results are considered the best available method to characterize the compressibility and shear strength of these soil seams encountered below top of sound rock.

2.5.4.2.2 Soil and Rock Engineering Properties from Field Investigations

During 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. Some of the test results provided more direct information about soil or rock properties than others. These field observations and tests included the following:

2.5.4.2.2.1 Standard Penetration Test Blow Counts (N)

This indicator of soil consistency was recorded at 0.76- to 1.52-m (2.5- to 5-ft.) depth intervals in each borehole from ground surface to the depth of soil boring refusal. Tests were performed in accordance with ASTM D1586 using autohammers for which energy transfer efficiency test data were available (Reference 2.5.4-208). SPT N values are shown on the soil boring logs in Appendix 2BB.

2.5.4.2.2.2 Rock Quality Designation

This indicator of rock soundness was recorded for each rock core run, in accordance with ASTM D6032 (Reference 2.5.4-209). RQD values are shown on the rock coring logs in Appendix 2BB and on the cross sections on Figures 2.5.4-204A, 2.5.4-204B, 2.5.4-205A, and 2.5.4-205B. Average RQD values for each borehole are summarized in Table 2.5.4-203.

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 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 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 (Reference 2.5.4-210). For example, R2 rock corresponds to a range in UCS from 5 to 24 MPa (725 to 3500 psi). Sound rock as defined in this FSAR generally corresponds to R2 or better rock (see Subsection 2.5.4.2.2.7). 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 considered at most 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 at regular depth intervals in four boreholes (BPA-47, BPA-48, BPA-49, and BPA-50). Point-load tests were performed in accordance with ASTM 5731 (Reference 2.5.4-211). By comparing rock UCS test results with adjacent point-load test results, a site-specific correlation factor of 21.3 has been established, which is a typical value. Figures 2.5.4-207A, 2.5.4-207B, 2.5.4-207C, and 2.5.4-207D show the estimated UCS based on PLTs at various depths at Boreholes BPA-47 through BPA-50.

The laboratory UCS test results for samples from Boreholes BPA-47 through BPA-50 are also shown on these figures. As shown, the test results generally correlate with the corresponding PLT tests. The most notable difference is Sample BPA-47, SC-5, shown at elevation 56.1 m (184 ft.) on Figure 2.5.4-207A, where the laboratory UCS result of 23.7 MPa (3444 psi) is significantly less than the PLT estimate of UCS greater than 55.2 MPa (8000 psi). Other less notable differences exist, with laboratory UCS results both above and below the PLT estimates of UCS, which are indicative of the variable nature of sound rock UCS with depth.

2.5.4.2.2.5 Rock Pressuremeter Test (PMT) Modulus (E_{pmt})

This quantitative indicator of rock compressibility was measured at numerous depths in four boreholes (BPA-3, BPA-23, BPA-41, and BPA-43). In-Situ Soil Testing, LC, performed the tests in accordance with the procedure in the SIWP (Reference 2.5.4-205). Rock pressuremeter modulus results are presented in Table 2.5.4-204 (Reference 2.5.4-212).

2.5.4.2.2.6 Hydraulic Conductivity Tests

Twenty monitoring wells were installed at the HAR site to monitor seasonal fluctuations in groundwater elevations and to evaluate hydraulic conductivity of soil and rock. Monitoring well locations were installed in accordance with the SIWP (Reference 2.5.4-205). Figures 2.4.12-203 and 2.4.12-204 show the locations of these wells. Table 2.4.12-205 provides a summary of monitoring well construction details. Tables 2.4.12-206 and 2.4.12-207 summarize groundwater elevations and gradients in on-site groundwater monitoring wells, and Table 2.4.12-208 provides a summary of in-well slug test results.

Results of hydraulic pressure (i.e., packer) tests performed in Boreholes BPA-45 and BPA-46 prior to monitoring well installation are presented in this subsection.

2.5.4.2.2.6.1 Hydraulic Pressure (Packer) Tests

Hydraulic pressure (i.e., packer) tests were performed over five adjacent 3-m (10-ft.) depth intervals below top of rock at Boreholes BPA-45 and BPA-46 near the center of HAR 2 and HAR 3, respectively. The tests were performed ascending from the bottom of the borehole to the top of rock over 3-m (10-ft.) intervals. Tested intervals ranged from 4.7 to 20 m (15.5 to 65.5 ft.) bgs at BPA-45, and from 5.5 to 20.7 m (18 to 68 ft.) bgs at BPA-46. The intervals were tested for water flow at three different gauge pressures. Monitoring wells MWA-3D and MWA-8D were subsequently installed within the rock core holes for BPA-45 and BPA-46, respectively, after completion of packer testing.

At four of the five packer test intervals at each borehole, the permeability of the rock mass was too low to record water flow. Measureable flow was recorded in one interval at each borehole; BPA-45 at 10.8 to 13.9 m (35.5 to 45.5 ft.) bgs, and BPA-46 at 11.6 to 14.6 m (38 to 48 ft.) bgs. The resulting flow curves were slightly non-linear, which indicated that joint infilling, joint erosion, or turbulent flow may have affected the test results. The calculation of hydraulic conductivity was performed on the linear portions of the results from each borehole, and are considered an estimate of the hydraulic conductivity for comparison with results of the in-well slug tests.

The following hydraulic conductivities were calculated in these intervals:

- At BPA-45, 10.8 to 13.9 m (35.5 to 45.5 ft.) bgs, the hydraulic conductivity was calculated as 0.015 meters per day (m/day) (0.05 feet per day [ft/day]).
- At BPA-46, 11.6 to 14.6 m (38 to 48 ft.) bgs, the hydraulic conductivity was calculated as 0.15 m/day (0.5 ft/day).

Fracture intervals were observed in rock core from both of these test intervals. The other tested intervals at BPA-45 and BPA-46, which did not produce measurable flow, also had some fracture intervals, but based on the test results these fractures are not sufficiently interconnected to sustain water flow. Due to

the very low porosity of intact rock mass, as indicated by the petrographic examination results presented in Subsection 2.5.4.2.3.1, the only significant groundwater flow is through secondary porosity from fractures within the rock mass.

2.5.4.2.2.6.2 Comparison of Hydraulic Conductivity Test Results

The hydraulic conductivity calculated from packer test results was consistent with results of in-well slug tests in monitoring wells MWA-3D and MWA-8D, as presented in Table 2.4.12-208. For the in-well slug test at MW-3D (i.e., at BPA-45), the hydraulic conductivity was 0.003 m/day (0.01 ft/day). The hydraulic conductivity at MW-8D (i.e., at BPA-46) was 0.06 m/day (0.2 ft/day).

Considering that groundwater flow was negligible at four of the five packer test intervals each at BPA-45 and BPA-46, the average hydraulic conductivity over the 15-m- (50-ft.-) total depth interval from the packer test results at each boreholes is very similar to the result of the in-well slug test at MWA-3D and MWA-8D. The packer test results therefore provide confirmation of the in-well slug test results at these wells. The packer tests also provide additional information on the depth intervals that produce groundwater flow at these locations.

2.5.4.2.2.7 Criteria for Soil Depth and Top of Sound Rock

The “soil depth” listed for each borehole in Table 2.5.4-201 is the depth where the SPT blow count was greater than 50 blows over 7.6 cm (3 in.), which is the criterion for soil boring refusal for this FSAR. The “depth to sound rock” listed in Table 2.5.4-201 is the shallowest occurrence of at least 1.5 m (5 ft.) of contiguous slightly weathered to fresh rock. Slightly weathered to fresh rock generally corresponds to an R-scale strength value of R2 or greater. However, as discussed in Subsection 2.5.4.2.2.3, the R-scale strength values are considered only semi-quantitative, and the data are not the primary strength data used for analysis. Sound rock is generally characterized by V_s greater than 914 m/sec (3000 fps), as described in Subsection 2.5.4.4. The rock UCS laboratory test results presented in Subsection 2.5.4.2.3.1 and the point-load test results on Figures 2.5.4-207A, 2.5.4-207B, 2.5.4-207C, and 2.5.4-207D provide more quantitative strength data for design.

2.5.4.2.3 Soil and Rock Engineering Properties from Laboratory Tests

Laboratory tests for soil and rock index and engineering properties were performed as described in this subsection.

2.5.4.2.3.1 Rock Laboratory Test Results

Results of rock laboratory tests are presented in this subsection. These include UCS and index tests performed by S&ME, Inc. and petrographic examinations performed by GeoSystems.

2.5.4.2.3.1.1 Rock UCS and Index Test Results

Table 2.5.4-205 shows the results of individual laboratory tests on rock samples. Table 2.5.4-206 presents statistical summaries of the tested rock properties for samples grouped by HAR unit and elevation ranges relative to the nuclear island subgrade elevation of 67.1 m (220 ft.). These data show that the rock underlying the nuclear islands is generally of high quality, characterized by an average unconfined compressive strength in excess of 41.4 MPa (6000 psi). (Reference 2.5.4-224)

2.5.4.2.3.1.2 Petrographic Examination Results

Petrographic examinations of 16 rock core specimens were performed to characterize the gradation, mineralogy, and lithologic description of representative rock core sections. Results of the investigations are summarized in Table 2.5.4-207 (Reference 2.5.4-203). Four of the rock samples had sufficient clay-size particles to be classified as mudstone using the Picard classification system used in the petrographic examinations, which is a fine-grained rock with intermediate amounts of clay and silt-size particles, as shown on Figure 2.5.4-208. None of the tested samples had sufficient clay-size particles to be classified as claystone.

Total porosity and cementation of the rock samples were low. Total porosity of the samples, including intergranular, secondary intergranular, and microporosity, was extremely low. Porosity of the sandstones ranged from zero to 4.8 percent, and porosity of the siltstones and mudstone samples was negligible. The sandstones had the highest amount of cementation, ranging from 3.6 to 7.6 percent. The siltstones contained very little cement, ranging from 0.4 to 3.2 percent. The mudstone samples had the lowest amount of cement, ranging from 0.8 to 2.8 percent. These results indicate that compaction was the main cause of lithification of the rock, especially for the fine-grained rock.
The origin of the parent rock for the sedimentary rock samples tested was identified as partially metamorophic. Some of the grains within the individual core samples were metamorphic, gneissic, schistose, phyllitic, and quartzite. Metamorphic sandstone grains were schistose and metamorphic. The metamorphic siltstone grains were gneissic, schistose, and quartzite, and metamorphic mudstone grains were gneissic, schistose, and phyllitic. Thirteen of the 16 petrographic examination samples were identified as arkose, subarkose, or lithic arkose.

As discussed in Subsection 2.5.4.1.1.1, each of the fine-grained samples was field classified as siltstone, using rock description criteria consistent with previous work for the HNP (Reference 2.5.4-201, Reference 2.5.4-202). Therefore, rock described as siltstone in this FSAR includes rock that classifies as either siltstone or mudstone by the Picard classification system.

2.5.4.2.3.2 Soil Laboratory Test Results

Laboratory tests were performed to characterize index, strength, and consolidation properties of residual soil samples (Reference 2.5.4-224).

Index tests (gradation, bulk density, moisture content, and Atterberg limits) were performed on samples collected from the residual soils above bedrock, and on samples collected from within isolated soil-like intervals below the top of sound bedrock. Table 2.5.4-208 lists the soil index properties for each sample, and also lists the summary statistics for each index property, subgrouped as soil samples collected above and below the top of sound bedrock.

Soil strength tests (UC, UU, and CU triaxial tests) were performed on thin-walled Pitcher tube soil samples collected from the residual soil zone above bedrock, and on a few samples collected from soil-like intervals within bedrock (collected as “special-care” rock core samples). Table 2.5.4-209 lists soil strength test results.

Consolidation tests were performed on six samples collected from the residual soil zone above bedrock. Table 2.5.4-210 lists the consolidation test results. Soil-like samples collected from rock cores were observed to be too disturbed by the rock coring process to perform representative consolidation tests.

These soil engineering property data have been used to evaluate nuclear island construction slope stability and have been used as an indicator of the shear strength of clay seams within rock.

2.5.4.2.4 Rock and Soil Properties for Use in Engineering Analyses

This subsection provides a summary of rock and soil properties from the investigations used in the engineering analyses.

2.5.4.2.4.1 Static Rock Engineering Properties

Table 2.5.4-205 presents static engineering properties of rock from laboratory testing. Table 2.5.4-206 presents summary statistics for samples grouped by HAR 2 and HAR 3 and by elevation range. The mean, standard deviation, minimum, and maximum values were calculated for each property for each sample group. These data are used directly for engineering analyses including bearing capacity and settlement as described in Subsection 2.5.4.10. Engineering properties of rock from laboratory testing include the following:

- UCS.
- Elastic modulus (secant and tangent).
- Poisson’s ratio (secant and tangent).

- Moisture content and bulk density.

Rock mass strength properties are estimated using the Hoek-Brown criteria (Reference 2.5.4-225). This method is dependent both on the UCS of intact rock and on the conditions of discontinuities in the rock. The intact rock UCS data in Table 2.5.4-206 were used to calculate Hoek-Brown strength parameters. Presentation of the calculation of Hoek-Brown strength parameters for rock mass under HAR 2 and HAR 3 are discussed in Subsection 2.5.4.10.1.2.

2.5.4.2.4.2 Dynamic Rock Engineering Properties

Table 2.5.4-211 presents average values of V_s, compressional wave velocity (V_p), and Poisson's ratio from the suspension logging surveys at HAR 2 and HAR 3. Figures 2.5.4-209A, 2.5.4-209B, 2.5.4-209C, 2.5.4-209D, 2.5.4-209E, 2.5.4-209F, and 2.5.4-209G present profiles showing V_s at individual boreholes. These data characterize rock in boreholes advanced to below elevation 18.3 m (60 ft.). These suspension logging data are used directly in the following engineering analyses:

- Elastic settlement (Subsection 2.5.4.10.3.1).

- Site response analyses (Subsections 2.5.4.7 and 2.5.2.5, specifically Figures 2.5.2-251, 2.5.2-252, 2.5.2-253, 2.5.2-254, 2.5.2-255, 2.5.2-256, 2.5.2-257, 2.5.2-258, 2.5.2-259, 2.5.2-260, and 2.5.2-261).

2.5.4.2.4.3 Elastic Modulus

The rock elastic modulus is calculated directly from the suspension logging V_s data, as described in Subsection 2.5.4.10.3.1. These calculations include a reduction factor of 0.5 to account for shear strain amplitude effects. The corresponding depth-weighted average Young's modulus of rock values below elevation 67.1 m (220 ft.) at HAR 2 and HAR 3 are 1.31×10^4 MPa (2.73×10^5 kilopounds (kips) per square foot [ksf]) and 1.36×10^4 MPa (2.84×10^5 ksf), respectively. If a lower reduction factor than 0.5 were used, the depth-weighted Young's modulus values would be correspondingly higher.

UCS tests with strain measurements provide additional data on rock elastic modulus. Statistics on the secant modulus at 50 percent of the failure strain from UCS tests are included in Table 2.5.4-206. The tangent modulus was also calculated at 50 percent of the failure strain for the same samples, as shown in Table 2.5.4-206. The results indicate the following:

- The average secant modulus values for rock below elevation 67.1 m (220 ft.) at HAR 2 and HAR 3 from the UCS tests are 10,600 MPa (1.54×10^6 psi) and 9860 MPa (1.43×10^6 psi), respectively. These values are 20 to 30 percent lower than the weighted average Young's modulus values calculated from V_s data. The differences are attributed to sample recompression effects at low applied stress levels during the UCS testing, caused by closure of microfractures formed by release of overburden stresses.

- The average tangent modulus values for rock samples collected below elevation 67.1 m (220 ft.) at HAR 2 and HAR 3 from the UCS tests are 1.31 x 10^4 MPa (1.90 x 10^6 psi) and 1.26 x 10^4 MPa (1.83 x 10^6 psi), respectively. These results are similar to the weighted average Young's modulus values calculated from V_s data.

The rock PMT modulus results listed in Table 2.5.4-204 are lower than the elastic modulus calculated from the V_s data and from the UCS modulus data, as follows (Reference 2.5.4-212):

- At HAR 2, the average PMT modulus from tests below elevation 67.1 m (220 ft.) is 6.2 x 10^3 MPa (1.3 x 10^5 ksf), which is approximately half of the value calculated from suspension logging V_s data.

- At HAR 3, the two shallowest PMT tests at each borehole appear to have been compromised by borehole widening during the coring process. These tests were performed at BPA-23 at 7.2 and 14 m (23.6 and 45.9 ft.) bgs, and at BPA-43 at 7.5 and 10.7 m (24.6 and 35.1 ft.) bgs. Inspection of the pressure-volume curves from these tests indicates atypically high volumes were required to contact the test membrane with the corehole wall, likely due to softening or widening of the corehole wall during rock coring. These tests were therefore terminated before an adequate pressure-volume relationship was established, with only two data points available to calculate the PMT modulus for these samples. The resulting modulus values are considered unrealistically low for these samples. For this reason, these four PMT tests are not considered representative of rock compressibility, and are not considered in subsequent engineering calculations.

- At HAR 3, the PMT modulus from two tests at Borehole BPA-43 are based on limited data. These tests were performed at BPA-43 at 14.3 and 17.1 m (46.9 and 56.1 ft.) bgs. Corresponding PMT modulus values are 390.1 and 556.4 MPa (8.1 x 10^3 and 1.2 x 10^4 ksf) respectively. Inspection of the pressure-volume curves for these tests indicates that a linear relationship was defined, but the limited number of available data points (four for each test) reduces the confidence in the associated modulus. For this reason, these two PMT tests are considered in subsequent engineering analyses, but the resulting PMT modulus values are considered conservative and bounding estimates of shallow HAR 3 rock compressibility.

- At HAR 3, the average PMT modulus from tests deeper than those presented above is 4.3 x 10^3 MPa (8.9 x 10^4 ksf), which is approximately one-third of the modulus value calculated from suspension logging V_s data.

The PMT modulus values have been considered as input for upper-bound estimates of rock elastic settlement as described in Subsections 2.5.4.10.3.1. The suspension logging V_s results, degraded for strain effects, are the primary data used to calculate the best estimates of elastic settlement.

2.5.4.2.4.4 Engineering Properties of Soil

The engineering properties of soil samples recovered from above sound rock are summarized in the following tables:

- Soil index test results in Table 2.5.4-208.
- Soil strength test results in Table 2.5.4-209.
- Soil consolidation properties in Table 2.5.4-210.

Soils present above sound rock will not be left in place under seismic Category I or II structures. Therefore, surficial HAR site soils will not affect safety-related structure foundation performance. These soil engineering property data have been used to evaluate nuclear island construction slope stability, and have been used as an indicator of the shear strength of clay seams within rock.

Index properties of samples from soil-like intervals within sound rock are included in Table 2.5.4-208. These results have been used to characterize the compressibility of clay seams within rock, as described in Subsection 2.5.4.10.3. As discussed in Subsection 2.5.4.2.1.6.3, samples of these soil-like intervals were considered unsuitable for laboratory strength or consolidation testing due to sample disturbance during coring.

2.5.4.2.4.5 Backfill Engineering Properties

Engineering properties of backfill to be placed adjacent to the seismic Category I structures (nuclear islands) and under seismic Category II structures are discussed in Subsection 2.5.4.5.3.

2.5.4.3 Foundation Interfaces

HAR COL 2.5-5
HAR COL 2.5-6

"Plant North" for HAR 2 and HAR 3 is rotated 65 degrees clockwise from State Plane North (i.e., N65E), as shown on Figure 2.5.4-202 (and other figures). Note that HAR 2 and HAR 3 "Plant North" is different than "Plant North" for the HNP, which is oriented N25W. Where "Plant" is listed before a direction, the direction listed is relative to Plant North directed N65E. Where "Plant" is not indicated before a direction, the direction listed is relative to State Plane North.

The current surface conditions at HAR 2 and HAR 3 are as follows:

- HAR 2 is located in an area previously graded to support construction of the HNP, with existing ground elevation ranging from approximately 79.2 to 81.1 m (260 to 266 ft.), as shown on Figure 2.5.4-203A. The ground surface at HAR 2 is covered with grass and gravel access roads, as shown on Figure 2.5.4-201.

- HAR 3 is located in a previously undeveloped area north of HAR 2, with rolling ground surface elevation ranging from approximately 77.7 m (255 ft.) to more than 82.3 m (270 ft.), as shown on Figure 2.5.4-203B. The ground surface at HAR 3 is covered with trees, brush, and grass, as shown on Figure 2.5.4-201.

Approximate ground surface topography prior to construction of the HNP is shown on Figure 2.5.4-210. This figure indicates that the existing ground surface near HAR 2 was modified during construction of the HNP. The existing ground surface at HAR 3 is similar to conditions present prior to construction of HNP.

The nominal site grade for HAR 2 and HAR 3 is elevation 79.6 m (261 ft.), which is equivalent to the AP1000 floor elevation. The surrounding grade will be lower to accommodate site grading, drainage, and local site flooding requirements. The HAR 2 and HAR 3 nuclear islands will be founded at basemat elevation 67.5 m (221.5 ft.), with a mudmat and waterproofing geomembrane extending below the basemat to elevation 67.4 m (221.0 ft.). It is anticipated that additional excavation below this will be performed to prepare the subgrade for the nuclear island foundations, such that the typical subgrade elevation will be 67.1 m (220.0 ft.). The planned excavation extents at HAR 2 and HAR 3 structures are shown in relation to the geologic profiles on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B, as discussed in Subsection 2.5.4.5.

Locations of boreholes and geophysical surveys are shown in relation to safety-related structures on Figures 2.5.4-203A and 2.5.4-203B, respectively. Borehole logs are included in Appendix 2BB. Detailed exploration and mapping of the nuclear island excavations, and at appropriate locations near the excavations, will be performed after excavation and prior to construction as discussed in Section C.I.2.5.4 of Regulatory Guide 1.206, and in accordance with Regulatory Guide 1.208.

2.5.4.4 Geophysical Surveys

HAR COL 2.5-6

Geophysical surveys were performed to characterize the properties of soil and rock at the HAR site. The geophysical survey methods, and results of the surveys, are presented in this subsection.

2.5.4.4.1 Description of Geophysical Surveys

Two general types of geophysical survey methods were used to characterize the properties of soil and rock at the HAR site: in-hole surveys and surface geophysical surveys. Figures 2.5.4-203A, 2.5.4-203B, and 2.5.4-213 show the geophysical survey locations at HAR 2 and HAR 3. Following are summaries of the scope, objectives, and methods for each of these survey methods.

2.5.4.4.1.1 Suspension Logging Surveys

These were advanced in seven deep boreholes at the HAR site to characterize shear and compressional wave velocity (V_s and V_p) profiles with depth. Surveys were performed at BPA-5, BPA-25, BPA-39, BPA-47, BPA-48, BPA-49, and BPA-50 by GeoVision of Corona, California (Reference 2.5.4-226, Reference 2.5.4-227). Suspension logging surveys were conducted using an OYO suspension logging probe. In this method, a seismic source generated a compression wave within the borehole, which propagated as compression and shear waves along the borehole wall. The time difference for the waves to arrive at a pair of geophones located at higher elevations than the source within the borehole, separated by a known distance of 1 m (3 ft.), allowed calculation of the V_S and V_p of the rock or soil near the borehole wall between the geophones.

The probe was moved a vertical distance of 0.4 m (1.35 ft.) between measurements to generate a semi-continuous profile of V_p and V_S within each tested borehole. GeoVision also performed acoustic televiewer, caliper, and verticality deviation surveys in these boreholes. Suspension logging profiles were performed in Boreholes BPA-5, BPA-25, and BPA-39 in May 2006, and in Boreholes BPA-47 through BPA-50 in December 2006.

2.5.4.4.1.2 Acoustic Televiewer Surveys

GeoVision of Corona, California, performed acoustic televiewer surveys within the same seven boreholes as the suspension logging surveys (Reference 2.5.4-226, Reference 2.5.4-227). Surveys were performed using a High-Resolution Acoustic Televiewer probe (HiRAT). The device provided high-resolution, oriented images of the borehole walls presented in "pseudo-color." The probe scanned the borehole wall using an ultrasound beam, and the amplitude and travel time of the reflected signal were recorded simultaneously by the probe. The dip and orientation of the probe were also recorded. Features such as fractures and bedding planes appear as sinusoidal traces on the oriented images produced by these surveys. The traces were then used to calculate the strike and dip of such features. Acoustic televiewer surveys were performed in May and December 2006.

2.5.4.4.1.3 Downhole Surveys

GeoVision of Corona, California, performed downhole surveys at BPA-48 and BPA-49. These surveys were performed as an alternate method to characterize the V_S profiles with depth to supplement the suspension logging results (Reference 2.5.4-226). In this method, an oriented geophone probe was secured against the borehole wall at a known depth, and a shear wave was created at the ground surface by striking a horizontal board with a sledge hammer. The time for shear wave arrival at the downhole probe allowed calculation of the shear wave velocity between the probe and the ground surface. The probe was moved a vertical distance of 1.5 to 3 m (5 to 10 ft.) between measurements. The data

were then processed to generate a profile of V_S with depth for each tested borehole. Downhole surveys were performed in December 2006.

