CHAPTER 2

SITE CHARACTERISTICS

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ACRONYMS AND ABBREVIATIONS

°F	degrees Fahrenheit
ΔT	vertical temperature difference
χ/Q	relative concentration, in sec/m^3
AADT	annual average daily traffic
ACFT	acre feet
ac-ft	acre feet
AF	amplification factor
AFB	Air Force Base
ALOHA	Areal Locations of Hazardous Atmospheres
AMRT	Average Mean Residence Time
ANL	Argonne National Laboratory
ANSS	Advanced National Seismic System
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating and Air-Conditioning Engineers
BB	Broad-Banded
BE	Best Estimate
bgs	below ground surface
BIS	Banks Information Solutions Inc.
BRA	Brazos River Authority
BRM	Brazos River Mile
BTS	Bureau of Transportation Statistics
CAV	Cumulative Absolute Velocity
C_d	overtopping discharge coefficient
CERI	Center for Earthquake Research and Information

ACRONYMS AND ABBREVIATIONS

CEUS	Central and Eastern United States
CFR	Code of Federal Regulations
cfs	cubic feet per second
CG	cloud-to-ground
CH	Fat Clay
CL	Lean Clay
CO_2	carbon dioxide
COC	Chamber of Commerce
COL	Combined Operating License
COLA	Combined Operating License Application
CoV	coefficient of variation (standard deviation/mean)
cm/sec	centimeters per second
Cp	peaking coefficient
CPNPP	Comanche Peak Nuclear Power Plant
CPSES	Comanche Peak Steam Electric System
CPT	cone penetration test
CSDRS	Certified Site Design Response Spectra
Ct	lag coefficient
CU	Consolidated-Undrained
D	Diameter
DCD	Design Control Document
DF	design factor
DFW	Dallas-Fort Worth
EAB	exclusion area boundary
EGC	Exelon Generation Company
Emb	Best Estimate Body-Wave Magnitude

ACRONYMS AND ABBREVIATIONS

EOF	emergency operation facility
EPA	U.S. Environmental Protection Agency
EPRI	Electric Power Research Institute
EPRI-SOG	Electric Power Research Institute Seismicity Owners
ER	Environmental Report
ESP	Early Site Permit
EST	Earth Science Team
ESWS	essential service water system
ETP	Energy Transfer Partners
ETR	energy transfer ratio
FEMA	Federal Emergency Management Agency
FIRS	Foundation Input Response Spectra
FSAR	Final Safety Analysis Report
ft	feet
g/cc	grams per cubic centimeter
G/G_{max}	dynamic shear modulus reduction
GMRS	Ground Motion Response Spectra
gpd	gallons per day
gpm	gallons per minute
GSI	Geologic Strength Index
HF	high frequencies
HiRAT	High Resolution Acoustic Televiewer
HMR	Hydrometeorological Report
hr	hour
HSA	Hollow Stem Auger
HVAC	heating, ventilating, and air conditioning

ACRONYMS AND ABBREVIATIONS

HW_r	head water elevation for the weir
IDLH	Immediately Dangerous to Life and Health
IFR	instrument flight rules
in	inch(es)
in/in	inch per inch
$I_{S(50)}$	point load strength index
ISRM	International Society for Rock Mechanics
JRB	Joint Reserve Base
K_h	hydraulic conductivities
ksi	kips per square inch
kW	kilowatts
L	Length
LB	Lower Bound
LEL	lower explosive limit
LF	low frequencies
LL	liquid limits
LLNL	Lawrence Livermore National Library
LMDCT	Linear Mechanical Draft Cooling Towers
LPZ	low population zone
Luminant	Luminant Generation
LVDT	Linear Variable Displacement Transducer
m^2	square meters
MAOP	maximum allowable operating pressure
M_b	body wave magnitudes
MCR	Main Control Room

ACRONYMS AND ABBREVIATIONS

M_d	duration or coda magnitude
METSYS	Meteorological System
mGal	milligals
Mgd	millions of gallons per day
M_L	local magnitude
M_{max}	maximum earthquake magnitude
MMI	modified Mercalli intensities
MOA	military operations area
mph	miles per hour
M_s	surface-wave magnitude
m/s	meters per second
m/sec	meters per second
msl	mean sea level
MW	megawatts
MW	monitoring well
Mw	moment magnitude
NAS	Naval Air Station
NCDC	National Climatic Data Center
NCTCOG	North Central Texas Council of Governments
NEIC	National Earthquake Information Center
NESDIS	National Environmental Satellite, Data, and Information Service
NFPA	National Fire Protection Association
NIBS	National Institute of Building Sciences
NIOSH	National Institute for Safety and Health
NLDN	National Lightning Detection Network

ACRONYMS AND ABBREVIATIONS

NMSZ	New Madrid Seismic Zone
NMT	New Mexico Institute of Mining and Technology
NOAA	National Oceanic and Atmospheric Administration
NRC	U.S. Nuclear Regulatory Commission
NRHM	non-radioactive hazardous materials
nT	nanoTeslas
NTAD	National Transportation Atlas Database
OBE	Operating Basis Earthquake
OCR	over-consolidation ratio
ODCM	Off-site Dose Calculation Manual
OGS	Oklahoma Geological Survey
OSHA	Occupational and Safety Health Administration
Pa	probability of activity
pcf	pounds per cubic foot
PDE	Preliminary Determination of Epicenters
PEL	Permissible Exposure Limit
PFD	Process Flow Diagram
PI	plasticity index
PGA	peak ground acceleration
PLI	Point Load Strength Index
PMF	Probable Maximum Flood
PMH	probable maximum hurricane
PMP	probable maximum precipitation
PMT	Pressuremeter tests
PMWP	Probable Maximum Winter Precipitation
PMWS	probable maximum windstorm

ACRONYMS AND ABBREVIATIONS

ppm	parts per million
PSHA	Probabilistic Seismic Hazard Analysis
psi	pounds per square inch
P-wave	compressional-wave
q_u	failure load
RG	Regulatory Guide
RGR	Rio Grande Rift
RMR	Rock Mass Rating
RQD	rock quality designations
RRC	Railroad Commission of Texas
SA	spectral acceleration
SACTI	Seasonal/Annual Cooling Tower Impact
SCR	Squaw Creek Reservoir
SCS	Soil Conservation Service
SES	Steam Electric Station
SH	State Highway
Smb	Estimate of Standard Deviation of Magnitude
SP	Spontaneous Potential
SPR	Single Point Resistance
SPT	Standard Penetration Test
SRCC	Southern Regional Climate Center
SSE	Safe Shutdown Earthquake
SSHAC	Senior Seismic Hazard Analysis Committee
SSI	Safe Shutdown Impoundment
SSI	Soil Structure Interaction
STORET	Storage and Retrieval

ACRONYMS AND ABBREVIATIONS

SWATS	Surface Water and Treatment System
S-wave	shear-wave
T_c	time of concentration
TCEQ	Texas Commission on Environmental Quality
TNT	trinitrotoluene
TR	Technical Release
TSC	technical support center
TSDC	Texas State Data Center and Office of the State Demographer
tsf	tons/ft^2
TVA	Tennessee Valley Authority
TWDB	Texas Water Development Board
TxDOT	Texas Department of Transportation
TXU	Texas Utilities Generation Company
U1-PC	U1 Plant Computer
U2-PC	U2 Plant Computer
U_{10}	wind speed at 10 meters above plant grade, in m/sec
UC	Unconfined Compression
UHRS	uniform hazard response spectra
UHS	ultimate heat sink
U.S.	United States
USACE	U.S. Army Corps of Engineers
USAPWR	United States Advanced Pressurized Water Reactor
USGS	U.S. Geological Survey
UTC	Universal Time Coordinated
UU	Unconsolidated-Undrained

ACRONYMS AND ABBREVIATIONS

V	velocity
W_b	width of breach (ft)
W_E	effective charge weight
W_{EXP}	weight of the explosive in question
Wts	weighting
WWTP	Waste Water Treatment Plant
Y_O	initial depth

2.0 SITE CHARACTERISTICS

This section of the referenced Design Control Document (DCD) is incorporated by reference with the following departures and/or supplements.

CP SUP 2.0(1) Add the following after the third paragraph of DCD Section 2.0.

Chapter 2 describes the characteristics of the CPNPP site. The site location and description are provided in sufficient detail to support a safety assessment. This chapter is divided into five sections:

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

In this chapter, the following definitions and figures are provided to assist the reader in understanding the scope of the discussion:

- CPNPP site - the 7950-ac area identified by the site boundary (Figure 2.1-201).
- CPNPP vicinity - the area within approximately the 6-mi radius around the site (Figure 2.1-202).
- CPNPP region - the area within approximately the 50-mi radius around the site (Figure 2.1-203).