2.5.4.4.1.4 Seismic Refraction Surveys

Technos, Inc., of Miami, Florida, performed three lines each at HAR 2 and HAR 3. These surveys were performed to characterize the V_p profiles in soil and shallow bedrock (Reference 2.5.4-228). Approximately 975 linear m (3200 linear ft.) of survey lines were performed at the two units. A Plant north-south survey line of approximately 244 m (800 ft.) and two shorter Plant east-west lines approximately 122-m (400-ft.) long were performed each at HAR 2 and HAR 3, at locations shown on Figure 2.5.4-203A and 2.5.4-203B, respectively. Data were recorded using a spread of 24 geophones spaced 3 m (10 ft.) apart. Compression waves were generated using an elastic weight drop hammer and a 4.5-kilogram (kg) (10-pound [lb.]) sledge hammer. Multiple overlapping spreads were performed to provide coverage along the survey lines. Work was performed in May and September 2006.

2.5.4.4.1.5 Magnetometer Surveys

Technos, Inc., of Miami, Florida, performed three lines each at HAR 2 and HAR 3. These surveys were performed to identify diabase dikes located near the AP1000 structures and to trace previously identified dikes at the HAR site, to assess the potential effects of these dikes on subsurface uniformity. Several diabase dikes were encountered during the HNP investigation, as summarized in Subsection 2.5.1. The iron-rich dikes are readily detected by magnetometer methods. (Reference 2.5.4-228). Three Plant north-south magnetometer survey lines were performed across HAR 2 and HAR 3. Additional magnetometer survey lines were advanced to the east of the HAR, perpendicular to the projected trace of known diabase dikes reported in the HNP FSAR (Reference 2.5.4-201), as shown on Figure 2.5.4-213. Data were collected using a G-858 cesium vapor Magmapper system. Total field and vertical gradient measurements were obtained simultaneously, and a base station sampled data every 15 seconds for diurnal corrections. Work was performed in May 2006.

2.5.4.4.1.6 Spectral Analysis of Surface Waves Surveys

The SASW surveys were performed to characterize the V_s profiles within soil and shallow bedrock near the nuclear islands (Reference 2.5.4-226). SASW geophysical surveys were performed at four locations (two each at HAR 2 and HAR 3) to characterize the V_s profile to a depth of between 21.3 to 30.5 m (70 to 100 ft.) bgs. Measurements of surface wave velocity were made with multiple geophone spacings at the ground surface, and the resulting dispersion curves were used to back-calculate the one-dimensional V_s profile with depth for each survey. A pair of surveys was performed at each location, with geophones aligned in approximately orthogonal directions for each survey. The centerpoints of each geophone array (the locations of the one-dimensional profiles) are shown on Figures 2.5.4-203A and 2.5.4-203B for HAR 2 and HAR 3, respectively, along

with the survey lines. SASW surveys were performed near Boreholes BPA-5 and BPA-48 at HAR 2, and near Boreholes BPA-25 and BPA-49 at HAR 3. SASW surveys were performed in December 2006.

2.5.4.4.1.7 Multi-Channel Analysis of Surface Waves Surveys

In addition to the above, Technos, Inc. of Miami, Florida, attempted one set of orthogonal multi-channel analysis of surface waves (MASW) profiles each at HAR 2 and HAR 3 (Reference 2.5.4-229). These surveys were attempted to characterize the spatial variability in V_s within soil and shallow bedrock near the nuclear islands. Upon review of the MASW survey results, V_s interpreted by this method are not consistent with results from other methods, and the method is not considered representative of the HAR 2 and HAR 3 subsurface materials. The MASW results are therefore not further considered in this FSAR.

2.5.4.4.2 Geophysical Survey Investigation Results

Four different geophysical survey methods were used to characterize subsurface engineering properties at the HAR site: suspension logging, downhole, SASW, and seismic refraction. Results of these surveys establish V_S and V_p within the soil and rock underlying and adjacent to the nuclear islands at HAR 2 and HAR 3. Magnetometer surveys were also performed to characterize locations of diabase dikes proximate to HAR 2 and HAR 3. Brief descriptions of these survey methods were presented in Subsection 2.5.4.4.1.

2.5.4.4.2.1 Suspension Logging Surveys

Charts showing the V_S results from the suspension logging surveys compared with other methods at or near each borehole are shown on Figures 2.5.4-209A, 2.5.4-209B, 2.5.4-209C, 2.5.4-209D, 2.5.4-209E, 2.5.4-209F, and 2.5.4-209G (Reference 2.5.4-226, Reference 2.5.4-227). Summary statistics for V_S, V_p, and Poisson's ratio at measured elevations below 67.1 m (220 ft.) at HAR 2 and HAR 3 are presented in Table 2.5.4-211.

The following subsections discuss suspension logging survey results at HAR 2 and HAR 3.

2.5.4.4.2.1.1 HAR 2

The following trends are indicated by the suspension logging V_S data at Boreholes BPA-5, BPA-47, and BPA-48 at HAR 2:

- V_s is greater than 1372 m/sec (4500 fps) at all boreholes below the HAR 2 nuclear island basemat elevation (67.1 m [220 ft.]), except within the isolated gray shaley siltstone layer that dips to the east across the nuclear island (HAR 2 Marker Bed B, as described in Table 2.5.4-202). V_s within this marker bed drops to between 1116 to 1250 m/sec (3660 to 4100 fps) over an approximate 1-m (a few feet) interval at each borehole, quickly transitioning

to higher velocities above and below the marker bed, as shown on Figures 2.5.4-209A, 2.5.4-209D, and 2.5.4-209E. The data indicate that the lower V_s within this stratum is continuous along dip beneath the nuclear island and is associated with increased fracture density and some isolated clay seams along bedding planes.

- Figure 2.5.4-214A shows the superimposed V_s profiles from the three suspension logging profiles at HAR 2, at their measured elevations. As shown, the results indicate generally increasing V_s with depth at HAR 2. The scatter observed among borehole results at the same elevation is largely due to gently dipping strata under HAR 2.

- A comparison of the V_s data between the three boreholes indicates that V_s is reasonably consistent along dip-correlated strata. On Figure 2.5.4-214B, the V_s profiles for Boreholes BPA-5 and BPA-48 are elevation-adjusted such that V_s measured in common dip-related strata are shown overlying one another (shown at true elevation for BPA-47). As observed by comparison of these profiles, common strata between the boreholes typically have similar V_s below the nuclear island basemat grade, and the V_s profiles form a distinctive pattern with depth that is similar for all three boreholes.

- For dip-correlated strata below the nuclear island subgrade elevation of 67.1 m (220 ft.), V_s measured in the boreholes fall within 20 percent of the average among the three holes, except at a few isolated intervals (Figure 2.5.4-214B). The exceptions typically span only one or two suspension logging data points, representing up to approximately 1 m (3 ft.) in thickness.

As listed in Table 2.5.4-202, the dip calculated by matching the V_s profile patterns with depth at the three boreholes has a magnitude of 8.9 degrees, directed 91 degrees clockwise from north. This dip angle is nearly identical to the dip listed in Table 2.5.4-202 for HAR 2 Marker Bed B, which was based on lithologic comparisons of marker beds using a different set of boreholes. The slight difference in dip direction is not unexpected considering the relatively shallow dip magnitude.

2.5.4.4.2.1.2 HAR 3

The following trends are indicated by the V_s data at Boreholes BPA-25, BPA-49, and BPA-50 at HAR 3:

- The transition from low V_s in soil to higher V_s in bedrock occurs more gradually over a deeper elevation range at HAR 3 than at HAR 2, reflecting the deeper extent of soil and weathered bedrock at HAR 3 as shown on Figures 2.5.4-209B, 2.5.4-209F, and 2.5.4-209G.

- Below the HAR 3 nuclear island basemat elevation (67.1 m [220 ft.]), V_s is typically greater than 1067 m/sec (3500 fps), with a few exceptions. At

BPA-49, isolated intervals have V_s slightly less than 1067 m/sec (3500 fps) between elevation 64.0 and 67.1 m (210 and 220 ft.), associated with thin clay seams observed at those intervals. Occasional lower velocity intervals are associated with isolated clay seams observed in the boreholes. Within these deeper intervals, V_s decreases to below 1219 m/sec (4000 fps) with velocities greater than 1524 m/sec (5000 fps) immediately above and below. Specific V_s measurements close to or less than 1067 m/sec (3500 fps) at these boreholes consist of the following:

- BPA-49: At 59.3 and 58.8 m (194.6 and 193.0 ft.), V_S decreases to 1116 and 1167 m/sec (3660 and 3830 fps).

- BPA-50: At 50.6 and 50.5 m (166.1 and 165.6 ft.), V_S drops to 1015 and 1036 m/sec (3330 and 3400 fps).

- Figure 2.5.4-215A shows the superimposed V_s profiles from the three suspension logging borehole surveys at HAR 3, at their measured elevations. The results show generally increasing V_s with depth at HAR 3. The scatter observed among borehole results at the same elevation is largely due to dipping strata under HAR 3. Above elevation 67.1 m (220 ft.), the differences in V_s are also associated with differences in degree of weathering; at BPA-50, existing ground elevation is higher (83.2 m [272.98 ft.]) than at BPA-25 or BPA-49 (81.2 and 79.8 m [266.37 and 261.87 ft.], respectively), and sound rock is present at a correspondingly higher elevation at BPA-50.

- Comparison of the V_s data between the three boreholes at HAR 3 indicates that V_s is reasonably consistent for dip-correlated strata. On Figure 2.5.4-215B, the V_s profiles for Boreholes BPA-25 and BPA-50 are elevation-adjusted such that V_s measured in common dip-related strata are shown overlying one another (shown at true elevation for BPA-49). Inspection of the profiles shows that common strata between the boreholes typically have similar V_s below the nuclear island basemat grade, and the V_s profile forms a distinctive pattern with depth that is similar for all three boreholes.

- For dip-correlated strata below the nuclear island subgrade elevation of 67.1 m (220 ft.), V_s measured in the boreholes fall within 20 percent of the average among the three holes, except at a few isolated data points (Figure 2.5.4-215B).

As listed in Table 2.5.4-202, the dip calculated by matching the V_s profile patterns with depth at the three boreholes has a magnitude of 19.9 degrees, directed 109 degrees clockwise from north. The dip magnitude and direction are similar to the dip listed in Table 2.5.4-202 for HAR 2 Marker Beds A and B, which were based on lithologic comparisons of marker beds using a different set of boreholes, and dip calculated from acoustic logging surveys, as described in Subsection 2.5.4.1.1.2.

2.5.4.4.2.2 Acoustic Televiewer Surveys

GeoVision interpreted the sinusoidal trace data for fractures and bedding planes from the acoustic televiewer surveys to calculate the dip and orientation (azimuth) of these features (Reference 2.5.4-226 and Reference 2.5.4-227). These results were then analyzed using the software program DIPS, Version 5.106, by RocScience, Inc. to evaluate trends in fractures and bedding planes at HAR 2 and HAR 3. Stereonet plots of bedding planes and fractures at HAR 2 and HAR 3 were created, as shown on Figures 2.5.4-216A and 2.5.4-216B. The mean dip magnitude and direction of groups of these features were then calculated and are shown on the stereonet plots.

The dip magnitude and direction of features reported as bedding planes in the GeoVision report are representative of nonconformable bedding local to the specific borehole (Reference 2.5.4-230). Such features are representative of features such as sandstone cross-beds, which are not true stratigraphic bedding planes. However, the aggregate mean dip and direction of all such features grouped by HAR 2 and HAR 3 provide an estimate of the true stratigraphic dip at each site. The average dip magnitudes and directions of the bedding planes at HAR 2 and HAR 3 are shown on Figures 2.5.4-216A and 2.5.4-216B, and shown on Table 2.5.4-202. These results are similar to values presented in Subsection 2.5.4.1.1.2, as determined by other methods.

The dip magnitude and direction of features reported as fractures in the GeoVison report were grouped into two sets at HAR 2 and HAR 3. Fracture Set 1 at both HAR 2 and HAR 3 were close to the dip magnitude and direction calculated for bedding features. Fracture Set 2 at HAR 2 and HAR 3 are unrelated to Fracture Set 1 and the bedding planes. The orientation of the mean planes for Fracture Set 2, both weighted and unweighted, were in the opposing quadrant of the stereonet for HAR 2 and HAR 3, and the dip magnitude was greater. These results are shown on Figures 2.5.4-216A and 2.5.4-216B. The orientation and dip magnitude of Fracture Set 2 is consistent with a primary joint set characterized in the HNP FSAR for Unit 1, as summarized in Subsection 2.5.1.2.4.

The strata dip and direction estimated using the acoustic televiewer results are considered secondary to the other methods described in Subsection 2.5.4.1.1.2. This is because the dip magnitudes calculated using the acoustic televiewer survey results are sensitive to the assumed borehole diameter used in the calculation (i.e., 10 cm [4 in.]). Results from stratigraphic interpretation between boreholes and V_S profile matching are considered more representative of the true bedding dip, and provide a better estimate of the average value over the large area under the HAR 2 and HAR 3 nuclear island footprints.

2.5.4.4.2.3 Downhole Velocity Surveys

The interpreted V_S profiles from the downhole surveys at BPA-48 and BPA-49, along with results of other V_S profiling methods (suspension logging and SASW),

are shown on Figures 2.5.4-209E and 2.5.4-209F, respectively (Reference 2.5.4-226).

As presented on Figures 2.5.4-209E and 2.5.4-209F, the results of the downhole profiling methods are consistent with the suspension logging results. The downhole survey method evaluates V_S 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 ft.) for the suspension logging method. Due to the resolution of the downhole data and frequency range of the source signal, the downhole method does not provide the same high resolution of thin V_S variations with depth as is provided by the suspension logging method. For this reason, V_S variations within intervals detected by suspension logging are not as apparent in the downhole results.

2.5.4.4.2.4 SASW Surveys

The resulting V_S profiles from the SASW surveys performed near Boreholes BPA-5, BPA-25, BPA-48, and BPA-49, along with V_S results from other methods, are plotted on Figures 2.5.4-209A, 2.5.4-209B, 2.5.4-209E, and 2.5.4-209F, respectively (Reference 2.5.4-226). As shown on these figures, the results of the SASW surveys generally are consistent with the suspension logging and downhole results, although more smoothing occurs with depth because of the decrease in sensitivity of the dispersion curve to changes in V_S with depth.

2.5.4.4.2.5 Seismic Refraction Surveys

Figures 2.5.4-217A and 2.5.4-217B present the V_p profiles resulting from the seismic refraction surveys at HAR 2 and HAR 3, respectively (Reference 2.5.4-228). Three V_p layers were identified at each site:

- Layer 1 representing unconsolidated residual soils.
- Layer 2 representing altered or weathered rock.
- Layer 3 representing generally unweathered and sound bedrock.

As shown on Figures 2.5.4-217A and 2.5.4-217B, Layer 1 and Layer 2 extend slightly deeper at HAR 3 than at HAR 2, reflecting the greater depth of soil and weathered rock at HAR 3 observed in site boreholes. The top of Layer 3 was interpreted to be between elevations 73.2 and 76.2 m (240 and 250 ft.) at HAR 2, and between 70.1 and 73.2 m (230 and 240 ft.) at HAR 3.

The interpreted value of V_p for Layer 2 ranges between 1586 and 1963 m/sec (5203 and 6439 fps) at HAR 2, and between 967 and 1155 m/sec (3174 and 3788 fps) at HAR 3. The interpreted values of V_p for Layer 3 are relatively consistent between the units, ranging between 3116 and 3660 m/sec (10,223 to 12,007 fps). These values of V_p for Layer 3 are also consistent with the range of

V_p from suspension logging surveys below elevation 67.1 m (220 ft.), as listed in Table 2.5.4-211.

The top of Layer 3 is shown in plan view on Figure 2.5.4-218. The elevations of the top of Layer 3 indicated on Figure 2.5.4-218 are generally lower than the elevations of the top of sound rock indicated on Figure 2.5.4-206A and Figure 2.5.4-206B. This is consistent with the trend indicated in the suspension logging results of V_s greater than 3000 fps (roughly equivalent to V_p of 6000 fps) near the top of sound rock and increasing with depth, such that the V_p results indicated by Layer 3 in the seismic refraction surveys are present below the top of sound rock.

As with the SASW and downhole surveys, the seismic refraction survey method does not allow sufficient resolution to characterize the thin layers with changes in V_p indicated by the suspension logging profiles. Further, the seismic refraction method cannot detect velocity inversions, where lower V_p material is present deeper than higher V_p material. Therefore, the top of Layer 3 indicated on Figures 2.5.4-217A, 2.5.4-217B, and 2.5.4-218 is considered to represent the shallowest encounter of fresh or slightly weathered rock at the profile locations. Thin intervals of very weak rock or clay are present below the indicated top of Layer 3 at some locations at both HAR 2 and HAR 3, which are not indicated on the seismic refraction profiles.

2.5.4.4.2.6 Magnetometer Surveys

The magnetometer surveys characterize the locations of diabase dikes in the proximity of HAR 2 and HAR 3. Figure 2.5.4-213 presents the magnetometer survey locations and corresponding magnetic response (Reference 2.5.4-228). In addition to the Plant north-south profiles performed at HAR 2 and HAR 3 (named Area A and Area B on Figure 2.5.4-213), surveys were performed at Area C and Area D to characterize the northern projections of “East Dike 1” and “East Dike 2” reported in the HNP FSAR. Surveys were also performed at Area E over a cluster of known diabase dikes, to confirm the ability for the magnetometer survey method to detect the dikes.

As shown on Figure 2.5.4-213, the magnetic responses indicate that dikes are not present at HAR 2 or HAR 3. The magnetic responses near the west ends of the profiles at HAR 2 are caused by a railroad spur in that area.

The northern projections of East Dike 1 and East Dike 2 were detected by magnetic responses at both Area C and Area D, as shown on Figure 2.5.4-213. This confirms that these dikes are not present under HAR 2 or HAR 3 but rather pass by to the east. A small spike is shown approximately 100 feet west of the response for East Dike 2 in the survey at Area D. This response is likely due to a fire water supply line in this area.

2.5.4.4.2.7 MASW Surveys

As summarized in Subsection 2.5.4.4.1, MASW surveys were attempted at the HAR sites. Although the MASW method was implemented in accordance with industry practice at HAR 2 and HAR 3, wavelengths that were detected by the two-dimensional (2-D) MASW method were not an accurate representation of information required for a representative MASW analysis, which likely resulted in an underestimation of V_s at some locations at the site. Therefore, the MASW survey results have not been used to characterize V_s for the HAR sites, and the results are not further presented herein.

2.5.4.4.2.8 Criteria for Use of Geophysical Survey Results as Design Parameters

Multiple geophysical survey methods were implemented at the HAR sites. The following information summarizes how the data from each of these methods have been used for engineering analyses, and the basis for this use.

- **Suspension Logging Survey Data.** Suspension logging profiles provide point measurements of V_s and V_p at discrete depths. As presented in Subsection 2.5.4.4.2.1, the V_s profiles among different boreholes within dip-related strata correlate very well. 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 2.5.4-209E and 2.5.4-209F. Therefore, 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.

- **SASW Survey Data.** SASW profiling provides surface wave measurements for indirect interpretations of V_s in the shallow subsurface (to approximate elevations of 21.3 to 30.5 m [70 to 100 ft.] at HAR 2 and HAR 3). The method provides V_s data averaged over a larger lateral distance than either the suspension logging or downhole velocity profiles. The SASW V_s results are consistent with results from the suspension logging and downhole velocity results. Therefore, they serve as another confirmation of the depth-averaged suspension logging profiles at shallow depths in rock. The SASW V_s values are also obtained in soils near the ground surface where suspension logging and downhole results are not available.

- **Seismic Refraction Survey Data.** Seismic refraction profiles provide measurements of V_p in the shallow subsurface, based on compressional

wave arrival times. However, the method is insensitive to V_p inversions (where lower V_p material underlies higher), and does not provide a direct measurement of V_s. Trends in V_p from the seismic refraction surveys are generally consistent with V_s and V_p results from other methods, as described in Subsection 2.5.4.4.2.5, and provide confirmation of these results. The seismic refraction data are not used directly for engineering analyses.

- **Magnetometer Survey Data.** The magnetometer survey data indicate that diabase dikes are not present under the HAR structures. The data confirm the northern trace of several dikes identified during the HNP investigation, as described in Subsection 2.5.4.4.2.6.

2.5.4.5 Excavations and Backfill

HAR COL 2.5-7

Soil and rock excavation will be required to found the HAR nuclear islands on sound rock at approximate subgrade elevation 67.1 m (220 ft.). This subsection describes the preliminary excavation and backfill plans for the nuclear islands, including planned excavation extents and methods, properties of backfill adjacent to seismic Category I structures and under seismic Category II structures, and groundwater dewatering methods that will be appropriate during construction.

2.5.4.5.1 Excavation Extents

Prior to excavation of the HAR structures, the site will be graded to approximate elevation 79.2 m (260 ft.). Nominal site grade is elevation 79.6 m (261 ft.), but the surrounding grade will be lower to accommodate site grading, drainage, and flooding requirements. Nuclear island excavation sideslopes of between 1.5H:1V to 2H:1V (horizontal to vertical) are planned from ground surface to the top of sound rock. Rock excavation sideslopes of between 0.25H:1V and 0.5H:1V are planned within sound rock. A 3-m- (10-ft.-) wide horizontal bench is planned at the top of sound rock excavations. These planned excavation slopes may change depending on excavation conditions and future construction considerations.

Figures 2.5.4-211A and 2.5.4-211B show the planned nuclear island excavation limits at HAR 2 along west-southwest to east-northeast (“Plant South” to “Plant North”) and north-northwest to south-southeast (“Plant West” to “Plant East”) cross sections, respectively. Figures 2.5.4-212A and 2.5.4-212B indicate this same information for HAR 3. Note that the excavation depth to the top of sound rock will vary along the sides of the nuclear islands. Therefore, the corresponding excavation extents (i.e., the elevation at which the flatter soil excavation slopes transition to the steeper rock excavation slopes) will likely vary along the length of the nuclear island sidewalls from the elevations shown on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B, based on conditions encountered during construction.

Based on slope stability analyses, the excavation limits shown on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B will result in acceptable FS for rock and soil slope stability during construction. Stability analyses were performed using the software package SLIDE Version 5.026 by RocSoft. The deeper extent of soil and weathered rock at HAR 3 relative to HAR 2 is represented in the HAR 3 excavation limits, especially on the north-northwest and west-southwest (“Plant West” and “Plant South,” respectively) sides of the HAR 3 nuclear island. The northwest corner of the HAR 2 nuclear island will also likely require deeper soil excavation than shown on Figures 2.5.4-211A and 2.5.4-211B, due to the deeper extent of soil and weathered rock at Borehole BPA-2. Bedding planes, and associated relatively weak clay intervals within bedrock, dip towards the excavation on the north-northwest and west-southwest (“Plant West” and “Plant South”) sides of both HAR units. Potential sliding along these bedding planes during construction was considered during the excavation stability analyses. Results confirm that the excavation slopes shown on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B will remain stable during construction without external support.

Excavation and backfill extents for the seismic Category II structures (Annex Buildings and the first bays of the Turbine Buildings) and for other structures are shown on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B. For the seismic Category II structures:

- Soil and highly weathered rock will be excavated from beneath the HAR Annex Buildings (seismic Category II structures) and replaced with concrete fill.

- Soil will be excavated beneath the HAR Turbine Buildings down to partially weathered rock and replaced with either compacted granular fill or concrete fill. For the Turbine Buildings:

    - Partially weathered rock exposed after soil excavation will be inspected for subgrade suitability. Intervals of highly or completely weathered (soil-like) rock or soil infill will be over-excavated and replaced with compacted granular fill or concrete fill. The depth and extent of such over-excavations will be determined based on the results of the subgrade inspection.

    - If compacted granular fill will be used, a minimum thickness (T_{min}) of compacted granular fill will be placed beneath the Turbine Building foundations. The value of T_{min} will be determined after the Turbine Building bearing pressures and subgrade modulus requirements are finalized. Limited rock over-excavation in isolated locations of high rock elevation, where encountered, will likely be required.

    - If compacted granular fill will be used, the zone of excavation and fill placement will extend beyond the Turbine Building walls to a

horizontal distance of at least twice the adjacent excavation depth beneath the Turbine Building foundations.

- The items listed here will apply to the entire footprint of each Turbine Building, not just to the Seismic Category II portion.

A 2-inch gap will be maintained between the Turbine Building basemats and the nuclear islands by use of a durable joint filler. If concrete fill is placed beneath the Turbine Building basemats, the 2-inch gap and joint filler will be installed throughout the depth of concrete fill between the concrete fill and the nuclear island.

Foundation design recommendations for other nonsafety-related structures are not included in this FSAR, and will be finalized prior to construction.

After the AP1000 structures are constructed and the excavations are backfilled, the excavation slopes will no longer be present and will therefore not affect the performance of the safety-related structures.

2.5.4.5.2 Excavation Methods and Subgrade Improvement

At both HAR 2 and HAR 3, rock at the nuclear island subgrade (elevation 67.1 m [220 ft.]) will need to satisfy the following criteria:

- Rock borings will have both 90 percent recovery and 50 percent RQD at the subgrade elevation.

- Rock will be fresh to slightly weathered.

- Subgrade will not have solution features, loose rock, nor open or soil-filled joints or fractures.