Table 2.0-1R provides a comparison of site-related design parameters for which the US-APWR is designed and site characteristics to CPNPP in support of this safety assessment.

Table 2.0-1R (Sheet 1 of 13)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.4(1)
CP COL 2.5(1)

Meteorology		
Parameter Description	Parameter Value	
	DCD	CPNPP 3 and 4
Normal winter precipitation roof load(11)	50 lb/ft^2	11.7 lb/ft^2
Extreme winter precipitation roof load(12)	75 lb/ft^2	37.8 lb/ft^2
48-hr probable maximum winter precipitation (PMWP)	36 in	31 in
Tornado maximum wind speed	230 mph	230 mph
	184 mph maximum rotational	184 mph maximum rotational
	46 mph maximum translational	46 mph maximum translational
Radius of maximum rotational speed	150 ft	150 ft
Rate of Pressure drop	0.5 psi/s	0.5 psi/s
Tornado maximum pressure drop	1.2 psi	1.2 psi
Tornado-generated missile spectrum and associated velocities	15 ft long schedule 40 steel pipe moving horizontally at 135 ft/s(1)	15 ft long schedule 40 steel pipe moving horizontally at 135 ft/s(1)
	4000 lb automobile moving horizontally at 135 ft/s(1)	4000 lb automobile moving horizontally at 135 ft/s(1)
	1 in diameter steel sphere moving horizontally at 26 ft/s(1)	1 in diameter steel sphere moving horizontally at 26 ft/s(1)
Extreme wind speed (other than in tornado)	155 mph for 3-second gusts at 33 ft aboveground level based on 100-year return period, with importance factor of 1.15 for seismic category I and II structures	96 mph for-3-second gust wind speed at 33-ft aboveground based on 100-year return period

Table 2.0-1R (Sheet 2 of 13)
Key Site Parameters

CP COL 2.1(1) CP COL 2.2(1) CP COL 2.3(1) CP COL 2.3(2)	Ambient design air temperature	1% exceedance maximum: 100°F dry bulb, 77°F coincident wet bulb, 81°F non-coincident wet bulb	1% exceedance maximum: 99°F dry bulb, 75°F coincident wet bulb, 78°F non-coincident wet bulb
		0% exceedance maximum: 115°F dry bulb, 80°F coincident wet bulb, 86°F non-coincident wet bulb, historical limit excluding peaks <2 hr	0% exceedance maximum: 112°F dry bulb, 78°F coincident wet bulb, 83°F non-coincident wet bulb, historical limit excluding peaks <2 hr 100-year return period maximum: 115°F dry bulb, 78°F coincident wet bulb 86°F non-coincident wet bulb
CP COL 2.3(3) CP COL 2.4(1)	Ambient design air temperature	1% exceedance minimum: -10°F dry bulb	1% exceedance minimum: 25°F dry bulb
CP COL 2.5(1)		0% exceedance minimum: -40°F dry bulb, historical limit excluding peaks <2 hr	0% exceedance minimum: -0.5°F dry bulb, historical limit excluding peaks <2 hr 100-year return period minimum: -5°F dry bulb
	Atmospheric dispersion factors (χ/Q values) for on-site locations:		
	Exclusion area boundary (EAB) 0-2 hrs	5.0×10^{-4} s/m^3	3.70×10^{-4} s/m^3
	EAB annual average	1.6×10^{-5} s/m^3	5.5×10^{-6} s/m^3
	Atmospheric dispersion factors (χ/Q values) for off-site locations:		

Table 2.0-1R (Sheet 3 of 13)
Key Site Parameters

	Low-population zone (LPZ) boundary 0-8 hrs 8-24 hrs 1-4 days 4-30 days		2.1×10^{-4} s/m^3 1.3×10^{-4} s/m^3 6.9×10^{-5} s/m^3 2.8×10^{-5} s/m^3	2.29×10^{-5} s/m^3 1.49×10^{-5} s/m^3 6.34×10^{-6} s/m^3 2.01×10^{-6} s/m^3
	Food production area annual average		5.0×10^{-6} s/m^3	Not calculated as a single value. Annual average χ/Q values provided as a function of distance and direction out to a 50-mile distance.
CP COL 2.1(1)	*Deposition factor (D/Q value) for on-site and off-site locations:*			
CP COL 2.2(1) CP COL 2.3(1)	EAB annual average		4.0×10^{-8} 1/m^2	5.5×10^{-8} 1/m^2
CP COL 2.3(2) CP COL 2.3(3)	*Atmospheric dispersion factors (χ/Q values) for main control room (MCR) heating, ventilation, and air conditioning (HVAC) intake for specified release points(2):*			
CP COL 2.4(1) CP COL 2.5(1)	Plant vent(5) 0-8 hrs 8-24 hrs 1-4 days 4-30 days	1.1×10^{-3} s/m^3 6.6×10^{-4} s/m^3 4.2×10^{-4} s/m^3 2.8×10^{-4} s/m^3	East HVAC Intake 0 – 2 hours: 6.3E-04 2 – 8 hours: 4.1E-04 8 – 24 hours: 1.7E-04 1 – 4 days: 1.1E-04 4 – 30 days: 9.0E-05	West HVAC Intake 0 – 2 hours: 9.4E-04 2 – 8 hours: 7.3E-04 8 – 24 hours: 3.1E-04 1 – 4 days: 1.9E-04 4 – 30 days: 1.6E-04

Table 2.0-1R (Sheet 4 of 13)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

Ground-level containment releases(4)		East HVAC Intake Containment Shell		West HVAC Intake Containment Shell	
0-8 hrs	2.2×10^{-3} s/m^3	0 – 2 hours	7.5E-04	0 – 2 hours	8.7E-04
8-24 hrs	1.3×10^{-3} s/m^3	2 – 8 hours	5.1E-04	2 – 8 hours	6.1E-04
1-4 days	8.3×10^{-4} s/m^3	8 – 24 hours	2.2E-04	8 – 24 hours	2.7E-04
4-30 days	5.5×10^{-4} s/m^3	1 – 4 days	1.4E-04	1 – 4 days	1.7E-04
		4 – 30 days	1.2E-04	4 – 30 days	1.4E-04

Table 2.0-1R (Sheet 5 of 13)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

Main steam relief valve and safety valve releases(6) 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 5.3×10^{-3} s/m^3 3.1×10^{-3} s/m^3 2.0×10^{-3} s/m^3 1.3×10^{-3} s/m^3	East HVAC Intake Main Steam Relief Valves 0 – 2 hours: 2.9E-03 2 – 8 hours: 1.7E-03 8 – 24 hours: 6.9E-04 1 – 4 days: 4.9E-04 4 – 30 days: 3.9E-04	West HVAC Intake Main Steam Relief Valves 0 – 2 hours: 3.4E-03 2 – 8 hours: 2.4E-03 8 – 24 hours: 9.9E-04 1 – 4 days: 6.6E-04 4 – 30 days: 4.5E-04
		East HVAC Intake Main Steam Safety Valves 0 – 2 hours: 3.3E-03 2 – 8 hours: 1.9E-03 8 – 24 hours: 7.6E-04 1 – 4 days: 5.4E-04 4 – 30 days: 3.8E-04	West HVAC Intake Main Steam Safety Valves 0 – 2 hours: 4.1E-03 2 – 8 hours: 2.7E-03 8 – 24 hours: 1.1E-03 1 – 4 days: 8.1E-04 4 – 30 days: 5.1E-04

Table 2.0-1R (Sheet 6 of 13)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

Steam line break releases(8)		East HVAC Intake Main Steam Line		West HVAC Intake Main Steam Line	
0-8 hrs	1.9×10^{-2} s/m^3	0 – 2 hours	1.6E-02	0 – 2 hours	6.6E-03
8-24 hrs	1.1×10^{-2} s/m^3	2 – 8 hours	8.3E-03	2 – 8 hours	4.3E-03
1-4 days	7.1×10^{-3} s/m^3	8 – 24 hours	3.5E-03	8 – 24 hours	1.8E-03
4-30 days	4.7×10^{-3} s/m^3	1 – 4 days	2.5E-03	1 – 4 days	1.3E-03
		4 – 30 days	1.7E-03	4 – 30 days	8.9E-04
Fuel handling area releases(7)		East HVAC Intake		West HVAC Intake	
0-8 hrs	1.1×10^{-3} s/m^3	0 – 2 hours	9.6E-04	0 – 2 hours	5.4E-04
8-24 hrs	6.4×10^{-4} s/m^3	2 – 8 hours	7.5E-04	2 – 8 hours	4.1E-04
1-4 days	4.1×10^{-4} s/m^3	8 – 24 hours	3.1E-04	8 – 24 hours	1.7E-04
4-30 days	2.7×10^{-4} s/m^3	1 – 4 days	2.0E-04	1 – 4 days	1.1E-04
		4 – 30 days	1.7E-04	4 – 30 days	7.8E-05
Atmospheric dispersion factors (χ/Q values) for MCR inleak for specified release points(3):					