Any rock that does not satisfy these criteria at the nuclear island subgrade elevation will be removed 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, ripping of moderately weathered rock, and pre-splitting and controlled blasting of sound rock, as was performed for the HNP (Reference 2.5.4-201).

- 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 Subsection 2.5.4.6.2. Based on observations during construction of the HNP, 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 weathered or 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 and management of seepage and piping.

- Complete geologic mapping of the subgrade and excavation sidewalls will be required prior to subgrade improvement activities.

- Excessively fractured or weathered rock will be over-excavated to the bottom of the weathered or fractured zone and filled with dental concrete. If the depth of fractured or weathered rock below the basemat subgrade becomes impractical to excavate, an alternate method of foundation improvement, such as pressure grouting, may be considered.

- Soil-filled joints or fractures 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 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.

2.5.4.5.3 Properties of Backfill Adjacent to Nuclear Islands

Backfill materials will be placed adjacent to the sidewalls of the nuclear islands. The backfill material adjacent to the nuclear island sidewalls will consist of concrete fill and compacted backfill. Concrete fill will be placed as backfill between rock excavations and the sidewalls of the nuclear islands below elevation 69.3 m (227.5 ft.), and at other select locations. Backfill soil will be placed above the concrete fill. Anticipated properties of the concrete fill and soil backfill are summarized herein. Final selection and testing of soil backfill sources will be performed prior to construction.

The characteristics and use of the materials described in Table 2.5.4-212 are as follows:

- Concrete fill. This will consist of structural (mass) concrete with no reinforcing. The concrete fill is required below elevation 69.3 m (227.5 ft.) to transfer passive resistance from adjacent rock to prevent sliding during the SSE. The concrete fill will have a minimum unconfined compressive

strength of 363 kPa (2500 psi). Concrete fill will also be used above elevation 69.3 m (227.5 ft.) at other select locations such as under the Annex Building foundations. It will also be used to fill and smooth excavated rock surfaces below the nuclear island subgrade.

- Compacted granular fill. This will consist of granular, well-graded sand and gravel. This material may include select material segregated during excavation of on-site sandy or gravelly soils, but will more likely be imported from off-site sources. Sources of off-site granular fill are plentiful in the site vicinity. Compacted granular fill may be placed between excavation sideslopes and the sidewalls of the nuclear islands above elevation 69.3 m (227.5 ft.).

- Compacted cohesive fill. This will consist of cohesive soils present at the HAR 2 and HAR 3 sites, with USCS classifications of lean clay (CL), silt (ML), clayey sand (SC), or silty sand (SM). Compacted cohesive fill may be placed between excavation sideslopes and the sidewalls of the nuclear islands above elevation 69.3 m (227.5 ft.), in areas where the backfill will not support overlying structures adjacent to the nuclear island.

Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B show the approximate planned limits of backfill adjacent to nuclear island structures at HAR 2 and HAR 3, respectively. Table 2.5.4-212 is a summary of the anticipated engineering properties for each type of backfill. The engineering properties listed in Table 2.5.4-212 will be included in the construction specifications. Backfill material sources, once identified, will be tested to demonstrate that they are consistent with the properties in Table 2.5.4-212. The development of the backfill specification, and associated testing and approval of backfill sources, will occur prior to construction.

2.5.4.6 Groundwater Conditions

HAR COL 2.5-6

HAR COL 2.5-8

Groundwater conditions for the HAR site were established by periodic measurements of groundwater levels since well installation in July 2006, as summarized in 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 and piezometers are presented in Subsection 2.4.12. Figures 2.4.12-203, 2.4.12-204, 2.4.12-205, 2.4.12-206, 2.4.12-207, 2.4.12-208, 2.4.12-209, and 2.4.12-210 show groundwater elevations and contours at the site in overburden and bedrock during monitoring events in August 2006, November 2006, February 2007, and May 2007, respectively.

Groundwater elevations and gradients at these wells are summarized in Tables 2.4.12-206 and 2.4.12-207, respectively.

As described in Subsection 2.4.12.5, post-construction groundwater elevations are not anticipated to exceed elevation 78.6 m (258 ft.). The nuclear islands will be founded on sound rock, and groundwater conditions are not expected to adversely affect foundation performance.

2.5.4.6.2 Construction Dewatering

Dewatering will be required to maintain groundwater below elevation 67.1 m (220 ft.) during excavation and construction of the nuclear islands. Expected construction dewatering flowrates and anticipated dewatering methods are summarized in this subsection.

As discussed in Subsection 2.5.4.2.2.6, hydraulic conductivity (slug) tests were performed in each of the monitoring wells installed near the HAR sites. Results are presented in Table 2.4.12-208. Monitoring wells MWA-3S, MWA-3D, MWA-8S, and MWA-8D are located at the HAR nuclear islands, with “S” wells set in soil and weathered rock, and “D” wells set below top of rock. Expected construction dewatering flow rates were calculated based on the hydraulic conductivity results from slug tests, pre-construction groundwater elevations, and excavation limits using the two dimensional (2-D) finite-element modeling software package PlaxFlow.

Models were developed to represent orthogonal cross sections at HAR 2 and HAR 3, which were then used to calculate the quantity of groundwater inflow into the excavation. In these calculations, the steady-state groundwater head during construction was assumed to be at an elevation of 79.2 and 77.7 m (260 and 255 ft.) at HAR 2 and HAR 3, respectively, based on groundwater elevations at the HAR monitoring wells. The hydraulic conductivity of all soils was modeled as 0.7 m/day (2.3 ft/day). Likewise, the hydraulic conductivity of all rock was modeled as 0.06 m/day (0.2 ft/day). These are based on values from field hydraulic conductivity test results in soil and rock, respectively, as summarized in Table 2.4.12-208.

Based on the analyses, construction dewatering flow rates of approximately 9.5×10^4 liters per day (lpd) (2.5×10^4 gallons per day [gpd]) and 1.70×10^5 lpd (4.5×10^4 gpd) are anticipated at HAR 2 and HAR 3, respectively. Most of this flow will be through overburden soils in the upper 3 m (10 ft.) of excavation at HAR 2, and 7.6 m (25 ft.) of excavation at HAR 3. Additional flow will pass through fractures and joints in the underlying rock.

These groundwater inflow rates at HAR 2 and HAR 3 can be managed by pumps installed within the excavation. If unexpectedly high flow rates are encountered during excavation, more active methods of dewatering (such as a perimeter well network) will be considered and could be implemented during the course of construction. Construction dewatering methods will be specified in the

excavation, subgrade improvement, and verification program (see Subsection 2.5.4.5.2).

2.5.4.7 Response of Soil and Rock to Dynamic Loading

HAR COL 2.5-2

HAR 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 HAR site and vicinity are presented in Subsection 2.5.3. As stated therein, several faults have been identified within the site and vicinity, including the "Site Fault" located near the HNP. However, the NRC staff concurred that all the faults in the HNP site and Harris Dam areas predate the mineralization that formed after regional deformation — at least 2.5 million years ago, and likely more than 136 to 190 million years ago. No zones of Quaternary deformation that would require additional investigation are identified within the HAR site area. None of the mapped bedrock faults within a 40-km (25-mi.) radius or lineaments within an 8-km (5-mi.) radius of the HAR site are assessed to be a capable tectonic source.

Results of V_s and V_p surveys at the HAR site, including profiles and summary statistics, are presented in Subsection 2.5.4.4. Results of V_s from suspension logging, downhole logging, and SASW surveys performed within and near boreholes at HAR 2 and HAR 3 are presented on Figures 2.5.4-209A, 2.5.4-209B, 2.5.4-209C, 2.5.4-209D, 2.5.4-209E, 2.5.4-209F, and 2.5.4-209G. Results of V_p from seismic refraction surveys are presented on Figure 2.5.4-217A, 2.5.4-217B, and 2.5.4-218. Interpretations of these data relative to the site geologic conditions are presented in Subsection 2.5.4.4.2. These data were used to develop site-specific dynamic velocity profiles for site response analyses as presented in Subsection 2.5.2.5.

As discussed in Subsection 2.5.2.5, the GMRS is defined at the top of the first competent layer, which is above the subgrade elevation of 67.1 m (220 ft.) for the safety-related structures. The V_s profile of rock below safety-related structures is greater than 914 m/sec (3000 fps). The Annex Buildings (Seismic Category II structures) will also be founded on sound rock or concrete fill over rock, with V_S greater than 914 m/sec (3000 fps). Such materials are considered to be too stiff to perform laboratory tests using resonant column/cyclic torsional shear tests for reliable modulus degradation and damping versus shear strain determinations, and for this reason such testing was not performed. Subsection 2.5.2.5.1.4 describes the modulus degradation and damping relationships that were used to develop the GMRS and the FIRS for HAR 2 and HAR 3.

2.5.4.8 Liquefaction Potential

HAR COL 2.5-9 The potential for liquefaction of soils beneath and adjacent to Seismic Category I and II structures at the HAR site was evaluated, as summarized herein.

The HAR 2 and HAR 3 nuclear islands (Seismic Category I structures) will each be founded on sound rock. The Annex Buildings (Seismic Category II structures) will be founded at or near the top of sound rock. Where the Annex Building foundations are above sound rock, overburden soils and weathered rock will be excavated to sound rock and replaced with concrete fill under the seismic Category II portions of the Annex Building foundations, as described in Subsection 2.5.4.5.1. The Turbine Buildings (the first bay of each is a Seismic Category II structure) may be founded on either compacted granular fill or concrete fill over partially weathered and sound rock.

The evaluation of the liquefaction resistance of compacted granular fill is summarized in the following subsections. This evaluation is based on the backfill properties specified in Table 2.5.4-212, which were specified in part to prevent the development of liquefaction. A liquefaction screening evaluation of native soils present at the HAR sites is also presented. The screening evaluation demonstrates that native soils are predominantly cohesive and therefore are not susceptible to liquefaction.

2.5.4.8.1 Liquefaction Resistance of Nonsafety-Related Compacted Granular Fill

Liquefaction resistance of compacted granular fill was evaluated using two separate empirical methods:

- The first method is based on correlations of the cyclic resistance ratio (CRR) with V_s as presented in Andrus et al., 2004 (Reference 2.5.4-239), which is an update to the V_s-based procedure in Youd et al., 2001 (Reference 2.5.4-240).

- The second method is considered confirmatory to the first method, and is based on correlations between SPT blowcount and CRR, as presented in Youd et al., 2001 (Reference 2.5.4-240). In this method, the minimum relative compaction listed in Table 2.5.4-212 is correlated to relative density, and in turn correlated to equivalent SPT blowcount at representative depths of backfill for the evaluation.

The following ground motion parameters were applied in these methods:

- **Design Earthquake Magnitude (M) = 7.1.** This is selected as the mean moment magnitude of the Charleston source zone, as summarized in Subsection 2.5.2.

- **Peak Ground Acceleration (PGA) = 0.173g.** This is based on the GMRS for HAR 3 (PGA of 0.137g) as presented in Subsection 2.5.2, divided by the factor 0.792 to remove the contribution of CAV adjustments. The GMRS is defined at the top of the shallowest competent geologic layer, and incorporates site amplification relative to deep hard rock motions. The PGA of 0.173g is multiplied by an additional amplification factor of 1.2 to conservatively account for site amplification affects between the GMRS elevation and site grade.

In both methods, an FS against liquefaction is calculated based on the ratio of the CRR to the cyclic stress ratio (CSR). As presented in Regulatory Guide 1.198, soil elements with FS greater than 1.4 would suffer relatively minor cyclic pore pressure generation, and are considered non-liquefiable. The groundwater elevation for the FS evaluation was conservatively assumed to be at site grade.

Evaluations based on the first method confirm that FS against liquefaction will be greater than 1.4 throughout the backfill depth for backfill V_s greater than 500 fps. Evaluations based on the second method confirm that FS against liquefaction will be greater than 1.4 throughout the backfill depth if it is compacted to 95 percent relative compaction. Both of these parameters are specified in Table 2.5.4-212.

The results of these evaluations demonstrate that compacted granular fill, which may be placed adjacent to Seismic Category I structures or beneath Seismic Category II structures, will provide acceptable resistance to liquefaction for the specified ground motions.

2.5.4.8.2 Liquefaction Resistance of Native Soils

Liquefaction resistance of native in-situ soils located outside the area of excavation and backfill has been evaluated using two methods:

- Method 1: Soil texture-based screening.
- Method 2: Empirical evaluation based on SPT blowcounts.

The approach and results for each method are described below.

2.5.4.8.2.1 Soil Texture-Based Screening

A soil texture-based screening assessment was performed based on Seed et al., 2003 (Reference 2.5.4-241), Bray and Sancio, 2006 (Reference 2.5.4-242), and Boulanger and Idriss, 2006 (Reference 2.5.4-243). Soil laboratory test results for moisture content, percent fines, and Atterberg limits, as presented in Table 2.5.4-208, were screened against the following criteria for this assessment:

- **Fines Fraction:** Soils with more than 15 to 35 percent fines (passing the No. 200 sieve) usually have sufficient fines to separate individual sand and gravel grains. Such material can exhibit cohesive or cohesionless behavior

depending on the characteristics of the fines (Reference 2.5.4-241). Where fines are either non-plastic or are low plasticity silts and/or silty clays, a potential for liquefaction can exist; therefore, further textural screening was performed for fine-grained materials. Granular soils (with less than 35 percent fines) were evaluated for liquefaction potential based on in-situ conditions (see Subsection 2.5.4.8.2.2).

- **Plasticity and Water Content (soils with significant fines content):** Soils with significant fines content can be screened for liquefaction potential based on the plasticity index (PI), liquid limit (LL), and water content (w) (Reference 2.5.4-241), as follows:
  - "Zone A" soils have PI < 12 and LL < 37, and are considered potentially liquefiable if w > 0.80 LL.
  - "Zone B" soils have PI < 20 and LL < 47 (and are not "Zone A"), and are considered potentially liquefiable if w > 0.85 LL.
  - "Zone C" soils have PI > 20 or LL > 47, and are not considered susceptible to liquefaction.

Of the 18 BPA-series borehole soil samples collected above top of rock with index properties presented in Table 2.5.4-208, 15 have fines content greater than 35 percent. This indicates that the HAR site soils are predominantly fine grained. Of the 15 fine-grained soil samples, two classify as "Zone C" and are not considered susceptible to liquefaction. The other 13 fine-grained soil samples classify as either "Zone B" or "Zone A", but the moisture content of each sample is approximately 0.5 LL or less. These results are significantly lower than the threshold of 0.8 LL that may indicate susceptibility to liquefaction.

Based on these data, none of the fine-grained HAR site native soils are considered susceptible to liquefaction. This is consistent with the origin of the HAR site soils, which are residual soils formed by in-place weathering of parent siltstone and sandstone. Residual soils such as these are not among the types of soils commonly considered susceptible to liquefaction as listed in Regulatory Guide 1.198.

If native fine-grained soils are stockpiled for use as cohesive backfill, and the soils are compacted to meet the requirements in Table 2.5.4-212, the moisture content of the compacted cohesive fill will be less than 0.8 LL. Therefore, compacted cohesive fill will not be susceptible to liquefaction.

2.5.4.8.2.2 Empirical Evaluation of SPT Blowcounts of Native Soils

An evaluation of the FS against liquefaction of native granular soils was also performed based on correlations between SPT blowcount and CRR, as presented in Youd et al., 2001 (Reference 2.5.4-240). The earthquake

characteristics used for this evaluation are the same as listed in Subsection 2.5.4.8.1.

Of the 369 SPT tests evaluated in residual soils among the HAR boreholes, only 11 indicate a low or intermediate FS against liquefaction (FS less than 1.4), and are also located below the approximate site grade elevation of 79.2 m (260 ft.). Most of these represent the shallowest sample collected in the borehole (upper few feet bgs), and all of these samples were located above elevation 76.2 m (250 ft.). Based on these data, in-situ native granular soils are not liquefiable except potentially within a few feet of site grade.

2.5.4.8.3 Recommendations for Nonsafety-Related Backfill and Adjacent Native Soils

The following construction guidelines apply to backfill adjacent to Seismic Category I structures and beneath Seismic Category II structures, and to native soils adjacent to these structures:

- Construction verification testing should be performed to ensure that compacted granular fill meets the criteria listed in Table 2.5.4-212 for relative compaction and for V_s.
- Potentially liquefiable in-situ native granular soils near site grade should be either improved or replaced with non-liquefiable soil adjacent to structure foundations and compacted granular fill.

Selection and testing of backfill sources will be performed prior to construction, as described in Subsection 2.5.4.5. This verification testing will ensure that the backfill materials satisfy the criteria listed in Table 2.5.4-212.

2.5.4.9 Earthquake Site Characteristics

HAR COL 2.5-2

The methods used to calculate site amplification at the GMRS elevation (top of competent layer) are presented in Subsection 2.5.2.5. Derivation of the GMRS for HAR 2 and HAR 3, as well as the FIRS of the nuclear islands and the Annex Buildings, are presented in Subsection 2.5.2.6. The development of the subsurface profile for site amplification calculations considered the differences in subsurface profiles at HAR 2 and HAR 3.

As summarized in Subsection 2.5.2.5, two subsurface profiles were analyzed to represent the variable conditions at HAR 3, based on differences in the shallow V_s profiles at Boreholes BPA-25, BPA-49, and BPA-50. These differences are associated with the dipping strata underneath HAR 3, as well as the variable extent of weathering associated with existing ground elevations. The site amplification functions for these two profiles (named HAR-3a and HAR-3b) were then enveloped to develop the GMRS spectra at HAR 3.

One profile is considered representative of subsurface conditions at HAR 2, because the differences in the V_s profiles below the GMRS elevation at Boreholes BPA-5, BPA-47, and BPA-48 are small. The differences at HAR 2 are smaller than at HAR 3 in part because the dip of bedding planes is shallower and the depth extent of weathering is more consistent than at HAR 3.

The horizontal and vertical GMRS at HAR 2 and HAR 3 are presented on Figures 2.5.2-306 and 2.5.2-307.

2.5.4.10 Static and Dynamic Stability

HAR COL 2.5-10 Evaluations of the static and dynamic stability of the HAR 2 and HAR 3 nuclear islands were performed for foundation bearing capacity, sliding, foundation settlement, and lateral pressures against below-grade walls. These evaluations are presented in Subsection 2.5.4.10.1, 2.5.4.10.2, 2.5.4.10.3, and 2.5.4.10.4, respectively. Sound rock is present at the HAR 2 and HAR 3 nuclear island subgrade elevation of 67.1 m (220 ft.). Isolated very weak rock and soil like intervals below the top of sound rock have been conservatively modeled in these evaluations. The source and derivation of the rock engineering properties used in these evaluations are described in Subsection 2.5.4.2.

2.5.4.10.1 Bearing Capacity

The allowable bearing pressures at the HAR nuclear island subgrades under static and dynamic loading conditions have been evaluated, as presented in this subsection. The resulting allowable bearing pressures exceed the bearing pressure requirements for the AP1000 nuclear islands, as listed in the DCD, and therefore satisfy safety requirements. Conservative methods and rock mass strength parameters were used in these analyses, and appropriate FS values for static and dynamic loading conditions were considered, as summarized in Subsection 2.5.4.10.1.3.

2.5.4.10.1.1 Bearing Capacity Analysis Methodology

The ultimate bearing capacities of the rock mass at the HAR 2 and HAR 3 nuclear island basemat subgrades was calculated for static and dynamic loading using multiple methods:

- The primary method was based on classic bearing capacity equations adopted for rock foundations (Reference 2.5.4-231).

- The second method involved the use of two simple empirical methods as a secondary check of the primary method results (Reference 2.5.4-232, Reference 2.5.4-233).

- The third method involved the use of the finite element modeling software package Plaxis as an alternate method to evaluate foundation stability based on rock mass strength conditions at HAR 2 and HAR 3, and to conservatively evaluate potential effects of thin soil intervals within rock at Borehole BPA-6 near the HAR 2 nuclear island.

- The final method involved an evaluation of dynamic bearing capacity provided by the specific oriented rock discontinuity sets at HAR 2 and HAR 3. This evaluation was based on the force equilibrium of rock and soil wedges under and adjacent to the Plant West sides of the nuclear islands.

Details of these methods are described in the following Subsections 2.5.4.10.1.1.1, 2.5.4.10.1.1.2, 2.5.4.10.1.1.3, and 2.5.4.10.1.1.4.

2.5.4.10.1.1.1 General Bearing Capacity Equation

The general bearing capacity equation is commonly used to calculate the ultimate bearing capacity under both static and dynamic loading conditions, as shown in Equation 2.5.4-1 (Reference 2.5.4-231):

$$q_{ult} = c(N_c C_c) + 0.5\gamma' B(N_\gamma C_\gamma) + \gamma' D N_q \qquad (2.5.4\text{-}1)$$

In Equation 2.5.4-1, q_{ult} is the ultimate bearing capacity, c is the cohesion, $\gamma'D$ is the effective surcharge pressure at the foundation depth, γ' is the effective unit weight of foundation media, and B is the foundation width in the least direction. For foundations located below the groundwater elevation, such as occurs at the HAR sites, the effective unit weight of the foundation media is used. C_c and C_γ are shape correction factors to account for a rectangular footing. The factors N_c, N_q, and N_γ in the bearing capacity equation are based on empirical relationships with the foundation media strength properties (friction angle, ϕ), as follows:

$$N_c = 2N_\phi^{\,0.5}(N_\phi + 1) \qquad (2.5.4\text{-}2)$$

$$N_\gamma = N_\phi^{\,0.5}(N_\phi^{\,2} - 1) \qquad (2.5.4\text{-}3)$$

$$N_q = N_\phi^{\,2} \qquad (2.5.4\text{-}4)$$

$$N_\phi = \tan^2\left(45 + \phi/2\right) \qquad (2.5.4\text{-}5)$$

The local shear failure condition was also analyzed. For this condition, q_{ult} is calculated as in Equation 2.5.4-1, except that the third term "$\gamma'DN_q$" is excluded, as follows:

$$q_{ult} = c(N_c C_c) + 0.5\gamma' B(N_\gamma C_\gamma) \qquad (2.5.4\text{-}6)$$

By definition, ultimate bearing capacity calculated using the local shear failure condition is less than using the general condition, and is considered the controlling value of q_{ult}.

Use of the bearing capacity equations required determination of an apparent cohesion and effective friction angle for the rock mass. The strength of the rock mass was defined as a function of the intact rock strength and condition of discontinuities using the Hoek-Brown method (Reference 2.5.4-225). In this method, a set of strength parameters (c and ϕ) was calculated to represent the rock mass based on the intact rock UCS and condition of discontinuities, as represented by the geologic strength index (GSI). Conservative values of UCS and GSI were used for rock beneath the nuclear islands in this calculation, as described in Subsection 2.5.4.10.1.2.

Equation 2.5.4-6 and the Hoek-Brown strength parameters were used to calculate the ultimate bearing capacities under static and dynamic loads, as follows:

- The static ultimate bearing capacity was calculated using an equivalent foundation width B, representative of the width of the nuclear island basemat. Equivalent width B was reduced to approximately 37.8 m (124 ft.), from the maximum width of approximately 48.8 m (160 ft.) in the Plant east-west direction through the containment structure, to account for the variable Plant east-west width of the nuclear island. The reduced width represented a weighed average of the variable width of the nuclear island.

- The method for calculating dynamic bearing capacity considered the overturning moment that would occur from the inertial response of the nuclear island during seismic loading, in addition to the vertical gravity load from the structure. The overturning moment was represented by an equivalent transient bearing pressure subgrade loading profile, as presented in Figure 2.4-2 of report APP-GW-GLR-044 (Reference 2.5.4-234). The critical loading profile has a peak load near the edge of the nuclear island basemat, which decreases rapidly with distance from the edge of the basemat. This loading profile was used in the dynamic bearing capacity analyses as follows:

    1. The asymmetrical load profile was converted to an equivalent vertical load and moment, and a corresponding load eccentricity (e) was calculated.

    2. The reduced effective foundation width (B' = B-2e) was calculated, based on the eccentricity.

    3. The local bearing capacity equation (Equation 2.5.4-6) was then used to calculate the ultimate bearing capacity under static loading using the effective width, B', rather than the total width of the island (Reference 2.5.4-231, Reference 2.5.4-235).

The allowable bearing pressure (P) was determined from the ultimate bearing capacity of the rock (q_{ult}) by dividing the ultimate bearing capacity by an appropriate FS, as in Equation 2.5.4-7:

$$P = q_{ult}/FS \qquad (2.5.4\text{-}7)$$

Appropriate FS under static and dynamic loading are discussed in Subsection 2.5.4.10.1.3.

2.5.4.10.1.1.2 Empirical Methods

Two alternate empirical methods were used to confirm the results from the general bearing capacity method:

1. The first alternate method was presented in American Association of State Highway and Transportation Officials (AASHTO) (Reference 2.5.4-233). This method estimates the bearing capacity of rock by modifying the UCS of intact rock using a reduction factor, which is correlated to the RQD of the rock mass. The most conservative scenario of assuming jointed or broken rock was considered in this calculation.

2. The second alternative was based on the Hoek-Brown rock strength criterion (Reference 2.5.4-232). The main inputs were the UCS of intact rock and the Hoek-Brown strength criterion constants, which were determined from the type and quality of rock.

Like the general bearing bearing capacity evaluations, this approach considers the mass properties for the rock without specific reference to the location and orientation of discontinuities.