Table 2.0-1R (Sheet 7 of 13)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

Plant vent[9] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.3×10^{-3} s/m^3 7.8×10^{-4} s/m^3 4.9×10^{-4} s/m^3 3.3×10^{-4} s/m^3	Bounded by the χ/Q values calculated for the Main Control Room HVAC See plant vent to Main Control Room intake (above)[13]
Plant vent[10] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.4×10^{-3} s/m^3 8.0×10^{-4} s/m^3 5.1×10^{-4} s/m^3 3.3×10^{-4} s/m^3	Bounded by the χ/Q values calculated for the Main Control Room HVAC See plant vent to Main Control Room intake (above)[13]
Ground-level containment releases[4] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 2.4×10^{-3} s/m^3 1.4×10^{-3} s/m^3 9.1×10^{-4} s/m^3 6.0×10^{-4} s/m^3	See ground-level containment releases to Main Control Room intake (above)[13]
Main steam relief valve and safety valve releases[6] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 5.3×10^{-3} s/m^3 3.1×10^{-3} s/m^3 2.0×10^{-3} s/m^3 1.3×10^{-3} s/m^3	See main steam relief valve and safety valve releases to Main Control Room intake (above)[13]

Table 2.0-1R (Sheet 8 of 13)
Key Site Parameters

Steam line break releases[8] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.9×10^{-2} s/m^3 1.1×10^{-2} s/m^3 7.1×10^{-3} s/m^3 4.7×10^{-3} s/m^3	See steam line break releases to Main Control Room intake (above)[13]
Fuel handling area releases[7] 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.1×10^{-3} s/m^3 6.7×10^{-4} s/m^3 4.3×10^{-4} s/m^3 2.8×10^{-4} s/m^3	See fuel handling area releases to Main Control Room intake (above)[13]
Hydrologic Engineering		
Parameter Description	**Parameter Value**	
	DCD	**CPNPP 3 and 4**
Maximum flood (or tsunami) level	1 ft below plant grade	793.66 ft msl for SCR 820.90 ft msl for a Local Intense Precipitation at units 3 and 4 site.
Maximum rainfall rate (hourly)	19.4 in/hr for seismic category I/II structures	19.0 in/hr
Maximum rainfall rate (short-term)	6.3 in/5 min for seismic category I/II structures	6.2 in/5 min
CP COL 2.1(1) CP COL 2.2(1) Maximum groundwater level	1 ft below plant grade	1 ft below plant grade

Table 2.0-1R (Sheet 9 of 13)
Key Site Parameters

CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)
CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

Geology, Seismology, and Geotechnical Engineering		
Parameter Description	Parameter Value	
	DCD	CPNPP 3 and 4
Maximum slope for foundation-bearing stratum	20° from horizontal in untruncated strata	Layer C slopes at less than 10 degree across the footprint and the site area.
Safe-shutdown earthquake (SSE) ground motion	0.3 g peak ground acceleration	The SSE is the envelope of the GMRS and the minimum earthquake requirement of 10 CFR 50 Appendix S, based on the shape of the Certified Site Design Response Spectra (CSDRS) scaled down to a PGA of 0.1 g. The CSDRS is itself a modified RG 1.60 shape formed by shifting the control points at 9 Hz and 33 Hz to 12 Hz and 50 Hz, respectively.
SSE (certified seismic design) horizontal ground response spectra	Regulatory Guide (RG) 1.60, enhanced spectra in high frequency range (see Figure 3.7.1-1)	The minimum DCD spectrum envelops all four FIRS, down to frequencies of 0.5 Hz. Values of the horizontal 10^{-5} UHRS and FIRS are shown in Table 2.5.2-229 for the seven spectral frequencies.
SSE (certified seismic design) vertical ground response spectra	RG 1.60, enhanced spectra in high frequency range (see Figure 3.7.1-2)	For vertical FIRS motions, the same considerations used for the GMRS were used for the FIRS. That is, as a conservative assumption the V/H ratio for the FIRS spectra is assumed to be equal to the V/H ratio from RG 1.60.
Potential for surface tectonic deformation at site	None within the exclusion area boundary	No potential tectonic surface deformation has been identified at the site.

Table 2.0-1R (Sheet 10 of 13) Key Site Parameters

	Subsurface stability – minimum allowable static bearing capacity	15,000 lb/ft^2	The minimum allowable bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	
	Subsurface stability – minimum allowable dynamic bearing capacity, normal conditions plus SSE	60,000 lb/ft^2	The minimum allowable dynamic bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	
	Subsurface stability – minimum shear wave velocity at SSE input at ground surface	1000 ft/s	The site stratigraphy has a measured velocity in excess of 1000 ft/sec	
CP COL 2.3(1) CP COL 2.3(2)	Subsurface stability – liquefaction potential	None (for seismic category I structures)	The site strata is not prone to liquefaction	
	Settlement	Total settlement of R/B complex foundation(14)(15) 6.0 in. Differential settlement across R/B complex foundation(14)(15) 2.0 in. Maximum differential settlement between buildings(14)(16) 0.5 in. Maximum tilt of R/B complex foundation generated during operational life of the plant(14)(16) 1/2000	Maximum and differential settlement of all the seismic Category I buildings and structures including R/B, PS/B, ESWPT, UHSRS and PSFSV is less than 1/2 in.	
	Atmospheric dispersion factors (χ/Q values) for Technical Support Center (TSC) HVAC intake for specified release points(2):			
	Plant Vent(5) 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.4×10^{-3} s/m^3 8.0×10^{-4} s/m^3 5.1×10^{-4} s/m^3 3.3×10^{-4} s/m^3	0-2 hrs 0-8 hrs 8-24 hrs 1-4 days 4-30 days	1.1×10^{-3} s/m^3 6.9×10^{-4} s/m^3 2.8×10^{-4} s/m^3 2.1×10^{-4} s/m^3 1.3×10^{-4} s/m^3

Table 2.0-1R (Sheet 11 of 13)
Key Site Parameters

Ground-level containment releases(4)		0-2 hrs	8.0×10^{-4} s/m^3
0-8 hrs	1.9×10^{-3} s/m^3	0-8 hrs	5.1×10^{-4} s/m^3
8-24 hrs	1.1×10^{-3} s/m^3	8-24 hr	2.3×10^{-4} s/m^3
1-4 days	7.2×10^{-4} s/m^3	1-4 days	1.6×10^{-4} s/m^3
4-30 days	4.8×10^{-4} s/m^3	4-30 days	1.1×10^{-4} s/m^3
Main steam relief valve and safety valve(6)		0-2 hrs	1.3×10^{-3} s/m^3
0-8 hrs	1.7×10^{-3} s/m^3	0-8 hrs	9.6×10^{-4} s/m^3
8-24 hrs	9.9×10^{-4} s/m^3	8-24 hrs	3.9×10^{-4} s/m^3
1-4 days	6.3×10^{-4} s/m^3	1-4 days	2.7×10^{-4} s/m^3
4-30 days	4.2×10^{-4} s/m^3	4-30 days	2.0×10^{-4} s/m^3
Steam line break releases(8)		0-2 hrs	1.3×10^{-3} s/m^3
0-8 hrs	1.4×10^{-3} s/m^3	0-8 hrs	9.6×10^{-4} s/m^3
8-24 hrs	8.4×10^{-4} s/m^3	8-24 hrs	3.9×10^{-4} s/m^3
1-4 days	5.3×10^{-4} s/m^3	1-4 days	3.2×10^{-4} s/m^3
4-30 days	3.5×10^{-4} s/m^3	4-30 days	2.4×10^{-4} s/m^3
Fuel handling area releases(7)		0-2 hrs	4.4×10^{-4} s/m^3
0-8 hrs	6.7×10^{-4} s/m^3	0-8 hrs	2.8×10^{-4} s/m^3
8-24 hrs	3.9×10^{-4} s/m^3	8-24 hrs	1.1×10^{-4} s/m^3
1-4 days	2.5×10^{-4} s/m^3	1-4 days	8.5×10^{-5} s/m^3
4-30 days	1.7×10^{-4} s/m^3	4-30 days	5.0×10^{-5} s/m^3
Atmospheric dispersion factors (χ/Q values) for TSC inleak for specified release points(3):			