2.5.4.10.1.1.3 Two-Dimensional Finite Element Modeling

As a supplemental method, two-dimensional (2-D) finite-element modeling using the software package Plaxis version 8.5 was performed as an alternate calculation of the static and dynamic (seismic) bearing capacity of the HAR 2 and HAR 3 nuclear islands. The modeling was also performed to conservatively evaluate the potential effects of thin soil intervals within rock at Borehole BPA-6 near the HAR 2 nuclear island. Two-dimensional models cut in the north-south direction (“Plant West” to “Plant East”) at the HAR 2 and HAR 3 nuclear islands were developed. The resulting section represented a unit width strip through the structure.

In the 2-D models, the spatial variations of the subsurface stratigraphy, material properties, and pore pressure distribution, as well as the approximate excavation extents and backfill material properties, were considered. The stiffness of the below-grade structures (for example, the basemat and wall of the nuclear islands) was modeled using the elastic property and thickness of these

structures. Because the above-grade structures were not considered in the 2-D models discussed herein, relatively rigid fixed-end anchors were used to laterally brace the top of the walls.

The following three cases were modeled to evaluate a range of subsurface conditions:

- **Case 1: HAR 2 Lower-Bound Properties**. The lower-bound material parameters of the rock below HAR 2 were used in this model, as described in Subsection 2.5.4.10.1.2.

- **Case 2: HAR 3 Lower-Bound Properties**. The lower-bound material parameters of the layered shallow and deep rock below HAR 3 were used in this model.

- **Case 3: Potential Influence of Borehole BPA-6 Conditions**. In this model, the HAR 2 lower-bound rock properties were assigned, except for inclusion of a 1.5-m- (5-ft.-) thick soil seam where it was encountered at Borehole BPA-6. This seam was assumed to dip to the south (“Plant East”) towards the HAR 2 nuclear island, passing just under the bottom depth of Borehole BPA-7 and continuing along dip to under HAR 2. Two different strength values were considered for this seam (ϕ' of 33 degrees and 10 degrees) in different model runs to evaluate the sensitivity of bearing capacity to this seam.

To model static loading, a uniform bearing pressure of 0.43 MPa (8900 psf) was applied. To model dynamic conditions, the variable bearing pressure distribution shown on Figure 2.4-2 of report APP-GW-GLR-044 (Reference 2.5.4-234) was applied as a pseudo-static loading. The phi/c reduction procedure in Plaxis was utilized to calculate the FS under static and seismic loading conditions.

2.5.4.10.1.1.4 Oriented Rock Discontinuity Model

As a supplemental method, the FS against dynamic bearing loads along the Plant West sides of HAR 2 and HAR 3 were evaluated based on the specific orientations and engineering properties of the rock discontinuity sets present at the HAR sites. These analyses were performed to show that structure loads would not result in failure along discontinuities located beneath and adjacent to the nuclear island, as a supplement to the global bearing capacity analyses. The Plant West side was evaluated in this method because the peak site parameter for dynamic bearing demand (35 ksf) occurs at this location.

Bedding-oriented discontinuities (denoted discontinuity Set A herein) and high-angle joints (denoted discontinuity Set B herein) are oriented as summarized in Table 2.5.4-202. For these evaluations, the upper-bound estimate of the dip angle of bedding discontinuity Set A at HAR 3, based on the acoustic televiewer surveys (23.2 degrees from horizontal), was used to provide a conservatively low estimate of bearing capacity. In addition, approximately vertical joint sets are present as summarized in Subsection 2.5.1 (denoted

discontinuity Sets C and D herein), which strike roughly parallel to the HAR Plant North-South and East-West orthogonal axes, respectively. These discontinuity sets represent oriented planes of weakness relative to the strength of intact rock. Interface strength properties that represent these discontinuity sets are summarized in Subsection 2.5.4.10.1.2.2.

In this supplemental assessment of bearing stability, the force equilibrium of two rock and soil wedges under applied peak seismic bearing and lateral loads was calculated. The wedges were defined as follows:

- Wedge I represents rock underneath the Plant West side of the nuclear island, extending from the Plant West edge of the nuclear island to a distance B1 from the Plant West edge under the nuclear island. The base of Wedge I is defined by discontinuity Set B, which dips down in the general Plant West direction. The Plant West side of Wedge I is vertical, located at the Plant West side of the nuclear island.

- Wedge II represents rock and soil adjacent and Plant West of Wedge I. This wedge resists lateral loads from Wedge I. The base of Wedge II is defined by discontinuity Set A, which dips down in the general Plant East direction. The Plant East edge of Wedge II is vertical, adjacent to Wedge I.

- The sidewalls of both wedges, and the interface between Wedge I and II, are represented by discontinuity Sets C and D. In this way, Wedges I and II are defined as rectangular prisms, which are each rectangular in plan view and triangular in profile. Wedges I and II are similar in orientation to Wedges 1 and 3 considered in the sliding evaluation described in Subsection 2.5.4.10.2, and as shown in Figure 2.5.4-219. Both the true dips angles of discontinuity Sets A and B, and the apparent dips of these sets in the Plant East-West directions, were evaluated as separate cases for HAR 2 and HAR 3.

Forces acting on Wedges I and II were modeled as follows:

- The peak vertical load acting under the foundation over Wedge I (σ_{peak}) was assigned, and the corresponding average pressure acting over Wedge I within distance B_1 was calculated based on the dynamic bearing pressure distribution shown on Figure 2.4-2 of report APP-GW-GLR-044.

- A lateral inertial structure force was applied at the top of Wedge I. This force is proportional to the total nuclear island inertial sliding force under seismic loads ($F_{lat.struct}$) as defined in Subsection 2.5.4.10.2, multiplied by the fraction of the total nuclear island weight applied with distance $B_{1,}$ which is a function of σ_{peak} described above.

- Horizontal and vertical accelerations were also modeled for Wedges I and II rock and soil. These accelerations were conservatively assigned as the GMRS PGA values for HAR 3, as defined in Subsection 2.5.2. The directions of the horizontal and vertical rock and soil accelerations were specified in the

direction that resulted in a conservative estimate of bearing capacity (Plant West direction and upwards, respectively).

The trial value of σ_{peak} was iterated until wedge force equilibrium was obtained for the corresponding trial value of B_1. The corresponding FS_b was calculated as σ_{peak} / σ_{demand} of 35 ksf. FS_b values were calculated for various distances B_1, until the critical distance B_1 corresponding to the lowest overall FS_b was determined.

Four cases were evaluated, one each based on the true dip and apparent dip values at HAR 2 and HAR 3.

This approach required conservative simplifications, resulting in low estimates of the resulting FS_b. For example, the strike of the bedding features (discontinuity Set A) and high-angle joints (discontinuity Set B) are oriented at a skew of approximately 20 to 40 degrees from the nuclear island orthogonal axes and to the vertical joint Sets C and D. Based on this true configuration, as load is applied down- or up-dip along these dipping discontinuities, a high normal force will develop along at least one vertical joint interface (Sets C or D) that forms a “sidewall” of the loaded wedge. Due to the very tight nature of Sets C and D and high interface friction angle (see Subsection 2.5.4.10.1.2.2), a corresponding high frictional resistance will develop along this discontinuity. In the two-dimensional model, no such normal force increase is considered to develop on the vertical sidewalls – they are conservatively modeled as parallel to the direction of seismic loading (perpendicular to strike Sets A and B), and only at-rest normal forces are considered for the friction resistance along the vertical wedge sidewalls.

A secondary method by Landanyi and Roy, 1971 as presented in Wyllie 1999 (Reference 2.5.4-232), was also used to evaluate bearing capacity, as an order-of-magnitude confirmation of the oriented rock discontinuity evaluations described above.

2.5.4.10.1.2 Rock Mass and Rock Discontinuity Strength Parameters for Bearing Capacity Analyses

This subsection defines the rock mass strength parameters used in the general and finite element bearing capacity evaluations described in Subsections 2.5.4.10.1.1.1, 2.5.4.10.1.1.2, and 2.5.4.10.1.1.3, and the rock discontinuity strength parameters used in the oriented rock discontinuity bearing capacity evaluations described in Subsection 2.5.4.10.1.1.4.

2.5.4.10.1.2.1 Rock Mass Strength Parameters

Rock mass strength parameters were used to represent the global characteristics of the rock mass. In this approach the strength and quality of the rock is considered by defining reduced strength parameters for the rock mass rather than explicitly modeling strength along discontinuities. The rock mass strength parameters were selected for the bearing capacity evaluations described in Subsections 2.5.4.10.1.1.1, 2.5.4.10.1.1.2, and 2.5.4.10.1.1.3 using the following steps:

1. The mean and standard deviation of rock UCS were calculated from the laboratory UCS test results under and near each nuclear island. These test results are for samples collected within one foundation width B of the nuclear island basemat elevation. Table 2.5.4-206 summarizes the results.

2. The GSI was calculated for each rock core run at boreholes under and near each nuclear island. Statistics on the GSI (mean and standard deviation) for rock under each nuclear island subgrade elevation were then calculated to develop mean and lower-bound values, as summarized in Table 2.5.4-213.

3. "Mean" Hoek-Brown strength parameters were calculated for the rock mass (equivalent c and ϕ) using the average values of rock UCS and GSI for rock below the nuclear island subgrade elevations of 67.1 m (220 ft.) at HAR 2 and HAR 3.

4. "Lower-Bound" Hoek-Brown strength parameters were then calculated for the rock mass. Lower-bound values of rock UCS and GSI were selected as one standard deviation below the mean for rock below elevation 67.1 m (220 ft.). Subsection 2.5.4.10.1.3 describes the rationale for use of this criterion to select "lower-bound" values for analyses.

5. "Worst-Case" Hoek-Brown strength parameters were also calculated using the lowest UCS laboratory test result from any intact rock sample collected from below elevation 67.1 m (220 ft.), along with the lower-bound GSI. The lowest UCS test result is 9.57 MPa (1388 psi), encountered at BPA-6 at HAR 2.

Table 2.5.4-213 summarizes the various values of rock UCS and GSI and resulting Hoek-Brown strength parameters used for the bearing capacity analyses. The following trends in rock mass quality and strength are indicated by Table 2.5.4-213:

- The lower-bound UCS within one foundation width below the nuclear island under HAR 2 is higher than at HAR 3.

- Rock at HAR 3 exhibits more strength variation with depth within one foundation width below the nuclear island than at HAR 2. This is related to the larger depth of soil and underlying weathered rock at HAR 3.

The subsurface profile under the nuclear island at HAR 3 was divided into two elevation ranges for bearing capacity analyses — a shallow layer between elevation 51.8 and 67.1 m (170 and 220 ft.) and a deeper layer below elevation 51.8 m (170 ft.). The GSI and UCS results at HAR 2 were more consistent with depth below the nuclear island than at HAR 3; therefore, one rock layer with lower-bound strength properties was modeled for HAR 2.

2.5.4.10.1.2.2 Rock Discontinuity Strength Parameters

The following rock discontinuity strength parameters were determined for discontinuity Sets A through D, and were applied in the bearing capacity evaluations described in Subsection 2.5.4.10.1.1.4:

- **Discontinuity Set A (bedding features):** Undrained shear strength (S_u) = 0.072 MPa (1500 psf) above the nuclear island subgrade elevation, and S_u = 0.105 MPa (2200 psf) below the subgrade elevation. This is based on the strength of bedding-related clay seams present within otherwise sound rock, which have similar index properties as soil samples collected above rock as summarized in Table 2.5.4-208. These S_u values are significantly less than average S_u from UU triaxial tests performed on soil samples collected near the ground surface with similar index properties. Use of this value conservatively assumes that the clay seams are fully continuous and planar within the extents of the rock wedges considered in the evaluations.

- **Discontinuity Set B (high-angle joints):** Interface friction angle (ϕ_r) = 45 degrees below top of sound rock, with no cohesion. This is based on the typically rough and tight conditions of high-angle discontinuities observed in the HAR rock cores, and observations presented in the HNP foundation conditions report (Reference 2.5.4-244). At the HNP, the high-angle and vertical joints were observed to be tight and to often undulate a meter or more (several feet) from planar over the mapping extent with significant asperities. The joints were also reported to be vertically discontinuous, often initiating and terminating at parallel bedding planes a few meters (several feet) apart.

  The value of ϕ_r was calculated as follows (Reference 2.5.4-232):

$$\phi_r = \phi_b + JRC \log_{10} \frac{JCS}{\sigma'} \qquad (2.5.4\text{-}8)$$

  Where ϕ_b is the base friction angle for a planar rock surface (27 to 34 degrees for siltstone), JRC is the joint roughness coefficient (20 for rough tensional joints), JCS is the compressive strength of rock at the joint surface, and σ' is the applied normal stress. For the HAR site conditions and observed

high-angle joint conditions, the resulting ϕ_r is calculated to be greater than 60 degrees. Use of ϕ_r = 45 degrees without cohesion is therefore considered conservative to characterize such joints. For discontinuities above the top of sound rock, a lower ϕ_r = 30 degrees was conservatively applied, which is the approximate base friction angle without asperities (ϕ_b).

- **Discontinuity Sets C and D (vertical joints):** The same properties applied to discontinuity Set B were assigned to Sets C and D, because the discontinuity conditions are similar.

2.5.4.10.1.3 Bearing Capacity Results and Design Criteria

The bearing capacity results corresponding to the four methods described in Subsections 2.5.4.10.1.1.1, 2.5.4.10.1.1.2, 2.5.4.10.1.1.3, and 2.5.4.10.1.1.4 are presented below. Design criteria, including minimum acceptable FS values against static and dynamic bearing loads, are then presented.

2.5.4.10.1.3.1 General Bearing Capacity Method Results

Table 2.5.4-214 presents the ultimate bearing capacities calculated using the general bearing capacity method described in Subsection 2.5.4.10.1.1.1. The resulting FS based on the ratio of the ultimate bearing capacities to design static bearing demand of 0.43 MPa (8900 psf) and peak dynamic bearing demand of 1.68 MPa (35,000 psf) are also presented for each result.

2.5.4.10.1.3.2 Empirical Method Results

Table 2.5.4-214 presents the ultimate bearing capacity results for the two alternative empirical methods described in Subsection 2.5.4.10.1.1.2. As shown, these are similar to and confirm the results from the general bearing capacity method.

2.5.4.10.1.3.3 Two-Dimensional Finite Element Modeling Results

The two-dimensional finite element analyses described in Subsection 2.5.4.10.1.1.3 each result in FS greater than the corresponding cases presented in Table 2.5.4-214. This is in part because the shear strength of the soil and rock materials above the basemat of the nuclear islands was ignored in the general bearing capacity method, while the finite-element models inherently considered the shear strength of the materials above the basemat. The finite element analyses also indicate that soil conditions postulated on the basis of a 1.5-m- (5-ft) thick soil seam present at Borehole BPA-6 do not affect foundation performance of the HAR 2 nuclear island. For these analyses the resulting FS was 4 or greater for each analyzed static and dynamic condition.

2.5.4.10.1.3.4 Oriented Rock Discontinuity Model Results

The evaluations based on the oriented rock discontinuity model described in Subsection 2.5.4.10.1.1.4 result in minimum FS_b against dynamic bearing loads of 5.4 at HAR 2, and 2.6 at HAR 3. These results are associated with the true dip values for discontinuity Sets A and B, including the conservatively high estimate of Set A true dip of 23.2 degrees at HAR 3; the resulting FS_b corresponding to the apparent dip values are greater than for the true dip values.

The results of the secondary method by Landanyi and Roy, 1971 as presented in Wyllie 1999 (Reference 2.5.4-232), also each result in FS_b greater than 2.0 for each case evaluated. These results provide an order-of-magnitude confirmation of the results from the oriented rock discontinuity evaluations presented above.

2.5.4.10.1.3.5 Bearing Capacity Design Criteria

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 2.5.4-237). A lower minimum FS of 1.5 against the peak seismic bearing loads has been applied for safety-related structures (e.g., Reference 2.5.4-245). As shown in Table 2.5.4-214 and described in the preceding subsections, the minimum FS of 3.0 (static) and 2.0 (dynamic) are satisfied by each of the presented cases for HAR 2 and HAR 3, including those based on the lower-bound rock mass strength parameters.

Allowable bearing pressures are calculated based on the ultimate bearing capacities calculated using the lower-bound strength parameters at HAR 2 and HAR 3, and using FS of 3.0 and 2.0 for static and dynamic loading, respectively.

Allowable bearing pressures are designated as follows:

- HAR 2 – Static Loading: 2.59 MPa (54 ksf)
- HAR 2 – Dynamic Loading: 3.26 MPa (68 ksf)
- HAR 3 – Static Loading: 1.39 MPa (29 ksf)
- HAR 3 – Dynamic Loading: 1.77 MPa (37 ksf)

The FS values in Table 2.5.4-214 and allowable bearing pressure values are considered very conservative for the following reasons:

- They are based on the lower-bound rock mass strength parameters based on rock UCS and GSI values corresponding to one-standard deviation below the mean. This is equivalent to the 16th percentile value – such that 84 percent of the rock has strength higher than this value. In addition, use of these lower-bound strength values provides very high confidence that the mean

values of UCS and GSI for rock under the nuclear island subgrades are greater than these values. Further, these strength values are considered highly conservative because only thin zones will be represented by these lower-bound rock mass strength parameters, and bearing capacity will be more closely represented by the average within the zone of influence beneath the foundation. As shown in Table 2.5.4-214, FS based on mean rock mass strength (best estimate) are significantly higher than those based on lower-bound strength values.

- Results of the finite element analyses in Subsection 2.5.4.10.1.3.3 indicate that the FS results from the general and empirical methods are conservative. The oriented rock discontinuity evaluations in Subsection 2.5.4.10.1.3.4 also result in FS greater than those obtained from the general bearing capacity method in Subsection 2.5.4.10.1.1.1 using the lower-bound rock mass strength properties.

- Each of the site-specific FS against dynamic bearing demand presented in this subsection is based on a peak dynamic bearing demand of 1.68 MPa (35 ksf), consistent with the AP1000 design parameter corresponding to the 0.3g SSE design event. However, this peak bearing pressure corresponds to hard-rock site conditions. For a firm-rock site (such as HAR), the peak bearing pressure is 1.34 MPa (27.9 ksf) for the 0.3g SSE design event. The HAR-specific bearing pressure will be significantly less than this lower value because the HAR GMRS and FIRS are bounded by the 0.3g SSE design spectra. For these reasons, the FS against dynamic bearing demand at HAR 2 and 3 are greater than those presented herein.

2.5.4.10.1.4 Annex Building and Turbine Building Bearing Capacities

The bearing capacities of the HAR 2 and HAR 3 Annex Buildings and the first bay of the Turbine Buildings (seismic Category II structures) have been evaluated based on static and dynamic loads provided by Westinghouse. For the Annex Buildings, the analyses were based on a static bearing pressure of 0.129 MPa (2700 psf) and a dynamic bearing pressure of approximately 0.152 MPa (3175 psf). For the first bay of the Turbine Buildings, the analyses were based on a static bearing pressure of 0.134 MPa (2800 psf) and a dynamic bearing pressure of approximately 0.579 MPa (12,100 psf). For both structures' building foundations (founded on compacted granular fill, concrete, or rock), the static FS are greater than 3.0 and the dynamic FS are greater than 2.0. Confirmation of final static and dynamic FS against allowable bearing pressures for the Annex and Turbine Buildings will be completed upon final determination of the Annex and Turbine Building bearing pressures.

2.5.4.10.2 Resistance to Sliding

HAR COL 2.5-6

The HAR 2 and HAR 3 nuclear islands will each be founded on sound rock. During excavation, loose material at the subgrade elevation will be removed, resulting in a relatively clean, exposed rock, as discussed in Subsection 2.5.4.5.2. The concrete mudmat, geomembrane, and nuclear island foundation will be constructed over the exposed rock. The mudmat and any underlying concrete fill will interlock with the rock subgrade and create a stronger bond than the overlying waterproofing membrane interface. As described in Subsection 2.5.4.5.3, concrete fill will also be placed within the space between the excavated rock and the nuclear island sidewall below elevation 69.3 m (227.5 ft.). These construction plans were used to evaluate sliding stability of the nuclear islands under peak seismic design loads. Subsection 2.5.4.10.2.1 provides a summary of the evaluation methodology, and Subsection 2.5.4.10.2.2 presents the results.

The weakest interface beneath the nuclear island basemat will be between the mudmat concrete and the geomembrane waterproofing layer. The design of the waterproof membrane beneath the nuclear island basemat is described in DCD Section 3.4.1.1.1.1. Per the DCD, the specific static coefficient of friction between horizontal membrane and concrete must be ≥ 0.55. An ITAAC will be performed to verify that the mudmat-waterproofing interface beneath the nuclear island basemat has a coefficient of friction to resist sliding of ≥ 0.55.

2.5.4.10.2.1 Sliding Evaluation Methodology

The sliding stability evaluations were based on the force equilibrium of five wedges representing soil and rock below and on the active and passive sides of the nuclear island. Figure 2.5.4-219 shows the configuration of the nuclear island and associated rock and soil wedges considered in the sliding stability evaluations. Conservative assumptions were applied to convert the three-dimensional nuclear island and subsurface configuration to an equivalent limited-width two-dimensional model. Two-dimensional profiles associated with each of the four orthogonal directions at HAR 2 and HAR 3 (total of eight profiles) were evaluated. The resulting FS against sliding in each direction were then compared to minimum required value (FS_s) of 1.1.

The orientations and conditions of rock discontinuity sets were conservatively modeled as described for the bearing capacity evaluations in Subsections 2.5.4.10.1.1.4 and 2.5.4.10.1.2.2. Driving forces included the inertial lateral and vertical forces of the nuclear island due to the SSE and the external lateral forces on the active side of the nuclear island. Dynamic forces were represented by equivalent static loads. These loads represented instantaneous peak forces computed on the basis of spectral accelerations from the FIRS and GMRS for the HAR 2 and HAR 3 site. In Figure 2.5.4-219, SSE seismic forces are directed to the left, with concentration of bearing load near the left side of the

nuclear island. Vertical forces associated with SSE loading were assumed to occur in the direction that results in the lowest FS against sliding.

Two sets of dipping discontinuities were considered in the model, denoted as Set 1 (apparent dip down in the direction of seismic loading) and Set 2 (apparent dip in opposite direction). The dip angles from horizontal (β_1 and β_2) and strength properties in rock (ϕ_{1r}, ϕ_{2r} and c_{1r}, c_{2r}) were assigned based on the orthogonal direction under consideration; Set 1 was assigned properties consistent with the discontinuity Set (A or B) which dips down in the modeled direction of SSE loading. Set 2 was assigned properties of either discontinuity Set B or A that were not assigned to Set 1. Depth to sound rock was modeled separately on either side of the nuclear island (D_{s1} and D_{s2}).

Five rock and soil wedges are considered in the model. Three of the wedges contribute net driving forces (Wedges 1, 4, and 5) that must be resisted for stability, and two provide net resisting forces (Wedges 2 and 3). For each wedge, driving and/or resisting forces are contributed by the wedge self-weight and lateral inertial force; whether these act as driving or resisting forces depends on the dip direction of the base of the wedge. Resisting forces are provided by friction and cohesion along the base and the sidewalls of each wedge. Groundwater table is conservatively modeled at site grade for all calculations.

Two separate methods are used to calculate the driving force on the active side of the nuclear island. The first method is based on wedge force equilibrium (the resulting force is $F_{lat5.wedge}$). The second is based on the force from compacted backfill, based on the elastic method presented in ASCE 4-98 (Reference 2.5.4-246) (the resulting force is $F_{lat5.backfill}$). This second method conservatively considers that the granular backfill is present from site grade to the nuclear island subgrade elevation, without reduction for the concrete fill below elevation 69.3 m (227.5 ft). The net lateral force on the active NI endwall (F_{active}) is assigned as the greater of the two forces $F_{lat5.wedge}$ or $F_{lat5.backfill}$.

Based on the input parameters, the stability of each wedge was calculated sequentially starting with Wedge 1. For Wedges 1, 4, and 5, the separate external lateral forces required for wedge stability (denoted F_{lat1}, F_{lat4}, and F_{active}, respectively) were calculated, which act as driving forces in the overall NI stability calculation. For Wedges 2 and 3, the additional external lateral forces that can be resisted while maintaining stability (denoted R_{lat2} and R_{lat3}, respectively) were calculated, which act as resisting forces in the overall stability calculation.

The peak inertial driving force from the nuclear island during seismic loading ($F_{lat.struct}$) was calculated based on a HAR site-specific SSI bounding study by WEC (Reference 2.5.4-247). The FS against sliding (FS_{slide}) was calculated directly as the ratio of the sum of lateral resisting forces to the sum of lateral driving forces, as follows:

$$FS_{slide} = (R_{passive} + R_{base}) / (F_{active} + F_{lat\_struct}) \qquad (2.5.4\text{-}9)$$

In this equation, $R_{passive}$ is the net passive resistance provided between Wedges 1 and 3 ($R_{lat3} – F_{lat1}$). R_{base} is the net resistance provided by Wedges 2 and 4 ($R_{lat2} – F_{lat4}$), or the available friction resistance at the waterproofing membrane interface, whichever is lower. F_{active} and F_{lat_struct} are as defined previously.

The resulting FS_{slide} was then compared to the minimum allowable sliding FS of 1.1 for evaluation of acceptable stability.