Table 2.0-1R (Sheet 12 of 13)
Key Site Parameters

Plant Vent[5]		0-2 hrs	1.1×10^{-3} s/m^3
0-8 hrs	1.4×10^{-3} s/m^3	0-8 hrs	6.9×10^{-4} s/m^3
8-24 hrs	8.0×10^{-4} s/m^3	8-24 hrs	2.8×10^{-4} s/m^3
1-4 days	5.1×10^{-4} s/m^3	1-4 days	2.1×10^{-4} s/m^3
4-30 days	3.3×10^{-4} s/m^3	4-30 days	1.3×10^{-4} s/m^3
Ground-level containment releases[4]		0-2 hrs	8.0×10^{-4} s/m^3
0-8 hrs	1.9×10^{-3} s/m^3	0-8 hrs	5.1×10^{-4} s/m^3
8-24 hrs	1.1×10^{-3} s/m^3	8-24 hrs	2.3×10^{-4} s/m^3
1-4 days	7.2×10^{-4} s/m^3	1-4 days	1.6×10^{-4} s/m^3
4-30 days	4.8×10^{-4} s/m^3	4-30 days	1.1×10^{-4} s/m^3
Main steam relief valve and safety valve[6]		0-2 hrs	1.3×10^{-3} s/m^3
0-8 hrs	1.7×10^{-3} s/m^3	0-8 hrs	9.6×10^{-4} s/m^3
8-24 hrs	9.9×10^{-4} s/m^3	8-24 hrs	3.9×10^{-4} s/m^3
1-4 days	6.3×10^{-4} s/m^3	1-4 days	2.7×10^{-4} s/m^3
4-30 days	4.2×10^{-4} s/m^3	4-30 days	2.0×10^{-4} s/m^3
Steam line break releases[8]		0-2 hrs	1.3×10^{-3} s/m^3
0-8 hrs	1.4×10^{-3} s/ m^3	0-8 hrs	9.6×10^{-4} s/m^3
8-24 hrs	8.4×10^{-4} s/ m^3	8-24 hrs	3.9×10^{-4} s/m^3
1-4 days	5.3×10^{-4} s/ m^3	1-4 days	3.2×10^{-4} s/m^3
4-30 days	3.5×10^{-4} s/ m^3	4-30 days	2.4×10^{-4} s/m^3

Table 2.0-1R (Sheet 13 of 13)
Key Site Parameters

Fuel handling area releases(7)		0-2 hrs	4.4×10^{-4} s/m^3
0-8 hrs	6.7×10^{-4} s/m^3	0-8 hrs	2.8×10^{-4} s/m^3
8-24 hrs	3.9×10^{-4} s/m^3	8-24 hrs	1.1×10^{-4} s/m^3
1-4 days	2.5×10^{-4} s/m^3	1-4 daysr	8.5×10^{-5} s/m^3
4-30 days	1.7×10^{-4} s/m^3	4-30 days	5.0×10^{-5} s/m^3

NOTES:

1. The specified missiles are assumed to have a vertical speed component equal to 2/3 of the horizontal speed.
2. These dispersion factors are chosen as the maximum values at all intake points.
3. These dispersion factors are chosen as the maximum values at all inleak points.
4. These dispersion factors are used for a loss-of-coolant accident (LOCA) and a rod ejection accident.
5. These dispersion factors are used for a LOCA, a rod ejection accident, a failure of small lines carrying primary coolant outside containment and a fuel-handling accident inside the containment.
6. These dispersion factors are used for a steam generator tube rupture, a steam system piping failure, a reactor coolant pump rotor seizure and a rod ejection accident.
7. These dispersion factors are used for a fuel-handling accident occurring in the fuel storage and handling area.
8. These dispersion factors are used for a steam system piping failure.
9. These dispersion factors are used for a LOCA.
10. These dispersion factors are used for a rod ejection accident, a failure of small lines carrying primary coolant outside containment and a fuel-handling accident inside the containment.