2.5.4.10.2.2 Sliding Evaluation Results

Table 2.5.4-218 presents the key input parameters used in the sliding stability evaluations, the net driving (lateral) forces $F_{lat.struct}$ and F_{active}, the net resisting forces R_{base} and $R_{passive}$, and the resulting FS against sliding (FS_{slide}) for each of the orthogonal directions of nuclear islands at HAR 2 and HAR 3.

For each orthogonal direction, multiple iterations were performed to identify the critical value of B_1 that results in the lowest FS_{slide}. Two results are presented for each NI orthogonal direction: (1) the first result corresponds to the distance B_1 that resulted in the lowest FS_{slide}; and (2) the second result corresponds to B_1 of zero, such that bearing pressures did not contribute to the lateral sliding forces. This second result represents a typical sliding evaluation decoupled from the bearing capacity evaluation.

As shown in Table 2.5.4-218, FS_{slide} is 1.75 or greater for each case evaluated; therefore, each of these results satisfies the minimum FS of 1.1 required against sliding with significant margin. The resulting FS_{slide} for each of the Plant North and Plant East profiles at HAR 2 and HAR 3 were very high, with FS_{slide} greater than 12 for each case. This is due in part to the steep dip angle and high friction angle of discontinuity Set B, which is oriented to resist sliding in the Plant North and Plant East directions.

Table 2.5.4-218 also lists the passive force required to provide a minimum FS_{slide} of 1.1 for each profile. As shown, for six of the eight profiles, no passive force is required. For the Plant West profiles at HAR 2 and HAR 3, only 17 and 22 percent of the available net passive resistances are needed to provide the minimum FS_{slide} of 1.1, respectively.

As indicated by the sliding evaluations, the concrete fill below elevation 69.3 m (227.5 ft.) will sufficiently transfer the required passive resistance to the adjacent rock to prevent sliding. Fill above elevation 69.3 m (227.5 ft.), consisting of concrete, compacted cohesive fill, or compacted granular fill, is not required to prevent sliding.

2.5.4.10.3 Settlement

HAR COL 2.5-12
HAR COL 2.5-16

The HAR nuclear islands will be founded on sound rock. As described in this subsection, elastic settlement of the rock under foundation loads is proportional to the elastic modulus of the rock mass. Isolated clay seams within sound rock have the potential to undergo recompression settlement under structure loads. As described in this subsection, the total settlements (elastic plus recompression settlement) and differential settlements estimated for the HAR 2 and HAR 3 nuclear islands are small and within tolerable limits.

2.5.4.10.3.1 Elastic Settlement under Foundation Loads

Elastic settlement of the rock mass under the nuclear islands was calculated based on the elastic properties of the rock and the distribution of foundation loads with depth. The following two separate methods were used to estimate the rock mass elastic modulus:

- In the first method, the V_s, V_p, and Poisson's ratio results from the suspension logging tests at six boreholes (BPA-5, BPA-47, and BPA-49 at HAR 2, and BPA-25, BPA-49, and BPA-50 at HAR 3) were used to calculate the small strain elastic and constrained moduli values at regular depth intervals. The small-strain constrained modulus was conservatively degraded by 50 percent to model larger strain effects.

- In the second method, the PMT results from multiple depths at four boreholes (BPA-3 and BPA-41 at HAR 2, and BPA-23 and BPA-43 at HAR 3) were used to estimate the constrained modulus profile for the rock.

The relationship between settlement of a rock interval, the stress increase, and the elastic modulus is based on simple elastic theory, as presented in Equation 2.5.4-10:

$$\Delta S = \sum_i H_i \Delta\sigma_i / M_i \qquad (2.5.4\text{-}10)$$

In Equation 2.5.4-10, 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 both modulus estimation methods are small, as listed in Table 2.5.4-215. By the first

method, elastic settlement was calculated as approximately one mm (0.03 to 0.04 in.) at HAR 2 and HAR 3. By the second method, the elastic settlement was calculated as approximately 2.5 mm (0.1 in.) at HAR 2 and approximately 5 mm (0.2 in.) at HAR 3. These settlements would occur as the nuclear island facilities are constructed. No additional elastic settlements would occur after construction, when foundation loading is constant.

Elastic settlements calculated by the first method are considered the "best estimates" of settlement. Results using the second method are included as conservative "upper bound" estimates of elastic settlement in Table 2.5.4-215.

2.5.4.10.3.2 Recompression Settlement

Isolated clay seams were observed within otherwise sound rock below the nuclear island basemat elevation in several boreholes, as discussed in Subsection 2.5.4.1.1. Intervals of clay within sound rock for consolidation testing were not collected because the clay seams typically were too thin, interbedded with very weak or weathered rock intervals, or disturbed by rock coring activities to produce reliable test results. While these clay seams are a possible source of additional settlement, the potential magnitude of recompression-related settlements is considered to be small for the following reasons:

- As shown in Table 2.5.4-208, index test data on samples from several of the clay seams encountered below the top of sound rock indicate that sample water content is less than the plastic limit for all samples (typically approximately half of the plastic limit). This provides evidence that these clay seams, where they occur, are overconsolidated (Reference 2.5.4-237). In this context, overconsolidation means that the clay seams were previously loaded under higher pressures than the current overburden stress state, as caused by erosion of overlying soil or rock.

- Likewise, consolidation tests performed on intact residual soil samples from near the ground surface indicate that site soils are overconsolidated and have low compressibility, as listed in Table 2.5.4-210. These samples have low moisture content and plastic limit similar to the clay samples collected from seams within rock, as listed in Table 2.5.4-208.

- Some of the observed clay seams were also likely created by disturbance during the coring process, and actually exist as weak rock seams in situ. This is supported by the relatively less frequent occurrence of observed clay seams within sound rock in the larger diameter HQ-size rock cores (BPA-5, BPA-47, and BPA-48 at HAR 2, and BPA-25, BPA-49, and BPA-50 at HAR 3) compared with the NQ-size coreholes, especially at HAR 2.

- Finally, for clay seams of limited lateral extent (i.e., not fully continuous beneath the nuclear islands), some proportion of vertical stress will redistribute around the seam, thereby producing relatively smaller settlements within the seam than for a laterally continuous seam. This is a

particularly important mechanism in geologic structures that are characterized by predominantly sound rock with isolated softer intervals.

Recompression settlement was calculated for representative boreholes at HAR 2 and HAR 3 based on the observed clay seam thicknesses and representative consolidation properties. Table 2.5.4-215 includes the results. These results are considered conservative estimates of recompression settlement for the reasons above.

2.5.4.10.3.3 Total Settlement

The total settlements at representative boreholes under the HAR 2 and HAR 3 nuclear island basemats was calculated as the sum of elastic and clay seam recompression settlements. Table 2.5.4-215 presents the results of these calculations.

Following are key observations regarding the total estimated nuclear island settlement:

- As listed in Table 2.5.4-215, estimated total settlements range from one to 13 mm (0.04 to 0.5 in.) at boreholes under the HAR 2 nuclear island, and from 3 to 13 mm (0.1 to 0.5 in.) under the HAR 3 nuclear island.

- Borehole BPA-7 is located at the north edge of the HAR 2 nuclear island. Based on conditions at this borehole, only up to 3 mm (0.1 in.) of total settlement is estimated at this location. Borehole BPA-6 is located approximately 21 m (70 ft.) north of BPA-7, outside the HAR 2 nuclear island footprint. As discussed in Subsection 2.5.4.1.3, anomalously thick silt and clay seams were encountered within otherwise sound rock at Borehole BPA-6. If such seams were present underneath the HAR 2 nuclear island, up to 30 mm (1.2 in.) of recompression settlement would be estimated based on the methods in this section. However, since such seams encountered in Borehole BPA-6 were not encountered in any of the boreholes under the HAR 2 nuclear Island, the range of total settlements presented in Table 2.5.4-215 are considered representative of potential settlement at HAR 2.

The potential consolidation settlements listed in Table 2.5.4-215 are considered conservatively high, for reasons presented in Subsection 2.5.4.10.3.2. In addition to the reasons presented therein, the settlement calculations are based on a net foundation load of 0.43 MPa (8900 psf) at basemat grade, which is also conservative. This assumes that the overburden pressures within clay seams are fully reduced to the pressure due to weight of soil and rock between the foundation subgrade (elevation 67.1 m [220 ft.]) and the underlying clay seam during excavation, and that the clay seams fully rebounds after excavation across the footprint of the nuclear island prior to foundation loading. In reality, overburden pressures in clay seams under the edges of the excavation will not

be fully relieved by excavation. Therefore, the assumption of a net nuclear island foundation load of 0.43 MPa (8900 psf) is conservatively high.

The total settlements listed in Table 2.5.4-215 are within the range of acceptable settlements for the AP1000 of up to 76 mm (3 in.). The HNP has not undergone any unacceptable settlements, which further supports the expectation of satisfactory foundation performance at HAR 2 and HAR 3.

2.5.4.10.3.4 Differential Settlement

The potential differential settlement across the nuclear island basemats is calculated as the difference in total settlement divided by the distance between representative pairs of boreholes. This calculation provides an equivalent slope of differential settlement, also referred to as the angular distortion. The resulting differential settlement slopes are listed in Table 2.5.4-216. As shown, the slope of differential settlement is expected to be less than 0.00083 (or 1/1200) based on conservative estimates of total settlement, which is within the acceptable range for the AP1000 under both HAR 2 and HAR 3.

The subgrades and foundations of structures adjacent to the nuclear islands will be constructed to account for differential settlement with the nuclear islands. For adjacent structures founded on rock or concrete fill over rock, the differential settlements within the nuclear islands are not expected to exceed 13 mm (0.5 in.). For adjacent structures founded on granular fill over rock, the differential settlements with the nuclear islands are not expected to exceed 76 mm (3 in.). These differential settlements are within the acceptable range for the AP1000 under both HAR 2 and HAR 3. These analyses were based on the foundation loads provided by Westinghouse. Once foundation bearing loads for structures adjacent to the nuclear island are finalized, a detailed analysis of differential settlements between the nuclear islands and adjacent structures will be re-assessed.

2.5.4.10.3.5 Rebound Potential

Rebound of the nuclear island subgrades is evaluated by considering the vertical change in effective stress due to removal of the soil and rock above the foundation subgrades along with the elastic properties of rock and soil at and below the foundation level. Excavation depths below existing ground surface at HAR 2 and HAR 3 will range from approximately 10.7 to 15 m (35 to 50 ft.), with associated reduction in effective stress of approximately 0.17 to 0.24 MPa (3500 to 5000 psf). This range spans roughly half of the net foundation pressure at the nuclear island subgrade of 0.43 MPa (8900 psf).

Upon excavation, the nuclear island subgrade is expected to rebound by an amount up to half of the calculated elastic and recompression settlement presented in Subsection 2.5.4.10.3, with corresponding rebound up to 6 mm (0.25 in.) at HAR 2 and HAR 3. These are the maximum values near the center of the excavations. Rebound at the excavation edges will likely be less, for the

same reasons described in Subsection 2.5.4.10.3.3 for recompression settlement.

2.5.4.10.3.6 Design Criteria for Foundation Settlement

The following design criteria are tolerable values for the AP1000 nuclear island:

- Total settlement under nuclear island: up to 76 mm (3 in.).
- Differential settlement across nuclear island: 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 Subsection 2.5.4.10.3.1, 2.5.4.10.3.2, and 2.5.4.10.3.3, the engineering analyses indicate that these design criteria will be satisfied at HAR 2 and HAR 3. Conservative methods of settlement analyses and design parameters were used, as described in those subsections.

2.5.4.10.3.7 Subsurface Instrumentation

HAR COL 2.5-6 Settlement of the nuclear islands will be monitored throughout construction. A detailed settlement monitoring program will be developed prior to construction.

As presented in 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 verify the expected settlements. Settlement monuments will likely be installed and monitored throughout the construction process, as follows:

- Settlement benchmarks will be installed within the subgrade mudmat (at approximate elevation 67.1 m [220 ft.]) 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 benchmarks will be installed approximately 1 meter (a few feet) above site grade (at approximate elevation 80.2 m [263 ft.]) connected to the sidewalls of the nuclear island, directly above the deeper benchmark locations described previously. These benchmarks will be monitored during backfilling operations and, periodically, during and after construction of the nuclear island structures.

Monitoring will be continued until at least 90 percent of expected settlement has occurred. This will be evaluated by review of the settlement versus time curves at the benchmark locations.

2.5.4.10.4 Lateral Earth Pressures

HAR COL 2.5-7

HAR COL 2.5-11

Lateral earth pressures will develop against below-grade nuclear island sidewalls due to placement and compaction of soil backfill materials. At-rest earth pressures will act on the sidewalls after construction and are considered appropriate values for wall design during static loading, and are presented in Subsection 2.5.4.10.4.1. The maximum forces that could develop along the active nuclear island sidewalls during dynamic loading, and the passive forces available to resist sliding, are presented in Subsection 2.5.4.10.4.2.

2.5.4.10.4.1 At-Rest Lateral Earth Pressures

Equations for calculation of lateral earth pressures are presented in this subsection. The compacted granular and compacted cohesive backfill types described in Subsection 2.5.4.5.3 are considered in the calculation of lateral earth pressures, with the following effective stress parameters:

- Compacted granular backfill: ϕ' = 35 degrees, c = 0 psf
- Compacted cohesive backfill: ϕ' = 20 degrees, c = 400 psf

It was assumed that backfill adjacent to the nuclear island sidewalls will be compacted as described in Subsection 2.5.4.5.3 for calculation of lateral earth pressures. In addition, light, hand-operated compaction equipment will be used to compact the soil adjacent to the nuclear island sidewalls. This will minimize compaction-induced soil stresses against the sidewalls, rendering them small and insignificant. The nuclear island sidewalls will not yield to the lateral earth pressures; therefore, the at-rest pressure condition is appropriate for use in wall design under static loads.

The at-rest earth pressure coefficient (K_o) is calculated as follows (Reference 2.5.4-238):

$$K_o = 1-\sin(\phi')\ OCR^{\sin(\phi')} \qquad (2.5.4\text{-}11)$$

where OCR is the overconsolidation ratio, which is considered to be 1.0 for fill compacted with hand-guided equipment adjacent to the sidewalls.

The at-rest pressure, $P_{(at\text{-}rest)}$, against the nuclear island sidewalls at any depth z bgs can be calculated as follows:

$$P_{(at\text{-}rest)} = \sigma'_v{*}K_o + P_h + S_s{*}K_o \qquad (2.5.4\text{-}12)$$

where σ'_v is the effective overburden pressure at the depth z, P_h is the groundwater pressure, and S_s is the surface surcharge due to surface loads, such as adjacent buildings, and other terms are as defined previously.

Hydrostatic pressures (P_h) have been considered in the calculation of lateral at-rest pressures. A groundwater elevation of 78.6 m (258 ft.), the maximum estimated post-construction groundwater elevation at the HAR sites, was conservatively considered in the calculation of hydrostatic pressures. Hydrostatic pressures will act simultaneously on all sides of the nuclear islands.

The resulting at-rest lateral earth pressure profiles for the two soil backfill types are presented for representative sidewall elevations in Table 2.5.4-217. The actual depth of soil backfill against the nuclear island sidewalls will vary by location. Concrete backfill will be placed between the nuclear island sidewalls and rock excavation slopes below elevation 69.3 m (227.5 ft.) around the perimeter of each nuclear island and at select locations above this elevation, as discussed in Subsection 2.5.4.5.3. Once the concrete fill solidifies, the at-rest pressure from the concrete fill will be less than the at-rest pressure from compacted backfill, such that the at-rest pressures listed in Table 2.5.4-217 are considered a conservative envelope of the at-rest lateral earth pressures. The pressures presented in Table 2.5.4-217 include hydrostatic pressures, but do not include lateral pressures due to surface surcharge loads. Adjacent structures, where present, will increase the at-rest pressures from the values presented in Table 2.5.4-217 per Equation 2.5.4-12.

2.5.4.10.4.2 Dynamic Lateral Active and Passive Forces

The peak lateral forces acting on the active sides of the nuclear islands during seismic loading (F_{active}) were calculated as part of the nuclear island sliding evaluations described in Subsection 2.5.4.10.2.1. The values of F_{active} on each side of the HAR 2 and HAR 3 nuclear islands are presented in Table 2.5.4-218. These maximum forces encompass the at-rest plus dynamic earth pressures and the at-rest plus dynamic contributions from adjacent building surcharge loads.

The peak passive forces available to resist sliding during seismic loading ($R_{passive}$) are presented in Table 2.5.4-218 for each side of the HAR 2 and HAR 3 nuclear islands. The total fraction of this available passive resistance that is required to resist sliding with a minimum FS of 1.1 is also presented for each side of the nuclear islands in Table 2.5.4-218. As shown, at most only 22 percent of the net available passive force would potentially be required to prevent sliding. As discussed in Subsection 2.5.4.10.2.2, this maximum passive force will be fully transferred from the nuclear island sidewalls to the adjacent sound rock by the concrete fill placed below elevation 69.3 m (227.5 ft).

Hydrostatic forces are not included in the values of F_{active} or $R_{passive}$ presented in Table 2.5.4-218. This is because hydrostatic forces balance on opposite sides of

the nuclear island and therefore do not contribute a net driving or resisting force to sliding.

2.5.4.11 Design Criteria

HAR COL 2.5-6

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.

Table 2.0-201 compares the DCD site geotechnical parameter criteria with the corresponding site characteristics at HAR 2 and HAR 3, 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.

HAR 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 Subsection 2.5.4.2.1.1.1 and 2.5.4.2.1.1.2, respectively.
- Criteria for selection of rock core and soil samples for laboratory testing are presented in Subsection 2.5.4.2.1.6.2 and 2.5.4.2.1.6.3, respectively.
- Criteria for selection of rock and soil properties used in engineering analyses are presented in Subsection 2.5.4.2.4.
- Criteria for selection of geophysical survey results as design parameters are presented in 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 Subsection 2.5.4.5.2.

- Criteria for groundwater elevations are presented in Subsection 2.5.4.6.1. Selection of construction dewatering methods is presented in 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 Subsection 2.5.4.10.1. Selection of static and dynamic factors of safety are presented in 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 Subsection 2.5.4.10.3. Tolerable settlement limits are presented in Subsection 2.5.4.10.3.6.
- Criteria for estimation of nuclear island sidewall lateral earth pressures are presented in Subsection 2.5.4.10.4.

Each software package used for engineering analyses supporting design and evaluation of safety-related structures was validated and verified to operate properly on the computer used for the analyses, in accordance with the CH2M HILL quality assurance project plan (QAPP). Specific software packages used for these analyses are described in the design criteria subsections referenced above.

2.5.4.12 Techniques to Improve Subsurface Conditions

HAR COL 2.5-7

Sound rock is present at the nuclear island subgrade elevation, which is capable of supporting the structures with minor surface repairs to address local defects as necessary. Criteria for acceptable subgrade conditions are presented in Subsection 2.5.4.5.2.

Rock that does not satisfy the criteria at the nuclear island subgrade elevation will be removed or improved. A detailed excavation, subgrade improvement, and verification program will be developed prior to construction. Subgrade improvement methods anticipated to be included in this program are summarized in Subsection 2.5.4.5.2.

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 1 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-1	686804.8	2012129.3	261.27	82.7	12.8	CL and SC	NA	12.8	Slightly weathered to fresh below.	Siltstone
BPA-2	686894.0	2012134.4	259.73	100.0	17.7	ML, CL, and SM	Alternating siltstone and sandstone	31.0	Highly weathered, very weak rock with clay seams to depth noted. Fresh below.	Siltstone
BPA-3	686836.6	2012199.0	260.56	159.2	2.6	GM, CL	Alternating siltstone and sandstone	4.2	Highly weathered from 12.5 to 15.4 ft. bgs, with clay seams. Sound above and below this range.	Alternating siltstone and sandstone
BPA-4	686735.0	2012207.2	259.63	100.0	2.6	ML, CL	Siltstone	5.0	Moderately weathered from 10.8 to 15.0 ft. bgs, with clay seams. Sound above and below this range.	Siltstone to 19.5 ft. and sandstone to 27.9 ft.
BPA-5	686869.4	2012272.8	265.74	207.0	5.3	CL, SM	NA	7.5	Fresh below.	Alternating siltstone, sandstone and conglomerate

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 2 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-6	687037.2	2012258.8	264.26	110.0	2.8	CL, ML	Siltstone to 25 ft., sandstone	29.3	Highly weathered with clay seams from 12.0 to 22.2 ft. bgs. Sound above and below this range.	Alternating sandstone and siltstone
BPA-7	686973.9	2012288.8	264.98	145.0	10.1	SM, SP, ML	NA	10.4	Fresh below.	Alternating siltstone and sandstone
BPA-8	686839.1	2012351.4	267.44	147.5	2.6	CL	Siltstone	14.2	Intervals of strong rock start at 3.9 ft. bgs, but interbedded with weathered rock and clay seams to depth noted.	Alternating sandstone and siltstone
BPA-9	686777.5	2012379.1	266.83	87.3	3.2	CL	Siltstone	12.2	Highly weathered with clay seams from 17.55 to 19.45 ft. bgs. Sound above and below this range.	Alternating sandstone and siltstone
BPA-10	686920.4	2012383.0	265.94	87.0	7.6	ML	Sandstone and siltstone with clay seams alternating	12.5	Fresh below.	Alternating sandstone and siltstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 3 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-11	687016.5	2012396.5	264.31	159.2	7.7	SP-SM, ML, GC, CL	Siltstone	22.0	Slightly weathered to fresh below.	Siltstone to 29 ft., conglomerate to 30.8 ft., over siltstone
BPA-12	686970.3	2012488.0	263.94	87.0	12.6	CL	NA	12.6	Moderately weathered from 15.8 to 17.0 ft. Sound above and below this range.	Siltstone to 17 ft., over sandstone
BPA-13	686879.4	2012520.3	263.74	164.1	10.2	CL	Siltstone	19.1	Fresh below.	Siltstone, sandstone 23.3-28 ft.
BPA-14	686783.7	2012564.5	263.91	100.0	3.8	CL	Siltstone and clay seams alternating	17.6	Highly weathered with clay seams from 22.35 to 23.1 ft. bgs. Sound above and below this range.	Siltstone
BPA-15	687089.1	2012553.1	262.67	82.3	12.7	CL	CL	13.4	Fresh below.	Siltstone
BPA-16	687020.2	2012592.5	262.79	163.3	3.2	CL	Siltstone	17.7	Fresh below.	Siltstone to 27.6 ft., sandstone to 33 ft., over siltstone
BPA-17	686934.2	2012635.6	262.36	45.0	5.1	CL	Siltstone	35.0	Fresh below.	Siltstone to 40 ft., sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 4 of 15) Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-18	687144.9	2012671.5	261.85	45.0	10.3	ML, SM, CL	Siltstone	25.0	Slightly weathered and fresh below.	Siltstone
BPA-19	687070.7	2012705.5	261.36	82.0	12.7	CL	Siltstone and sandstone alternating	34.0	Fresh at 21.8-24.3 ft. and slightly weathered to fresh below depth noted.	Siltstone
BPA-20	686985.3	2012742.1	261.48	41.2	8.2	CL, SC	Siltstone and clay seams alternating	21.0	Sound, slightly weathered at 13.2-17.0 ft. Highly weathered with clay seams from 17.0 to 21.0 ft. bgs. Slightly weathered to fresh below.	Siltstone
BPA-21	687645.7	2011704.8	261.49	82.7	20.3	ML, SM	Alternating siltstone and sandstone	27.1	Slightly weathered and fresh below.	Alternating sandstone and siltstone
BPA-22	687755.6	2011728.0	262.12	85.0	20.2	CL, ML, SM	Alternating siltstone and sandstone	28.8	Slightly weathered below.	Sandstone to 29.9 ft., siltstone to 47.9 ft., over sandstone
BPA-23	687694.4	2011792.5	259.65	139.0	12.8	SM, CL, ML	Siltstone	28.0	Slightly weathered below.	Alternating siltstone and sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 5 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-24	687599.0	2011809.7	258.98	79.9	12.5	SM, CL, ML, and CH	Sandstone (with conglomerates, siltstone lenses, and clay seams) to 22.4 ft., over siltstone	12.6	Moderately weathered from 19.65 to 40.5 ft., with clay seams. Sound above and below this range.	Sandstone and conglomerate to 20.8 ft. and siltstone
BPA-25	687725.1	2011875.8	266.37	206.0	5.0	CL, CH	Sandstone to 13.25 ft., claystone to 16 ft., siltstone to 16.2 ft., over alternating sandstone siltstone	28.5	Slightly weathered to fresh below.	Alternating siltstone and sandstone
BPA-26	687898.8	2011865.2	252.34	85.0	15.1	SM, CH, CL	Siltstone, clay, and sandstone to 16.1 ft., over siltstone	25.0	Slightly weathered to fresh below.	Siltstone
BPA-27	687838.2	2011884.8	258.26	138.6	12.8	CL, SM	Siltstone with sandstone 20.1-20.4 ft.	28.0	Fresh below.	Siltstone
BPA-28	687697.1	2011943.3	272.70	155.5	17.8	SC, CL	Siltstone to 17.9 ft., over sandstone	25.5	Slightly weathered to fresh below.	Alternating siltstone and sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 6 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-29	687642.7	2011979.1	276.37	95.5	17.7	CL, SC, SM	NA	17.7	Slightly weathered to fresh below.	Alternating siltstone and sandstone with conglomerate at 26.4-26.9 ft.
BPA-30	687781.4	2011981.6	269.20	89.0	15.3	CH, CL	Alternating siltstone and sandstone with conglomerate 24.5-25.5 ft.	25.5	Slightly weathered to fresh below.	Siltstone
BPA-31	687883.1	2011990.2	263.09	145.0	15.1	SC, CL, SM	Siltstone	26.8	Fresh below.	Alternating siltstone and sandstone
BPA-32	687832.2	2012083.2	268.86	90.0	10.1	SM, CH, ML, CL	Sandstone to 20.95 ft., over siltstone	25.0	Fresh below.	Alternating siltstone and sandstone
BPA-33	687730.7	2012118.9	270.39	160.0	10.3	SM, CL	Sandstone, clay seam 26.8-28.3 ft., siltstone 28.3-32.85 ft.	32.6	Slightly weathered to fresh below.	Alternating siltstone and sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 7 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[a]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-34	687640.2	2012160.6	267.08	47.0	12.7	CL	Siltstone and clay seams alternating	30.5	Intervals of sound, slightly weathered rock alternating with moderately weathered rock and clay seams from 12.7 to 26.1 ft. bgs. Highly weathered with clay seams 26.1-30.5 ft. Fresh below.	Sandstone to 38.1 ft., over siltstone
BPA-35	687956.7	2012158.6	267.57	87.5	17.8	CH, SM, ML	Siltstone and sandstone alternating	24.9	Slightly weathered to fresh below.	Sandstone to 39.4 ft., over siltstone
BPA-36	687881.0	2012189.2	269.77	91.0	15.3	ML, CH, CL, SM	Sandstone to 26.8 ft., siltstone	34.7	Fresh below.	Alternating siltstone and sandstone
BPA-37	687795.2	2012225.3	266.51	46.0	10.3	SC, CL	Siltstone to 15.7 ft., sandstone to 19.3 ft. over siltstone	19.7	Slightly weathered to fresh and fresh below.	Siltstone to 37.2 ft. over sandstone
BPA-38	688010.8	2012267.8	264.45	45.5	22.1	CH, CL, ML	Siltstone and sandstone alternating	23.0	Fresh below.	Alternating siltstone and sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 8 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-39	687929.9	2012297.9	265.76	138.6	22.8	SM, CL, CH, SC	Siltstone	39.6	Fresh below.	Alternating siltstone and sandstone
BPA-40	687842.4	2012342.8	259.21	45.7	7.7	ML	Siltstone	12.9	Slightly weathered below.	Alternating siltstone and sandstone
BPA-41	686943.6	2012431.0	264.62	164.2	5.3	CL and SP-SM	Sandstone to 14.2 ft., siltstone	17.4	Interlayered moderately weathered and sound rock seams to this depth. Fresh below.	Alternating siltstone and sandstone
BPA-42	686687.3	2012578.9	264.29	85.0	7.6	CL	Siltstone	23.5	Slightly weathered below.	Siltstone
BPA-43	687805.9	2012031.0	269.72	150.0	15.3	CH, SM, CL, ML	Siltstone	26.9	Fresh below.	Alternating siltstone and sandstone
BPA-44	687543.3	2012170.4	266.34	87.0	10.3	SM, CH, SC, CL	Siltstone to 15.8 ft., sandstone	16.0	Slightly weathered and moderately weathered below.	Sandstone to 19.15 ft., over siltstone
BPA-45	686920.0	2012313.3	265.93	85.3	2.8	CL	Siltstone	5.3	Fresh below.	Alternating siltstone and sandstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 9 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BPA-46	687760.6	2011923.3	267.98	87.0	15.2	CL, SM	NA	15.2	Highly weathered at 27.55-30.0 ft. Sound above and below this range.	Alternating siltstone and sandstone
BPA-47	687003.1	2012367.5	264.45	207.0	6.5	ML	NA	6.5	Slightly weathered to fresh below.	Siltstone with conglomerate at 22.25-24.4 ft. and 29.35-29.55 ft.
BPA-48	686889.5	2012456.7	264.84	207.0	10.3	OH, CH, SM, CL	NA	10.3	Fresh below.	Siltstone
BPA-49	687867.1	2011965.5	261.87	203.0	19.1	CL	Siltstone to 19.6 ft., sandstone to 25.7 ft., shale to 26 ft., siltstone and sandstone alternating to 36 ft.	36.0	Highly weathered at 49.1-51.0 ft. Sound above and below this range.	Siltstone
BPA-50	687752.1	2012057.2	272.98	213.25	15.2	CL, ML, SM	Sandstone to 25.4 ft., claystone, clayey silt, and silt to 27.4 ft. and 29.25-29.8 ft. and siltstone to 27.4-29.25 ft.	29.8	Slightly weathered to fresh below.	Siltstone with sandstone at 37.5-43.5 ft.

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 10 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[a]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BGA-1	686595.5	2012465.3	263.88	85.0	5.1	CL	Siltstone	14.2	Fresh below.	Siltstone to 31.2 ft., conglomerate to 32.0 ft., and sandstone
BGA-2	687127.5	2012218.4	262.41	85.0	5.2	CL, SC	Gravel to 6.5 ft., sandstone to 17.95 ft., over siltstone	18.6	Slightly weathered to fresh below.	Siltstone
BGA-3	687317.4	2012132.9	259.89	166.0	10.1	SM, CL	NA	10.1	Slightly weathered to fresh below.	Siltstone, sandstone at 16.35-18.6 ft.
BGA-4	687537.9	2012026.1	274.99	95.0	15.2	SC, CH, CL, SM	Siltstone to 15.5 ft., sandstone to 21 ft., siltstone	25.0	Highly weathered at 20.0-25.0 ft. Sound above and below this range.	Alternating siltstone and sandstone
BGA-5	687151.7	2011758.3	268.36	90.0	12.6	CL, MH	NA	12.7	Slightly weathered to fresh below; moderately weathered at 31.3-41.3 ft.	Alternating siltstone and sandstone
BGA-6	687589.4	2011550.8	275.15	95.9	17.7	SC, CL, SM	Sandstone	41.5	Slightly weathered to fresh below.	Alternating siltstone and sandstone

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Table 2.5.4-201 (Sheet 11 of 15) Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BGA-7	687886.0	2011583.5	267.78	60.0	12.6	SM, CL, SC,	Alternating siltstone and sandstone	25.0	Slightly weathered to fresh below; moderately weathered at 30.0-33.0 ft. and 37.95-40.0 ft.; highly weathered at 33.0-37.95 ft.	Alternating siltstone, sandstone and conglomerate
BGA-8	688088.7	2011762.2	263.63	84.0	15.1	CL, CH	NA	15.2	Highly weathered from 26.3 to 28.0 ft. and moderately weathered from 37.4 to 41.0 ft. Slightly weathered to fresh.	Alternating siltstone and sandstone
BGA-9	688154.6	2012066.0	249.82	69.5	7.8	OL, CH, CL	Alternating siltstone and sandstone	24.5	Slightly weathered and fresh below; moderately weathered at 34.5-44.5 ft.	Siltstone
BGA-10	688222.3	2012329.5	246.22	54.5	22.6	CH, CL, SC, SM, MH	NA	22.6	Slightly weathered and fresh below.	Alternating siltstone and sandstone
BGA-11	687997.7	2012441.8	246.78	59.5	7.7	ML, CH, CL	Siltstone and sandstone	10.8	Slightly weathered to fresh below.	Sandstone to 15.6 ft., over siltstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 12 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BGA-12	687173.0	2012923.7	260.89	81.0	12.7	CL	Clayey silt (ML) and siltstone	14.0	Fresh below.	Siltstone
BGA-13	686459.9	2011681.2	260.62	79.2	7.7	GC, CL	Siltstone	24.2	Moderately weathered at 29.2-39.2 ft.; sound above and below this range.	Siltstone
BGA-14	687506.0	2011204.9	287.02	77.5	22.7	GM, ML, CL, SM	Siltstone	37.5	Slightly weathered and fresh below.	Siltstone
BGA-15	688250.0	2011676.9	266.59	65.6	15.3	CL	Siltstone	16.0	Slightly weathered to moderately to slightly weathered below.	Alternating siltstone and sandstone
BGA-16	687118.1	2012814.9	260.64	65.0	7.8	CL	Siltstone	15.2	Moderately weathered at 25.8-29.3 ft. Sound above and below this range.	Siltstone
BGA-17	687204.0	2012991.7	261.55	83.40	32.8	CL	NA	32.8	Slightly weathered and fresh below.	Siltstone and partially metamorphosed siltstone
BGA-18	686565.5	2012711.3	263.45	65.4	12.75	GM, CL	Siltstone	15.4	Fresh and moderately weathered below.	Siltstone to 16.0 ft., sandstone to 20.7 ft., over siltstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 13 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BCA-1	686832.1	2014476.2	256.24	100.3	45	CL	Sandstone with clay intervals	46.8	Moderately weathered to fresh siltstone.	Siltstone
BCA-2	688236.2	2013338.7	244.35	25.5	7.75	ML, CL	Siltstone and sandstone with clay intervals	17.1	Slightly weathered to fresh below.	Siltstone with sandstone intervals
BCA-3	687233.2	2012525.3	262.59	30.3	2.7	Fill, CL	NA	2.9	Moderately weathered to fresh below.	Siltstone
BCA-4	687298.2	2013040.6	262.88	40.2	15.1	GM, CL, ML, SC	NA	15.2	Mostly fresh; moderately weathered at top 1.5 ft.	Siltstone
BCA-5	687400.5	2012819.7	260.25	40.0	7.7	SM, ML	NA	7.6	Highly weathered at 19.6-22.1 ft.; sound above and below this range.	Siltstone
BCA-6	688172.3	2012652.4	245.37	40.5	10.9	ML, CL	Siltstone with clay intervals	19.4	Slightly weathered to fresh below.	Alternating siltstone and sandstone
BCA-7	687127.9	2013818.7	263.17	30.15	30.15	CL, ML, SP-SM	NA	NA	No rock coring performed.	
BCTA-1	687669.0	2013139.4	260.42	85.3	13.75	GM, CL, SM, SW-SM, ML, SP-SM	Siltstone with clay intervals	17.0	Slightly weathered to fresh below.	Siltstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 14 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[(a)]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BCTA-2	687760.2	2013343.4	260.97	85.0	23.5	SC, CL, CH, ML	Siltstone and sandstone with clay intervals	34.5	Moderately weathered from 44.5 to 49.5 ft.; sound above and below this range.	Alternating siltstone and sandstone
BCTA-3	687448.8	2013227.8	260.31	40.5	10.25	CL	Siltstone and sandstone alternating, with few clay seams	24.4	Slightly weathered to fresh below.	Sandstone with siltstone intervals
BCTA-4	687562.0	2013436.5	262.81	49.1	32.6	CL, CH	Not recovered.	33.4	Slightly weathered to fresh below.	Siltstone
BCTA-5	688529.9	2012723.2	243.52	86.3	10.2	CL, CH, MH	Siltstone with clay intervals	11.6	Moderately weathered at 19.5-24.5 ft.; sound above and below this range.	Siltstone
BCTA-6	688618.1	2012946.1	239.82	85.0	10.2	ML, SC	Siltstone and sandstone with clay intervals	15	Moderately weathered from 30.0 to 35.0 ft.; sound above and below this range.	Alternating siltstone and sandstone
BCTA-7	688279.2	2012780.6	232.79	40.0	10.2	CL	Moderately weathered siltstone with few clay intervals	20	Fresh below.	Siltstone

HAR COL 2.5-6

Table 2.5.4-201 (Sheet 15 of 15)
Summary of Borehole and Rock Core Observations

Borehole ID	Northing (NAD 27, ft.)	Easting (NAD 27, ft.)	Ground Elev. (NGVD 29, ft.)	Total Borehole Depth (ft. bgs)	Soil Depth (ft. bgs)	Soil Description	Weathered Rock Description	Top of Sound Rock Depth (ft. bgs)[a]	Notes on Sound Rock Depth	Sound Rock Description (upper 20 ft. of sound rock)
BCTA-8	688410.9	2013044.2	227.01	41.0	15.05	CL, ML	Slightly weathered and moderately weathered siltstone and sandstone	24.5	Slightly weathered and fresh below.	Siltstone with sandstone underlying

a) Top of sound rock depth is identified as the shallowest encounter of fresh or slightly weathered rock, with fresh or slightly weathered rock extending continuously for 5 ft. below this depth. Where highly weathered or very weak rock or clay seams are identified within 10 ft. below this depth, they are described in the “Notes on Sound Rock Depth” column.

Notes:
NAD = North American Datum
NGVD = NGVD National Geodetic Vertical Datum

HAR COL 2.5-1
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Table 2.5.4-202
Dip of Rock Strata

		Dip Orientation	
Marker Bed ID	**Brief Description of Marker Bed**	**Dip Angle (degrees from horizontal)**	**Direction (degrees clockwise from State Plane North)**
HAR 2			
Marker Bed A	Gray fine to coarse sandstone	6	96
Marker Bed B	Gray shaley siltstone	8.5	76
V_s Profiles	See Note[(a)]	8.9	91
Acoustic Televiewer Bedding Features	See Note[(b)]	6.6	75.5
HAR 3			
Marker Bed A	Gray fine sandstone to sandy siltstone	19.5	110
Marker Bed B	Dark gray fine sandstone to sandy siltstone	21	114
V_s Profiles	See Note[(a)]	19.9	109
Acoustic Televiewer Bedding Features	See Note[(b)]	23.2	113.6

Notes:
a) Dip angle and direction listed are based on the best match of V_s data from suspension logging profiles in Boreholes BPA-5, BPA-47, and BPA-48 at HAR 2, and in Boreholes BPA-25, BPA-49, and BPA-50 at HAR 3.

b) Bedding features as identified on the acoustic televiewer logs.

HAR COL 2.5-6

Table 2.5.4-203 (Sheet 1 of 2)
Rock Quality Designation (RQD) Results – Boreholes Near HAR 2 and HAR 3

Borehole ID	Elevation of Borehole Bottom	Average RQD Elevation 220 ft.[(a)] to Bottom	Average RQD Elevation 220-170 ft.	Average RQD Elevation 170 ft. & Below
BGA-1[(b)]	178.9	97	97	NA
BGA-2[(b)]	177.4	87	87	NA
BGA-3	93.4	88	92	86
BGA-4[(b)]	180.0	100	100	NA
BGA-6[(b)]	179.3	88	88	NA
BGA-16[(b)]	195.9	97	97	NA
BPA-1[(b)]	178.6	90	90	NA
BPA-2	159.7	85	83	93
BPA-3	101.4	89	87	90
BPA-4	159.6	91	88	100
BPA-5	58.7	97	100	96
BPA-6	154.3	66	77	35
BPA-7	120.0	94	97	92
BPA-8	119.9	91	94	88
BPA-9[(b)]	179.5	92	92	NA
BPA-10[(b)]	178.9	98	98	NA
BPA-11	105.1	87	90	84
BPA-12[(b)]	176.9	90	90	NA
BPA-13	99.6	87	88	86
BPA-16	99.5	93	96	91
BPA-19[(b)]	179.4	96	96	NA
BPA-21[(b)]	178.8	98	98	NA
BPA-22[(b)]	177.1	91	91	NA
BPA-23	120.7	93	89	97
BPA-24[(b)]	179.1	88	88	NA
BPA-25	60.4	94	90	95
BPA-26	167.3	94	94	98[(c)]
BPA-27	119.7	95	92	97
BPA-28	117.2	98	96	100
BPA-29[(b)]	180.9	92	92	NA
BPA-30[(b)]	180.2	86	86	NA
BPA-31	118.1	89	80	97

HAR COL 2.5-6

Table 2.5.4-203 (Sheet 2 of 2)
Rock Quality Designation (RQD) Results – Boreholes Near HAR 2 and HAR 3

Borehole ID	Elevation of Borehole Bottom	Average RQD Elevation 220 ft.[(a)] to Bottom	Average RQD Elevation 220-170 ft.	Average RQD Elevation 170 ft. & Below
BPA-32[(b)]	178.9	96	96	NA
BPA-33	110.4	89	93	86
BPA-36[(b)]	178.8	99	99	NA
BPA-38[(b)]	219.0	87[(c)]	87[(c)]	NA
BPA-39	127.2	97	97	97
BPA-41	100.4	90	92	88
BPA-43	119.7	94	90	96
BPA-44[(b)]	179.3	89	89	NA
BPA-45[(b)]	180.6	97	97	NA
BPA-46[(b)]	181.0	92	92	NA
BPA-47	57.5	92	92	92
BPA-48	59.1	94	95	93
BPA-49	58.9	93	83	98
BPA-50	59.7	94	96	93

Notes:
a) ft. = feet above mean sea level.

b) This borehole did not extended below elevation 170 ft.

c) Value represents only one data point.

HAR COL 2.5-6

Table 2.5.4-204
Rock Pressuremeter Test (PMT) Results

Borehole	Depth (ft.)	Elevation (ft.)	Pressuremeter Modulus, Er (ksi)	Pressuremeter Modulus, Er (ksf)
HAR 2				
BPA-3	17.1	243.5	366	5.3E+04
BPA-3	43.0	217.6	694	1.0E+05
BPA-3	51.8	208.8	668	9.6E+04
BPA-3	61.0	199.6	847	1.2E+05
BPA-3	83.0	177.6	633	9.1E+04
BPA-3	101.0	159.6	1022	1.5E+05
BPA-3	128.9	131.7	1055	1.5E+05
BPA-41	28.5	236.1	698	1.0E+05
BPA-41	51.8	212.8	732	1.1E+05
BPA-41	62.0	202.6	844	1.2E+05
BPA-41	79.1	185.5	1482	2.1E+05
BPA-41	92.8	171.8	860	1.2E+05
BPA-41	107.9	156.7	597	8.6E+04
BPA-41	129.3	135.3	1728	2.5E+05
HAR 3				
BPA-23	23.6	236.1	5.1 *	7.3E+02 *
BPA-23	45.9	213.8	19.5 *	2.8E+03 *
BPA-23	62.0	197.7	175	2.5E+04
BPA-23	80.1	179.6	503	7.2E+04
BPA-23	92.8	166.9	822	1.2E+05
BPA-23	125.0	134.7	1131	1.6E+05
BPA-43	24.6	245.1	3.0 *	4.3E+02 *
BPA-43	35.1	234.6	4.2 *	6.1E+02 *
BPA-43	46.9	222.8	57	8.1E+03
BPA-43	56.1	213.6	81	1.2E+04
BPA-43	69.9	199.8	350	5.0E+04
BPA-43	83.0	186.7	425	6.1E+04
BPA-43	101.0	168.7	429	6.2E+04
BPA-43	119.1	150.6	1119	1.6E+05

Notes:
ksf = kips per square foot
ksi = kips per square inch

* This test result is not considered representative of the tested rock because only 2 data points were available to calculate the modulus. This result is not used in engineering analyses.

HAR COL 2.5-6

Table 2.5.4-205 (Sheet 1 of 4)
Rock Laboratory Test Results

Borehole ID	Sample ID	Sample Type	Depth (ft. bgs)		Elevation (ft.)		UCS (psi)	Secant Modulus (x10^6 psi)[b]	Poisson's Ratio – Secant[b]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) (x10^6 psi)[b]	Tangent Modulus (Radial) (x10^6 psi)[b]	Poisson's Ratio – Tangent[b]
Samples At or Near HAR 2														
BPA-2	SC-3	Siltstone	76.25	77.5	183.5	182.2	4092	N/A [c]	N/A	165.9	1.1	N/A	N/A	N/A
BPA-3	SC-2	Siltstone	49.85	50.8	210.7	209.8	6625	1.18	0.19	167.3	1.3	1.90	5.33	0.36
BPA-3	SC-3	Siltstone	82.4	83.2	178.2	177.4	5633	0.86	0.15	166.4	1.3	1.48	4.83	0.31
BPA-3	SC-5	Sandstone	129.2	130.15	131.4	130.4	12638	2.34	0.22	164.7	2.2	2.73	6.14	0.44
BPA-5	SC-2	Siltstone	59.28	60.4	206.5	205.3	11245	2.14	0.17	168.2	1.1	2.56	9.13	0.28
BPA-5	SC-3	Sandstone/Shale	96	97.1	169.7	168.6	10924	2.28	0.24	168.8	1.2	2.43	6.64	0.37
BPA-5	SC-5	Siltstone	156	157.45	109.7	108.3	6925	2.44	0.23	168	0.8	2.44	9.10	0.27
BPA-6	SC-3	Sandstone	40	41.1	224.3	223.2	1138	N/A	N/A	154.6	4	N/A	N/A	N/A
BPA-6	SC-4	Siltstone	62.8	63.4	201.5	200.9	1388	N/A	N/A	165.6	2.5	N/A	N/A	N/A
BPA-6	GS-1[a]	Sandstone	74.2	75	190.1	189.3	1672	N/A	N/A	151.1	4.4	N/A	N/A	N/A
BPA-6	GS-2[a]	Sandstone	42.8	43.5	221.5	220.8	1508	N/A	N/A	142.8	4.8	N/A	N/A	N/A
BPA-7	SC-1	Siltstone	19	20	246.0	245.0	5288	0.45	0.13	163.7	3.4	0.82	1.87	0.44
BPA-7	SC-2	Siltstone	65.2	66.35	199.8	198.6	9049	1.36	0.2	169.8	1.5	1.71	4.49	0.38
BPA-7	SC-4	Siltstone	118.9	120	146.1	145.0	10636	2.24	0.15	166.2	0.8	2.52	9.64	0.26
BPA-8	SC-1	Siltstone	30.3	31.25	237.1	236.2	12449	1.97	0.09	164.5	1.3	3.04	8.43	0.36
BPA-8	SC-2	Sandstone	57.1	58.1	210.3	209.3	6971	0.71	0.11	155.9	3.7	1.36	4.02	0.34
BPA-8	SC-3	Siltstone	79.7	80.7	187.7	186.7	14670	2.1	0.17	166.1	1	2.86	7.09	0.40
BPA-15	SC-1	Siltstone	35.25	36.2	227.4	226.5	3217	0.44	0.4	166.4	1.8	0.36	0.69	0.52
BPA-16	SC-2	Sandstone	56.15	57.15	206.6	205.6	5636	0.71	0.15	154.2	3.8	1.27	3.10	0.41
BPA-16	SC-4	Siltstone	137.5	138.5	125.3	124.3	10445	1.58	0.2	168.7	0.7	2.01	5.60	0.36
BPA-16	SC-5	Siltstone	145.65	146.45	117.1	116.3	4151	0.89	0.16	167	1.1	1.39	4.91	0.28
BPA-19	SC-1	Sandstone	24.75	25.6	236.6	235.8	9926	1.23	0.13	166	1.9	1.98	4.84	0.41
BPA-19	SC-3	Siltstone	75.05	76.05	186.3	185.3	7134	0.73	0.24	167.5	1.4	1.53	3.24	0.47
BPA-41	SC-2	Siltstone	52.4	53.4	212.2	211.2	8354	1.69	0.33	163.3	2.6	1.73	3.59	0.48

HAR COL 2.5-6

Table 2.5.4-205 (Sheet 2 of 4)
Rock Laboratory Test Results

Borehole ID	Sample ID	Sample Type	Depth (ft. bgs)		Elevation (ft.)		UCS (psi)	Secant Modulus ($x10^6$ psi)[b]	Poisson's Ratio – Secant[b]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) ($x10^6$ psi)[b]	Tangent Modulus (Radial) ($x10^6$ psi)[b]	Poisson's Ratio – Tangent[b]
BGA-16	SC-3	Siltstone	60	60.8	200.6	199.8	4442	1.13	0.46	167.8	0.7	0.90	1.83	0.49
BPA-47	SC-12	Siltstone	17	18.05	247.5	246.4	1890	N/A	N/A	161.5	2.9	N/A	N/A	N/A
BPA-47	SC-1	Siltstone	27.5	28.35	237.0	236.1	2037	N/A	N/A	167.3	1.3	N/A	N/A	N/A
BPA-47	SC-2	Siltstone	44.45	45.3	220.0	219.2	3615	N/A	N/A	166.6	2.2	N/A	N/A	N/A
BPA-47	SC-3	Siltstone	65.05	66	199.4	198.5	1758	N/A	N/A	167	1.4	N/A	N/A	N/A
BPA-47	SC-4	Siltstone	72.5	73.2	192.0	191.3	10256	2.35	0.41	166.6	0.7	2.15	4.27	0.50
BPA-47	SC-5	Siltstone	80.1	81.2	184.4	183.3	3444	N/A	N/A	166.6	2.1	N/A	N/A	N/A
BPA-47	SC-6	Siltstone	102	102.9	162.5	161.6	5901	N/A	N/A	164.9	1.6	N/A	N/A	N/A
BPA-47	SC-8	Siltstone	113.85	144.8	150.6	119.7	1820	N/A	N/A	181.3	1.4	N/A	N/A	N/A
BPA-48	SC-1	Sandstone	38.25	38.85	226.6	226.0	2676	N/A	N/A	152.8	4.9	N/A	N/A	N/A
BPA-48	SC-3	Siltstone	46.15	47	218.7	217.8	4135	0.78	0.43	165.6	1.6	0.85	1.49	0.57
BPA-48	SC-4	Siltstone	57	57.65	207.8	207.2	5921	N/A	N/A	167.3	1.5	N/A	N/A	N/A
BPA-48	SC-6	Siltstone	125.5	126.2	139.3	138.6	8485	N/A	N/A	166.5	0.8	N/A	N/A	N/A
Samples At or Near HAR 3														
BPA-21	SC-1	Sandstone	35.2	36.35	226.3	225.1	3012	N/A	N/A	162.4	3	N/A	N/A	N/A
BPA-23	SC-2	Siltstone	21.8	23	237.9	236.7	2001	0.39	0.29	159.3	5.6	0.61	1.11	0.55
BPA-23	SC-3	Siltstone	51.1	52.5	208.6	207.2	12108	2.29	0.28	163	1.8	2.44	5.71	0.43
BPA-23	SC-4	Siltstone	85.6	86.8	174.1	172.9	6647	1.43	0.23	168.1	1.6	2.15	4.59	0.47
BPA-23	SC-6	Siltstone	112.15	113.15	147.5	146.5	4990	0.75	0.17	165.2	1.8	1.24	3.27	0.38
BPA-25	SC-2	Siltstone	53.35	54.65	213.0	211.7	2616	N/A	N/A	166.2	1.4	N/A	N/A	N/A
BPA-25	SC-3	Siltstone	93	94.1	173.4	172.3	17851	3.22	0.25	167.7	1.5	4.05	6.99	0.58
BPA-25	SC-5	Siltstone	141.1	142.4	125.3	124.0	9993	2.13	0.15	169.1	0.8	2.45	8.69	0.28
BPA-27	SC-1	Siltstone	55.8	56.4	202.5	201.9	8365	0.93	0.16	163.9	1.8	1.36	3.98	0.34
BPA-27	SC-3	Siltstone	80.45	81.25	177.8	177.0	9711	1.38	0.18	169.3	1	2.18	7.14	0.30
BPA-28	SC-1	Siltstone	35.5	36.5	237.2	236.2	5606	0.39	0.18	161.9	2.3	1.23	1.77	0.69

HAR COL 2.5-6

Table 2.5.4-205 (Sheet 3 of 4)
Rock Laboratory Test Results

Borehole ID	Sample ID	Sample Type	Depth (ft. bgs)		Elevation (ft.)		UCS (psi)	Secant Modulus ($x10^6$ psi)[(b)]	Poisson's Ratio – Secant[(b)]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) ($x10^6$ psi)[(b)]	Tangent Modulus (Radial) ($x10^6$ psi)[(b)]	Poisson's Ratio – Tangent[(b)]
BPA-30	SC-1	Siltstone	48	48.7	221.2	220.5	3757	N/A	N/A	165.3	1.2	N/A	N/A	N/A
BPA-32	SC-2	Siltstone	77.1	78.45	191.8	190.4	2742	0.39	0.26	169.6	1.8	0.50	1.27	0.39
BPA-36	SC-1	Sandstone	20.2	20.9	249.6	248.9	2739	0.18	0.21	146.7	7.5	0.28	0.72	0.40
BPA-36	SC-3	Sandstone	49.9	51	219.9	218.8	4134	0.77	0.13	145.9	6.7	1.36	4.13	0.33
BPA-36	SC-5	Siltstone	89.75	90.55	180.0	179.2	8212	0.95	0.12	163.3	1.3	1.99	4.37	0.46
BPA-39	SC-1	Siltstone	27.75	28.85	238.0	236.9	2711	0.3	0.36	162.8	1.8	0.28	0.65	0.42
BPA-39	SC-2	Siltstone	59.6	60.6	206.2	205.2	2932	N/A	N/A	164.4	1.7	N/A	N/A	N/A
BPA-43	SC-1	Siltstone	16.3	17.15	253.4	252.6	565.2	N/A	N/A	177.4	6.3	N/A	N/A	N/A
BPA-43	SC-2	Siltstone	39.15	39.8	230.6	229.9	7151	1.35	0.36	162.6	2.1	1.28	2.59	0.50
BPA-43	SC-3	Sandstone	65.9	67	203.8	202.7	6138	0.97	0.27	161.6	1.9	1.09	2.37	0.46
BPA-46	SC-1	Sandstone	48.1	49.2	219.9	218.8	6539	0.9	0.25	164.3	2	1.67	2.33	0.72
BPA-46	SC-2	Siltstone	70	70.95	198.0	197.0	4623	1.16	0.32	168.5	1.4	1.31	2.91	0.45
BPA-46	GS-1[(a)]	Siltstone	64.1	64.4	203.9	203.6	5839	N/A	N/A	165.8	1.2	N/A	N/A	N/A
BPA-49	SC-2	Siltstone	41	41.6	220.9	220.3	3484	N/A	N/A	165.9	2.1	N/A	N/A	N/A
BPA-49	SC-5	Siltstone	52.3	53.15	209.6	208.7	4860	N/A	N/A	164.1	1.7	N/A	N/A	N/A
BPA-49	SC-6	Siltstone	71	72	190.9	189.9	4834	1.52	0.38	168.5	1.2	1.26	2.94	0.43
BPA-49	SC-9	Siltstone	103	104	158.9	157.9	4758	N/A	N/A	166	1.7	N/A	N/A	N/A
BPA-50	SC-2	Siltstone	55.9	56.5	217.1	216.5	3612	N/A	N/A	160.5	2.4	N/A	N/A	N/A
BPA-50	SC-13	Sandstone	63.1	63.9	209.9	209.1	2347	N/A	N/A	150	6.5	N/A	N/A	N/A
BPA-50	SC-4	Siltstone	80.67	81.17	192.3	191.8	2214	N/A	N/A	167.6	2.3	N/A	N/A	N/A
BGA-3	SC-5	Siltstone	149.65	150.85	110.2	109.0	7049	1.36	0.35	168.1	0.4	1.29	3.02	0.43
BGA-6	SC-1	Sandstone	27.4	28.3	247.8	246.9	1008	N/A	N/A	141.6	6.8	N/A	N/A	N/A
BGA-6	SC-2	Siltstone	67.8	68.7	207.4	206.5	10040	N/A	N/A	185.5	1.3	N/A	N/A	N/A

HAR COL 2.5-6

Table 2.5.4-205 (Sheet 4 of 4)
Rock Laboratory Test Results

Borehole ID	Sample ID	Sample Type	Depth (ft. bgs)		Elevation (ft.)	UCS (psi)	Secant Modulus ($x10^6$ psi)[b]		Poisson's Ratio – Secant[b]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) ($x10^6$ psi)[b]	Tangent Modulus (Radial) ($x10^6$ psi)[b]	Poisson's Ratio – Tangent[b]
Other Samples														
BCA-1	SC-1	Siltstone	68.55	69.4	187.7	186.8	11448	N/A	N/A	165.8	1.1	N/A	N/A	N/A
BCTA-1	SC-1	Siltstone	40.3	41.25	220.1	219.2	9915	N/A	N/A	167.3	1.1	N/A	N/A	N/A
BCTA-2	SC-2	Sandstone	61.95	62.95	199.0	198.0	3333	0.48	0.24	148.7	4.3	0.89	1.41	0.63
BCTA-4	SC-1	Siltstone	40.45	41.45	222.4	221.4	8531	0.97	0.2	165.8	1.3	1.75	3.40	0.52
BCTA-5	SC-1	Siltstone	26.65	27.6	216.9	215.9	3434	0.33	0.48	163.7	2	0.27	0.51	0.53
BCTA-5	SC-2	Siltstone	78.45	79.4	165.1	164.1	7016	N/A	N/A	165.8	1.4	N/A	N/A	N/A
BCTA-7	SC-1	Siltstone	23.6	24.7	209.2	208.1	6611	0.98	0.43	167.3	1.2	0.96	1.82	0.53

a) Grab rock samples (designated as "GS") were collected after the field event. Water contents were not preserved in these samples prior to testing. Corresponding test results are provided in this table, but these have not been included in statistical calculations.

b) Result listed was measured at one-half of the failure strain.

c) "N/A" indicates that strain measurements were not performed for these tests, and therefore these parameters are not reported.

Notes:
pcf = pounds per cubic foot
psi = pounds per square inch

HAR COL 2.5-6

Table 2.5.4-206 (Sheet 1 of 2)
Rock Laboratory Test Result Statistics

HAR Unit and Sample Group	Statistic	UCS (psi)	Secant Modulus (x10^6 psi)[(a)]	Poisson's Ratio – Secant[(a)]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) (x10^6 psi)[(a)]	Tangent Modulus (Radial) (x10^6 psi)[(a)]	Poisson's Ratio – Tangent[(a)]
HAR 2 Results									
All Samples Below Elevation 220 ft.	*Average*	7032	1.54	0.23	166.4	1.6	1.90	5.32	0.39
	Minimum	1388	0.71	0.11	154.2	0.7	0.85	1.49	0.26
	Maximum	14670	2.44	0.46	181.3	3.8	2.86	9.64	0.57
	Standard Deviation	3488	0.65	0.10	4.4	0.8	0.61	2.33	0.09
	Count	28	19	19	28	28	19	19	19
Elevation 170 to 220 ft.	*Average*	6578	1.34	0.24	165.5	1.7	1.74	4.54	0.41
	Minimum	1388	0.71	0.11	154.2	0.7	0.85	1.49	0.28
	Maximum	14670	2.35	0.46	169.8	3.8	2.86	9.13	0.57
	Standard Deviation	3452	0.60	0.12	3.9	0.9	0.60	2.12	0.09
	Count	19	13	13	19	19	13	13	13
Below Elevation 170 ft.	*Average*	7992	1.96	0.20	168.5	1.2	2.25	7.00	0.33
	Minimum	1820	0.89	0.15	164.7	0.7	1.39	4.91	0.26
	Maximum	12638	2.44	0.24	181.3	2.2	2.73	9.64	0.44
	Standard Deviation	3567	0.61	0.04	5.0	0.5	0.48	1.93	0.07
	Count	9	6	6	9	9	6	6	6
HAR 3 Results									
All Samples Below Elevation 220 ft.	*Average*	6532	1.43	0.23	165.2	2.0	1.83	4.57	0.42
	Minimum	2214	0.39	0.12	145.9	0.4	0.50	1.27	0.28
	Maximum	17851	3.22	0.38	185.5	6.7	4.05	9.48	0.72
	Standard Deviation	3836	0.78	0.08	7.0	1.5	0.87	2.39	0.11
	Count	25	16	16	25	25	16	16	16

HAR COL 2.5-6

Table 2.5.4-206 (Sheet 2 of 2)
Rock Laboratory Test Result Statistics

HAR Unit and Sample Group	Statistic	UCS (psi)	Secant Modulus (x10^6 psi)[(a)]	Poisson's Ratio – Secant[(a)]	Bulk Density (pcf)	Moisture Content (%)	Tangent Modulus (Axial) (x10^6 psi)[(a)]	Tangent Modulus (Radial) (x10^6 psi)[(a)]	Poisson's Ratio – Tangent[(a)]
Elevation 170 to 220 ft.	*Average*	6200	1.33	0.24	164.9	2.2	1.78	4.06	0.45
	Minimum	2214	0.39	0.12	145.9	1.0	0.50	1.27	0.30
	Maximum	17851	3.22	0.38	185.5	6.7	4.05	7.14	0.72
	Standard Deviation	3957	0.76	0.08	7.8	1.6	0.90	1.84	0.11
	Count	20	12	12	20	20	12	12	12
Below Elevation 170 ft.	*Average*	7859	1.73	0.21	166.5	1.2	1.99	6.11	0.35
	Minimum	4758	0.75	0.15	164.3	0.4	1.24	3.02	0.28
	Maximum	12504	2.69	0.35	169.1	1.8	2.99	9.48	0.43
	Standard Deviation	3340	0.85	0.09	2.0	0.6	0.87	3.45	0.06
	Count	5	4	4	5	5	4	4	4

Notes:
a) Result listed was measured at one-half of the failure strain.

HAR COL 2.5-6

Table 2.5.4-207 (Sheet 1 of 4)
Petrographic Examination Thin Section Descriptions

Borehole ID	Sample ID	Depth (ft.)	Lithology	Grain Size (mm)	Sorting	Pore Fill/ Cements	Framework Grains/Allochems	Matrix	Grain Packing/ Compaction
BPA-2	SC-3	76.25-77.5	Micaceous mudstone	0.01-0.04	Poor	Abundant hematitic matrix	Monocrystalline quartz, plagioclase, metamorphic rock fragments, muscovite mica	Clay, hematite	Compacted to bioturbated
BPA-3	SC-3	82.4-83.2	Micaceous shaly siltstone	0.01-0.02	Poor	Shale matrix, minor hematite, minor pyrite	Monocrystalline quartz, plagioclase, muscovite mica	Clay, minor hematite	Compacted to bioturbated
BPA-10	SC-3	60.0-61.15	Medium to coarse grained sandstone	0.21-0.84	Moderate to poor	Silica cement, minor calcite, minor pyrite	Monocrystalline and polycrystalline quartz, plagioclase, schistose and gneissic metamorphic rock fragments, heavy mineral grains, muscovite mica	Minor clay matrix	Strong compaction, deformation of ductile grains
BPA-13	SC-4	126.4-127.4	Argillaceous siltstone	0.02-0.06	Poor	Clay matrix, pyrite	Monocrystalline quartz, plagioclase, schistose metamorphic rock fragments	Abundant clay matrix, hematite	Compacted fabric, deformed grains
BPA-16	SC-5	145.65-146.45	Micaceous siltstone	0.02-0.03	Poor	Clay matrix, pyrite	Monocrystalline quartz, abundant muscovite mica, plagioclase	Mixture of clay and fine mica flakes	Compacted fabric

HAR COL 2.5-6

Table 2.5.4-207 (Sheet 2 of 4)
Petrographic Examination Thin Section Descriptions

Borehole ID	Sample ID	Depth (ft.)	Lithology	Grain Size (mm)	Sorting	Pore Fill/ Cements	Framework Grains/Allochems	Matrix	Grain Packing/ Compaction
BPA-23	SC-3	51.1-52.5	Very fine grained sandstone	0.08-0.10	Poor	Clay matrix, minor silica, minor calcite pyrite	Monocrystalline and polycrystalline quartz, abundant muscovite mica, plagioclase, schistose and gneissic metamorphic rock fragments, heavy mineral grains	Clay matrix, hematite stain	Compacted to bioturbated fabric
BPA-23	SC-4	85.6-86.8	Micaceous siltstone	0.01-0.05	Poor	Abundant clay matrix, hematite, pyrite concentrations in some burrows	Monocrystalline quartz, muscovite mica, plagioclase	Clay matrix, hematite stain	Compacted to bioturbated fabric
BPA-25	SC-2	53.35-54.65	Argillaceous siltstone	0.04-0.06	Poor	Hematite stained matrix, pyrite	Monocrystalline quartz, muscovite mica, plagioclase	Clay matrix, heavy hematite stain	Compacted to bioturbated fabric
BPA-41	SC-2	52.4-53.4	Argillaceous, terrigeneous mudstone	0.04-0.08	Poor	Abundant hematite stained matrix, minor silica, minor calcite, pyrite	Monocrystalline and polycrystalline quartz, schistose and gneissic metamorphic rock fragments, plagioclase, heavy mineral grains	Abundant hematite stained shale matrix	Compacted fabric, deformation of ductile grains
BPA-41	SC-3	82.25-83.2	Argillaceous, terrigeneous mudstone	0.04-0.11	Poor	Hematite stained matrix, minor pyrite	Abundant muscovite mica, monocrystalline quartz, schistose and gneissic metamorphic rock fragments, plagioclase	Abundant fine muscovite mica, clay, hematite	Compacted to bioturbated fabric

HAR COL 2.5-6

Table 2.5.4-207 (Sheet 3 of 4)
Petrographic Examination Thin Section Descriptions

Borehole ID	Sample ID	Depth (ft.)	Lithology	Grain Size (mm)	Sorting	Pore Fill/ Cements	Framework Grains/Allochems	Matrix	Grain Packing/ Compaction
BPA-41	SC-5	139.9-140.8	Micaceous siltstone	0.02-0.06	Poor	Hematite stained matrix, pyrite, minor silica	Abundant muscovite mica, monocrystalline quartz, plagioclase feldspar, schistose metamorphic rock fragments	Hematite stained shale, fine muscovite	Compacted to bioturbated fabric
BPA-43	SC-2	39.15-39.8	Argillaceous siltstone	0.02-0.08	Poor	Hematite stained matrix, pyrite	Abundant muscovite mica, monocrystalline quartz, plagioclase	Hematite stained shale, fine muscovite	Compacted to bioturbated fabric
BPA-43	SC-3	65.9-67.0	Argillaceous very fine to fine grained sandstone	0.06-0.12	Moderate to poor	Silica, minor clay, hematite stained shale matrix	Monocrystalline quartz, polycrystalline quartz, muscovite mica, schistose and gneissic metamorphic rock fragments, heavy mineral grains	Shale, abundant hematite	Heavily compacted fabric, bioturbated(?), deformation of ductile grains
BPA-46	SC-1	48.1-49.2	Medium grained sandstone	0.18-0.60	Poor	Silica cement, minor clay matrix	Abundant schistose and gneissic metamorphic rock fragments, monocrystalline and polycrystalline quartz, plagioclase, heavy mineral grains	Minor clay matrix	Heavily compacted fabric, deformation of ductile grains
BPA-46	SC-2	70.0-70.95	Micaceous siltstone	0.01-0.02	Poor	Hematite stained matrix	Monocrystalline quartz, abundant muscovite mica, plagioclase	Hematite stained matrix	Compacted to bioturbated

HAR COL 2.5-6

Table 2.5.4-207 (Sheet 4 of 4)
Petrographic Examination Thin Section Descriptions

Borehole ID	Sample ID	Depth (ft.)	Lithology	Grain Size (mm)	Sorting	Pore Fill/ Cements	Framework Grains/Allochems	Matrix	Grain Packing/ Compaction
BGA-3	SC-5	149.65-150.85	Argillaceous, terrigeneous mudstone	0.03-0.10	Poor	Minor silica, pyrite, clay matrix	Monocrystalline quartz, abundant muscovite mica, plagioclase, schistose and gneissic metamorphic rock fragments	Clay matrix, fine muscovite	Crudely laminated to bioturbated

HAR COL 2.5-6
HAR COL 2.5-12

Table 2.5.4-208 (Sheet 1 of 3)
Soil Index Test Results

Borehole ID	Sample ID	Depth (ft. bgs.)		Elevation (ft.)		Dry Unit Weight (lb/ft^3)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Group Symbol	Particle Size Analysis (%P200)	Specific Gravity	Moisture Content (%)
Surficial Soil Samples													
BPA-1	PT-2	7.0	9.0	254.5	252.5	132	31	15	16	SC	14	NA	8
BPA-2	PT-1	11.0	13.0	248.6	246.6	137	30	17	13	SC	44	2.74	7
BPA-7	PT-1	5.0	7.0	259.8	257.8	140	31	19	12	CL	82	2.77	7
BPA-12	PT-1	5.0	7.0	258.9	256.9	146	35	20	15	SC	48	2.77	6
BPA-20	PT-1	2.0	4.0	259.5	257.5	127	33	18	15	CL	92	2.74	12
BPA-20	PT-2	5.0	7.0	256.5	254.5	141	38	20	18	Sandy CL	67	NA	7
BPA-21	PT-3	9.0	11.0	252.5	250.5	111	36	19	17	CL	87	NA	17
BPA-23	PT-2 (Upper)	5.5	6.5	254.4	253.4	NA [(a)]	21	16	5	SC-SM	15	2.64	NA
BPA-23	PT-2 (Lower)	6.5	7.5	253.4	252.4	127	26	16	10	SC	39	2.71	11
BPA-24	PT-2	9.0	11.3	249.9	247.6	144	29	18	11	Sandy CL	62	2.76	7
BPA-26	PT-2	9.0	11.0	243.2	241.2	126	30	17	13	CL w/ Sand	81	2.64	16
BPA-27	PT-1	3.8	5.8	254.4	252.4	128	37	20	17	CL	99	2.75	12
BPA-28	PT-2	10.0	11.5	262.6	261.1	122	31	21	10	CL w/ Sand	77	2.72	15
BPA-31	PT-2	7.0	9.0	256.3	254.3	122	42	22	20	CL	97	2.72	16
BPA-35	PT-1	6.0	8.0	261.8	259.8	117	48	21	27	CL	94	NA	15
BPA-38	PT-1	8.5	9.5	255.9	254.9	121	28	20	8	CL w/ Sand	84	2.68	14
BPA-40	PT-1	3.0	5.0	256.2	254.2	118	27	17	10	Sandy CL	57	2.74	13
BPA-43	PT-2	7.0	9.0	262.7	260.7	112	Sample is Non-Plastic			SP-SM	7	2.60	13
BCTA-1	PT-2	6.0	8.0	254.8	252.8	124	35	21	14	SC	36	NA	11
BCTA-2	PT-3	11.0	13.0	249.9	247.9	105	53	23	30	CH w/ Sand	81	2.74	23
BCTA-4	PT-3	13.0	15.0	249.8	247.8	125	30	15	15	CL	93	2.74	12

HAR COL 2.5-6
HAR COL 2.5-12

Table 2.5.4-208 (Sheet 2 of 3)
Soil Index Test Results

Borehole ID	Sample ID	Depth (ft. bgs.)		Elevation (ft.)		Dry Unit Weight (lb/ft^3)	Atterberg Limits			Group Symbol	Particle Size Analysis (%P200)	Specific Gravity	Moisture Content (%)
							Liquid Limit	Plastic Limit	Plasticity Index				
BCTA-4	PT-4	23.0	25.0	239.8	237.8	110	35	18	17	CL w/ Sand	75	2.77	18
BCTA-5	PT-2	6.0	8.0	237.7	235.7	118	32	18	14	CL	90	2.72	16
BCTA-8	PT-1	6.0	8.0	220.7	218.7	146	27	16	11	CL	87	2.78	5
BCA-1	ST-2	11.0	13.0	245.9	243.9	104	45	21	24	CL w/ Sand	79	2.72	20
BCA-1	ST-4	27.0	29.0	229.9	227.9	101	40	20	20	CL w/ Sand	82	2.71	24
					Average:	124.2	34	19	15		68	2.72	12.9
					Minimum:	101	21	15	5		7	2.60	5
					Maximum:	146	53	23	30		99	2.78	24
					St. Dev.:	13.0	7.3	2.2	5.8		27.3	0.05	5.1
					Count:	25	25	25	25		26	21	25
Soil and Soil-like Samples Within Rock													
BPA-3	GS-1[b]	80.5	80.8	180.1	179.8	NA	33	21	12	CL w/ Sand	76	NA	11
BPA-6	GS-3[b]	87	87.2	177.3	177.1	NA	32	20	12	CL w/ Sand	NA	NA	8
BPA-6	GS-4[b]	100.8	101.1	163.5	163.2	NA	39	22	17	CL	91	NA	11
BPA-11	GS-1[b]	114.6	114.9	149.7	149.4	NA	37	22	15	Sandy CL w/ Gravel	63	NA	10
BPA-31	GS-1[b]	60	60.3	203.1	202.8	NA	31	18	13	CL w/ Sand	80	NA	8
BPA-46	GS-2[b]	53.2	53.4	214.8	214.6	NA	29	17	12	SC	46	NA	7
BGA-16	SC-3	60	60.8	200.6	199.8	142	NA	NA	NA	NA	NA	NA	7
BPA-20	SC-1	18.35	19	243.1	242.5	131	NA	NA	NA	NA	NA	NA	12
BPA-41	SC-4	122.2	122.6	142.4	142.0	133	34	20	14	Sandy CL	50	2.81	12
BPA-43	SC-1	16.3	17.2	253.4	252.5	177 [c]	NA	NA	NA	NA	NA	NA	6
BPA-43	SC-4	88.15	88.85	181.6	180.9	160 [c]	31	18	13	CL	NA	2.71	9
BPA-49	SC-3	27.1	28.5	234.8	233.4	120	34	20	14	CL w/Sand	78	NA	15

HAR COL 2.5-6
HAR COL 2.5-12

Table 2.5.4-208 (Sheet 3 of 3)
Soil Index Test Results

Borehole ID	Sample ID	Depth (ft. bgs.)		Elevation (ft.)		Dry Unit Weight (lb/ft^3)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Limit	Atterberg Limits Plasticity Index	Group Symbol	Particle Size Analysis (%P200)	Specific Gravity	Moisture Content (%)
BPA-49	SC-4	49.9	50.2	212.0	211.7	NA	31	18	13	SC	45	NA	12
BPA-50	SC-7	107.95	108.8	165.0	164.2	134	33	18	15	Sandy CL	68	NA	10
BPA-50	SC-11	29.3	29.8	243.7	243.2	110	36	20	16	CL	87	NA	16
					Average:	128.4	33	19	14		66	2.76	11.0
					Minimum:	110	31	18	13		45	2.71	6
					Maximum:	142	36	20	16		87	2.81	16
					St. Dev.:	11.3	2	1.1	1.2		18	N/A	3.3
					Count:	6	6	6	6		5	2	9

Notes:
a) NA indicates test not performed.

b) Grab soil samples (designated as “GS”) were collected from stored rock core after the field event. Corresponding test results are provided in this table, but these have not been included in the statistical calculations.

c) The test result for this parameter is unrealistically high. The corresponding result is shown, but has not been included in the statistical calculations.

HAR COL 2.5-6

Table 2.5.4-209 (Sheet 1 of 3)
Soil Strength Test Results

Borehole ID	Sample ID	Depth (ft.)		Elevation (ft.)		Strength: Deviator Stress[(a)] (ksf)	Strength: Confining Pressure[(b)] (ksf)	Type of Test[(c)]	Soil Type[(d)]	Other Notes
Above Top of Sound Rock										
BPA-2	PT-1	11.0	13.0	248.6	246.6	8.7	1.4	CU	SC	Total Stress: phi = 56.8 deg, C = 0 ksf Effective Stress: phi = 49.2 deg, C = 0 ksf
						46.6	4.1	CU		
BPA-7	PT-1	5.0	7.0	259.8	257.8	9.9	0.9	UU	CL	
						11.7	4.0	UU		
BPA-12	PT-1	5.0	7.0	258.9	256.9	16.52	0	UC	SC	
BPA-20	SC-1	18.35	19.0	243.1	242.5	3.31	0	UC	NA	
BPA-20	PT-1	2.0	4.0	259.5	257.5	2.2	0.4	UU	CL	
						4.2	2.0	UU		
						6.1	4.0	UU		
BPA-23	PT-2	6.5	7.5	253.4	252.4	13.9	1.0	UU	SC	
BPA-24	PT-2	9.0	11.3	249.9	247.6	9.2	1.3	CU	Sandy CL	Total Stress: phi = 50.2 deg, C = 0 ksf Effective Stress: phi = 50.2 deg, C = 1.5 ksf
						4.8	2.2	CU		
						28.9	4.0	CU		
BPA-26	PT-2	9.0	11.0	243.2	241.2	1.23	1.01	UU	CL w/ Sand	
						2.46	4.32	UU		
						1.8	2.16	UU		
BPA-27	PT-1	3.8	5.8	254.4	252.4	3.4	1.0	UU	CL	
						10.1	2.2	UU		
						11.1	4.3	UU		
BPA-28	PT-2	10.0	11.5	262.6	261.1	8.2	1.44	UU	CL w/ Sand	

HAR COL 2.5-6

Table 2.5.4-209 (Sheet 2 of 3)
Soil Strength Test Results

Borehole ID	Sample ID	Depth (ft.)		Elevation (ft.)		Strength: Deviator Stress[a] (ksf)	Strength: Confining Pressure[b] (ksf)	Type of Test[c]	Soil Type	Other Notes
BPA-31	PT-2	7.0	9.0	256.3	254.3	3.39	1.01	UU	CL	
						1.76	4.32	UU		
BPA-38	PT-1	8.5	9.5	255.9	254.9	15	1.2	UU	CL w/ Sand	
BPA-40	PT-1	3.0	5.0	256.2	254.2	1.4	1.1	CU	Sandy CL	Total Stress: phi = 27.7 deg, C = 0 ksf Effective Stress: phi = 34.0 deg, C = 0 ksf
						7.7	4.0	CU		
BPA-43	PT-2	7.0	9.0	262.7	260.7	5.35	1.01	UU	SP-SM	
BCA-1	ST-2	11.0	13.0	245.9	243.9	2.11	0	UC	CL w/ Sand	
BCA-1	ST-4	27.0	29.0	229.9	227.9	1.44	0	UC	CL w/ Sand	
BCTA-2	PT-3	11.0	13.0	249.9	247.9	4.67	0	UC	CH w/ Sand	
BCTA-4	PT-3	13.0	15.0	249.8	247.8	1.6	0	UC	CL	
BCTA-4	PT-4	23.0	25.0	239.8	237.8	1.78	0	UC	CL w/ Sand	
BCTA-5	PT-2	6.0	8.0	237.7	235.7	6.75	0	UC	CL	
BCTA-8	PT-1	6.0	8.0	220.7	218.7	22.42	0	UC	CL	

HAR COL 2.5-6

Table 2.5.4-209 (Sheet 3 of 3)
Soil Strength Test Results

Borehole ID	Sample ID	Depth (ft.)		Elevation (ft.)		Strength: Deviator Stress[a] (ksf)	Strength: Confining Pressure[b] (ksf)	Type of Test[c]	Soil Type	Other Notes
Below Top of Rock										
BPA-41	SC-4	122.2	122.6	142.4	142.0	1.61	0	UC	Sandy CL	
BPA-43	SC-1	16.3	17.2	253.4	252.5	81.39	0	UC	NA	
BPA-43	SC-4	88.2	88.9	181.6	180.9	2.0	0	UC	CL	
BGA-16	SC-3	60.0	60.8	200.6	199.8	13.1	0	UC	NA	

Notes:

a) Deviator stress is the applied vertical pressure (at failure) less the confining pressure. This is twice the shearing stress on the deviatoric plane.

b) Cell pressure is listed for UU tests, Effective Confining Pressure (cell pressure – backpressure) is listed for CU tests.

c) Tests performed include the following:

CU = Consolidated Undrained
UC = Unconfined Compressive Strength
UU = Unconsolidated Undrained

d) NA indicates that soil type was not determined from index tests

HAR COL 2.5-6
HAR COL 2.5-12

Table 2.5.4-210
Soil Consolidation Test Results

Borehole ID	Sample ID	Depth (ft.)		Elevation (ft.)		Initial Void Ratio	P_c (ksf)	C_c	C_r	Soil Type
BPA-2	PT-1	11.0	13.0	248.6	246.6	0.297	2.8	0.05	0.014	SC
BPA-12[a]	*PT-1*	*5.0*	*7.0*	*258.9*	*256.9*	*0.194*	*3.9*	*0.03*	*0.010*	*SC*
BPA-24	PT-2	9.0	11.3	249.9	247.6	0.354	2.8	0.09	0.019	Sandy CL
BPA-38	PT-1	8.5	9.5	255.9	254.9	0.417	3.5	0.10	0.024	CL w/ Sand
BPA-40	PT-1	3.0	5.0	256.2	254.2	0.350	4.4	0.02	0.010	Sandy CL
BPA-43	PT-2	7.0	9.0	262.7	260.7	0.448	3.5	0.07	0.016	SP-SM

Notes:
a) Test results are not valid for this sample – the stress-strain curve for this test does not represent realistic soil behavior.

HAR COL 2.5-6

Table 2.5.4-211
Summary of Rock Dynamic Properties From Suspension Logging

Unit[(a)]	V_s[(b)] (fps)	V_p[(c)] (fps)	Poisson's Ratio
HAR 2 Results			
Average	6239	12413	0.33
Minimum	3660	9010	0.14
Maximum	7750	15080	0.42
St. Dev.	758	1069	0.03
HAR 3 Results			
Average	6073	12140	0.33
Minimum	3270	6670	0.23
Maximum	7940	14810	0.44
St. Dev.	985	1385	0.04

Notes:
a) Results for HAR 2 are from suspension logging results at BPA-5, 47, and 48. Results for HAR 3 are from suspension logging results at BPA-25, 39, 49, and 50. These results characterize rock properties between approximate elevation 60 ft. and 220 ft.

b) V_s = shear wave velocity

c) V_p = compressional wave velocity

HAR COL 2.5-7
HAR COL 2.5-9

Table 2.5.4-212
Engineering Properties of Safety-Related Structure Backfill

	As-Placed Engineering Properties[a]			
Backfill Type	**Strength Parameters[b]**	**Relative Compaction[c]**	**Soil Type (USCS)[d]**	**V_s (fps)**
Concrete Fill	28-Day Compressive Strength: 2500 psi	N/A	N/A	N/A
Compacted Granular Fill	Friction Angle: 35 deg.	95%	SW, GW	500 to 1250
Compacted Cohesive Fill	Friction Angle: 20 deg. Cohesion: 400 psf	95%	CL, ML, SM, SC	350 to 1250

Notes:
a) These engineering properties are considered representative values of the compacted backfill mass.

b) The friction angle and cohesion values listed are drained parameters.

c) Relative compaction criteria are based on the modified proctor compaction test (ASTM D 1557).

d) Unified Soil Classification System (USCS) designations are listed for compacted granular and compacted cohesive fill. Alternative types may be considered, as long as the other as-placed engineering property requirements are satisfied.

N/A = not applicable

HAR COL 2.5-6
HAR COL 2.5-10

Table 2.5.4-213
Summary of Rock Properties and Mass Strength Criteria for Bearing Capacity Analyses

	Elevation Range (ft.)		Rock Properties		Resulting Hoek-Brown Strength Parameters	
Unit and Rock Layer and Strenth Condition	**Top**	**Bottom**	**Unconfined Compressive Strength, q_u (psi)**	**GSI**	**Cohesion (psi)**	**Friction Angle (deg.)**
HAR 2						
All Depths – Average	220	60	7032	68	110	44
All Depths – Lower Bound[(a)]	220	60	3544	52	33	31
Lowest q_u Result[(b)]	220	60	1388	52	22	24
HAR 3[(c)]						
All Depths – Average	220	60	6532	70	118	44
All Depths – Lower Bound	220	60	2696	54	31	30
Shallow – Average	220	170	6200	65	82	42
Shallow – Lower Bound	220	170	2243	48	24	25
Deep – Average	170	60	7859	74	186	46
Deep – Lower Bound	170	60	4519	62	57	38

Notes:
a) “Lower Bound” q_u and geologic strength index (GSI) correspond to one standard deviation below the mean value for data from the listed HAR Unit and Elevation Range.

b) The lowest q_u test result from all rock samples below elevation 220 ft. at HAR 2 and HAR 3 (encountered at BPA-6 at HAR 2, which is not under the nuclear island) was considered along with the “lower bound” GSI for HAR 2. This represents an unrealistically low rock mass strength condition for HAR 2 and HAR 3, which is used as an assessment of sensitivity of the bearing capacity methods. This condition is not used in calculation of allowable bearing pressures.

c) Due to differences in rock mass properties in the 50 ft. below the nuclear island subgrade elevation at HAR 3 relative to deeper rock, two depth ranges were considered in the bearing capacity analyses for HAR 3. Conditions at HAR 2 are more consistent below the nuclear island subgrade elevation, so one layer was considered for HAR 2.

HAR COL 2.5-10

Table 2.5.4-214
Summary of Bearing Capacity Analyses at Nuclear Islands – Static and Dynamic Loading

Loading and Rock Strength Conditions for Analyses [a]							Ultimate Bearing Capacity Analysis Results [b]					
							General Bearing Capacity Method (USACE 1994)		Alternate Method 1 - AASHTO (2002)		Alternate Method 2 - Wyllie (1999)	
Unit	Load Condition	Rock Elevation Interval	Strength Condition	Cohesion (psi)	Friction Angle (deg.)	Unit Weigh t (pcf)	Ultimate Bearing Capacity (ksf)	Factor of Safety [c]	Ultimate Bearing Capacity (ksf)	Factor of Safety [c]	Ultimate Bearing Capacity (ksf)	Factor of Safety [c]
2	Static	All	Average	110	43.7	160	905	102	1620	182	460	52
2	Static	All	Lower-bound [d]	33	31	160	162	18.2	163	18	111	13
3	Static	All	Average	118	44	160	969	109	301	34	490	55
3	Static	All	Lower-bound	31	29.9	160	142	16.1	124	14	101	11
3	Static	Shallow	Lower-bound [e]	24	25.3	160	88	9.8	103	12	69	8
2	Dynamic	All	Average	110	43.7	160	793	23	1620	46	460	13.1
2	Dynamic	All	Lower-bound [d]	33	31.0	160	137	3.9	163	4.7	111	3.2
3	Dynamic	All	Average	118	44	160	852	24	301	8.6	490	14.0
3	Dynamic	All	Lower-bound	31	29.9	160	121	3.5	124	3.5	101	2.9
3	Dynamic	Shallow	Lower-bound [e]	24	25.3	160	74	2.1	103	3.0	69	2.0

Notes:
a) Refer to Table 2.5.4-213 for descriptions of rock strength conditions.

b) The general bearing capacity method (USACE 1994) is considered the primary method to evaluate bearing capacity. The alternative methods 1 and 2 are presented as confirmation of the primary method.

c) Factor of safety for static and dynamic load conditions are calculated as ultimate bearing capacity divided by 8.6 ksf and 35 ksf, respectively.

d) This strength condition is considered conservatively representative of HAR 2 rock mass conditions. Allowable bearing pressures are calculated based on these conditions.

e) This strenth condition is considered conservatively representative of HAR 3 rock mass conditions. Allowable bearing pressures are calculated based on these conditions.

HAR COL 2.5-12
HAR COL 2.5-16

Table 2.5.4-215
Estimated Elastic and Consolidation Settlement under Nuclear Islands

		Estimates of Elastic Settlement (in.)			Range in Estimated Total Settlement (in.) [(c)]	
Borehole	**Location**	**Based on Seismic Survey Data [(a)]**	**Based on PMT Data [(b)]**	**Estimated Recompression Settlement of Clay Seams (in.)**	**Best Estimate**	**Upper Bound**
BPA-3	HAR 2 - West Side	0.04	0.1	0.4	0.4	0.5
BPA-5	HAR 2 - Internal	0.04	0.1	0.0	0.0	0.1
BPA-7	HAR 2 - North Side	0.04	0.1	0.0	0.0	0.1
BPA-8	HAR 2 - South Side	0.04	0.1	0.2	0.2	0.3
BPA-41	HAR 2 - East Side	0.04	0.1	0.2	0.2	0.3
BPA-23	HAR 3 - West Side	0.04	0.2	0.2	0.2	0.4
BPA-25	HAR 3 - Internal	0.04	0.2	0.3	0.3	0.5
BPA-27	HAR 3 - North Side	0.04	0.2	0.3	0.3	0.5
BPA-28	HAR 3 - South Side	0.04	0.2	0.2	0.2	0.4
BPA-43	HAR 3 - East Side	0.04	0.2	0.1	0.1	0.3

Notes:
a) Elastic settlement based on seismic wave velocity (suspension logging survey) data is approximately 0.03 to 0.04 in. at boreholes at HAR 2 and HAR 3. A value of 0.04 in. is assigned to other boreholes at HAR 2 and HAR 3 as shown, at which suspension logging surveys were not performed.

b) Elastic settlement based on PMT results are considered conservative, bounding values. For the HAR 2 boreholes in which PMT was not performed, elastic settlement is assumed to be 0.1 in., based on results of PMT tests at BPA-3 and BPA-41. For HAR 3, elastic settlement is assumed to be 0.2 in. based on results at BPA-23 and BPA-43.

c) The "Best Estimate" of settlement considers elastic settlements based on seismic survey data. The "Upper Bound" estimate considers elastic settlements based on PMT results.

Table 2.5.4-216
Estimated Differential Settlement Under Nuclear Islands

Borehole Combination		Differential Settlement Location	Total Estimated Settlement at 1st Borehole (in.)		Total Estimated Settlement at 2nd Borehole (in.)		Distance Between Boreholes (ft.)	Range in Differential Settlement (Slope) [a]	
First Borehole	Second Borehole		Best Estimate	Upper Bound	Best Estimate	Upper Bound		Best Estimate	Upper Bound
Based on Borehole-Specific Settlement Results:									
BPA-3	BPA-41	HAR 2, West-East	0.4	0.5	0.2	0.3	255	0.00007	0.00007
BPA-7	BPA-8	HAR 2, North-South	0.0	0.1	0.2	0.3	149	0.00011	0.00011
BPA-23	BPA-43	HAR 3, West-East	0.2	0.4	0.1	0.3	263	0.00003	0.00003
BPA-27	BPA-28	HAR 3, North-South	0.3	0.5	0.2	0.4	153	0.00005	0.00005
Based on Maximum Settlement Range Across Shortest Dimension of Nuclear Island: [b]									
North	South	HAR 2, North-South	0.4	0.5	0.0	0.1	87.5	0.00039	0.00038
North	South	HAR 3, North-South	0.3	0.5	0.1	0.3	87.5	0.00019	0.00019

Notes:
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.

b) The range of estimated settlement for all boreholes considered at each HAR unit were considered by assigning the highest estimated settlement for the boreholes at each site for the "First Borehole", and lowest settlement for the "Second Borehole". The short dimension of the nuclear island (87.5 ft.) is the "Plant South" edge (west edge of nuclear island at HAR 2 and HAR 3).

HAR COL 2.5-11

Table 2.5.4-217
Estimated Lateral Earth Pressures on Nuclear Island Sidewalls

Elevation (ft.)	At-Rest Earth Pressure (ksf)	
	Compacted Granular Fill	Compacted Cohesive Fill
260	0	0
258	0.1	0.2
250	0.8	1.0
240	1.8	2.1
230	2.7	3.2
220 [a]	3.6	4.2

Notes:

a) The bottom of the nuclear island basemat will be at elevation 221.5 ft., with excavation extending to approximate elevation 220 ft. The actual elevation of the bottom of the soil backfill against the nuclear island sidewalls (transition to concrete fill) will vary by location.

Table 2.5.4-218
Results of Sliding Stability Evaluations

Case ID	Plant Direction of Seismic Loading	Discontinuity Set 1 Strength Parameters			Discontinuity Set 2 Strength Parameters			Depth to Sound Rock (below site grade)		Adjacent Building Surcharge	Critical	Net Passive Resistance	Net Base Sliding Resistance	Inertial Sliding Force	Net Active Force	Resulting	Net Passive Force <u>Required</u> for FS_{slide} = 1.1	
		β_1	ϕ_{1r}	c_{1r}	β_2	ϕ_{2r}	c_{2r}	Active Side D_{s1}	Passive Side D_{s2}	S_s	B_1	$R_{passive}$	R_{base}	$F_{lat.struct}$	F_{active}	FS_{slide}	Required Passive Force	Percent of $R_{passive}$
		(deg)	(deg)	(psf)	(deg)	(deg)	(psf)	(ft)	(ft)	(psf)	(ft)	(kips x 10^3)	(kips x 10^3)	(kips x 10^3)	(kips x 10^3)		(kips x 10^3)	(%)
HAR 2																		
HAR2-N1	North	8	0	1500	45.6	45	0	8	11	520	0	946.0	104.4	50.3	8.3	17.9	0.0	0%
HAR2-N2	North	8	0	1500	45.6	45	0	8	11	520	0	946.0	104.4	50.3	8.3	17.9	0.0	0%
HAR2-E1	East	3.9	0	1500	63.4	45	0	15	6	0	0	1073.0	104.4	60.6	14.0	15.8	0.0	0%
HAR2-E2	East	3.9	0	1500	63.4	45	0	15	6	0	0	1073.0	104.4	60.6	14.0	15.8	0.0	0%
HAR2-S1	South	45.6	45	0	8	0	1500	11	8	1700	0	49.4	76.0	50.3	11.7	2.03	0.0	0%
HAR2-S2	South	45.6	45	0	8	0	1500	11	8	1700	0	49.4	76.0	50.3	11.7	2.03	0.0	0%
HAR2-W1	West	63.4	45	0	3.9	0	1500	6	15	1700	0	175.6	62.8	60.6	24.0	2.82	30.2	17%
HAR2-W2	West	63.4	45	0	3.9	0	1500	6	15	1700	0	175.6	62.8	60.6	24.0	2.82	30.2	17%
HAR 3																		
HAR3-N1	North	15.8	0	1500	54.5	45	0	27	17	520	0	652.5	104.4	50.3	8.3	12.9	0.0	0%
HAR3-N2	North	15.8	0	1500	54.5	45	0	27	17	520	0	652.5	104.4	50.3	8.3	12.9	0.0	0%
HAR3-E1	East	17.8	0	1500	48.6	45	0	31	7	0	0	1811.0	104.4	60.6	14.0	25.7	0.0	0%
HAR3-E2	East	17.8	0	1500	48.6	45	0	31	7	0	0	1811.0	104.4	60.6	14.0	25.7	0.0	0%
HAR3-S1	South	54.5	45	0	15.8	0	1500	17	27	1700	15	40.5	86.6	50.3	11.7	2.05	0.0	0%
HAR3-S2	South	54.5	45	0	15.8	0	1500	17	27	1700	0	27.2	104.4	50.3	11.7	2.13	0.0	0%
HAR3-W1	West	48.6	45	0	17.8	0	1500	7	31	1700	15	70.6	77.8	60.6	24.0	1.75	15.3	22%
HAR3-W2	West	48.6	45	0	17.8	0	1500	7	31	1700	0	49.6	104.4	60.6	24.0	1.82	0.0	0%

2.5.5 STABILITY OF SLOPES

HAR COL 2.5-14

The nominal site grade at the HAR site will be constructed at 79.6 m (261 ft.). The actual grade will be lower by a foot or two (approximate elevation 79.3 m [260 ft.]) to accommodate grading, drainage, and local site flooding requirements to a distance of at least 152 m (500 ft.) in each direction from nuclear island structures, as shown on Figure 2.4.1-205. No permanent slopes will be present at the site that could adversely affect safety of the nuclear island.

HAR COL 2.5-15

No manmade earth or rock dams are present on-site that could adversely affect the safety of HAR 2 or HAR 3. Potential failure of off-site dams is addressed in Subsection 2.4.4.

HAR COL 2.5-14

Construction slope stability for the nuclear island excavations has been evaluated as discussed in Subsection 2.5.4.5. The nuclear island excavation sideslopes presented on Figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B will provide adequate stability during construction. Upon construction and placement of backfill along the nuclear island sidewalls, the excavation slopes will have no potential to adversely affect the nuclear islands.

STD DEP 1.1-1

2.5.6 COMBINED LICENSE INFORMATION

2.5.6.1 Basic Geologic and Seismic Information

HAR COL 2.5-1

This COL item is addressed in Subsections 2.5.1 and 2.5.4.1.

2.5.6.2 Site Seismic and Tectonic Characteristics Information

HAR COL 2.5-2

This COL item is addressed in Subsections 2.5.2, 2.5.4.7, and 2.5.4.9.

2.5.6.3 Geoscience Parameters

HAR COL 2.5-3

This COL item is addressed in Subsections 2.5.2.6 and 2.5.4.11.

2.5.6.4 Surface Faulting

HAR COL 2.5-4

This COL item is addressed in Subsection 2.5.3.

2.5.6.5 Site and Structures

HAR COL 2.5-5 This COL item is addressed in Subsections 2.5.4.1, and 2.5.4.3.

2.5.6.6 Properties of Underlying Materials

HAR COL 2.5-6 This COL item is addressed in Subsections 2.5.4.2, 2.5.4.3, 2.5.4.4, 2.5.4.6, 2.5.4.7, and 2.5.4.10.2.

2.5.6.7 Excavation and Backfill

HAR COL 2.5-7 This COL item is addressed in Subsections 2.5.4.5, 2.5.4.10.4, and 2.5.4.6.

2.5.6.8 Groundwater Conditions

HAR COL 2.5-8 This COL item is addressed in Subsection 2.5.4.6.

2.5.6.9 Liquefaction Potential

HAR COL 2.5-9 This COL item is addressed in Subsection 2.5.4.8.

2.5.6.10 Bearing Capacity

HAR COL 2.5-10 This COL item is addressed in Subsection 2.5.4.10.

2.5.6.11 Earth Pressures

HAR COL 2.5-11 This COL item is addressed in Subsections 2.5.4.10.4 and 2.5.4.11.

2.5.6.12 Static and Dynamic Stability of Facilities

HAR COL 2.5-12 This COL item is addressed in Subsection 2.5.4.10.3.

2.5.6.13 Subsurface Instrumentation

HAR COL 2.5-13 This COL item is addressed in Subsection 2.5.4.10.3.7.

2.5.6.14 Stability of Slopes

HAR COL 2.5-14 This COL item is addressed in Subsection 2.5.5.

2.5.6.15 Embankments and Dams

HAR COL 2.5-15 This COL item is addressed in Subsections 2.4.4 and 2.5.5.

2.5.6.16 Settlement of Nuclear Island

HAR COL 2.5-16 This COL item is addressed in Subsection 2.5.4.10.3.

2.5.6.17 Waterproofing System

HAR COL 2.5-17 This COL item is addressed in FSAR Subsection 14.3.3.1.

2.5.7 REFERENCES

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APPENDIX 2AA
EARTHQUAKE CATALOG

HAR COL 2.5-1
HAR COL 2.5-2

The updated earthquake catalog prepared for the project constitutes this appendix. The development of this catalog is described in Subsection 2.5.2. This catalog was used to select the final catalog of earthquakes occurring within 320 km (200 mi.) of the HAR 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 for bias due to uncertainties

m_b – Body-wave magnitude

Type – Category for earthquakes:

- EPRI, from EPRI-SOG (1988).
- Added Historical, newly identified earthquakes added to EPRI-SOG catalog (occurring from 1776 to February 1985).
- Post, earthquakes occurring post-EPRI-SOG catalog (May, 1985 to April, 2008).

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 HAR site in km

M – Moment magnitude

M_o – Seismic moment

APPENDIX 2BB
GEOTECHNICAL BORING LOGS

HAR COL 2.5-1
HAR COL 2.5-5
HAR COL 2.5-6

This appendix contains geotechnical boring logs that are the basis for discussion in relevant sections of Section 2.5. The logs are of soil and rock borings and represent a record of subsurface conditions at the HAR site.

The appendix contains the logs of 83 bore holes.