11. Normal winter precipitation roof load is determined by converting ground snow load p_g in accordance with ASCE 7-05. The ground snow load p_g is based on the highest ground-level weight of:
    - the 100-year return period snowpack,
    - the historical maximum snowpack,
    - the 100-year return period snowfall event, or
    - the historical maximum snowfall event in the site region.
12. The extreme winter precipitation roof load is based on the sum of the normal ground level winter precipitation plus the highest weight at ground level resulting from either the extreme frozen winter precipitation event or the extreme liquid winter precipitation event. The extreme frozen winter precipitation event is assumed to accumulate on the roof on top of the antecedent normal winter precipitation event. The extreme liquid winter precipitation event may not accumulate on the roof, depending on the geometry of the roof and the type of drainage provided. The extreme winter precipitation roof load is included as live load in extreme loading combinations using the applicable load factor indicated in Design Control Document (DCD) Section 3.8.
13. χ/Qs were conservatively determined using the distances from each release location to the closer of either the Electrical Room HVAC or the Control Room HVAC intake. For all release locations except the main steam line break, the Class 1E Electrical Room HVAC intakes are closer to the release points than the Control Room HVAC intakes.
14. Acceptable parameters for settlement without further evaluation.
15. Settlements occuring during construction and operational life.
16. Settlements occuring during operational life only.

2.1 GEOGRAPHY AND DEMOGRAPHY

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

STD COL 2.1(1) Replace the content of DCD Section 2.1 with the following.

This section of the Final Safety Analysis Report (FSAR) provides information regarding the site location and description including the distribution of infrastructure, natural features, and population in the plant area. The discussion below is provided to address the guidance in NUREG-0800 (Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants) and Regulatory Guide 1.206 (Combined License Applications for Nuclear Power Plants [LWR Edition]). Radius distances defined by NUREG-1555 (Standard Review Plans for Environmental Reviews for Nuclear Power Plants) are used for the population analysis, rather than the distances described in RG 1.206 as an alternative method. The alternative method is used to ensure consistency of the population data between the FSAR and Environmental Report (ER). No other exceptions to the regulatory documents noted or alternative methods are used in the development of this section.

2.1.1 Site Location and Description

CP COL 2.1(1) Replace the content of DCD Subsection 2.1.1 with the following.

The following subsection presents site location and description information, including a site map and a boundary for establishing effluent release limits.

2.1.1.1 Specification of Location

Luminant Generation Company LLC (Luminant) proposes to construct and operate two MHI US-APWR reactors at their 7950-ac CPNPP site. The two reactors are referred to as CPNPP Units 3 and 4. The units and supporting infrastructure are sited in the area delineated in Figure 2.1-201.

The CPNPP site is located in Hood and Somervell counties, approximately 15 km (9.6 mi) south of the City of Granbury (Figure 2.1-202). The entire 80-km (50-mi) region is within the State of Texas. The CPNPP site is approximately 39 km (24 mi) west of Cleburne, 52 km (32 mi) west of Burleson, and 65 km (40 mi) southwest of downtown Fort Worth (Figure 2.1-203). The CPNPP site is situated on a peninsula located on the southwestern bank of the Squaw Creek Reservoir (SCR). Prominent natural and manmade features, including rivers and lakes, state and county lines, and military and transportation facilities are illustrated in Figures 2.1-201, 202, 203, and 204. Industrial facilities within 5 mi of CPNPP are illustrated in Figure 2.2-201. There are no military facilities located in the CPNPP vicinity. Figure 2.1-202 illustrates the features within a 10-km (6-mi) radius of the site center point. Detailed information regarding nearby industrial, transportation, and military facilities is presented in Section 2.2.

The CPNPP site lies mainly within the 7.5-minute Hill City Quadrangle but extends into the western portion of the Nemo Quadrangle. The quadrangles that bracket the site area are Tolar, Granbury, Acton, Glen Rose East, Glen Rose West, Chalk Mountain, and Paluxy (Reference 2.1-213).

The nearest population center (as defined by 10 CFR 100.3) to the CPNPP site is Cleburne as the city's population exceeds 25,000 (Reference 2.1-228). Cleburne is located 38.6 km (24.0 mi) east. The closest communities to the CPNPP center point are the cities of Glen Rose located 8.3 km (5.2 mi) southwest and Granbury located 15 km (9.6 mi) north. Granbury is the largest city within a 16-km (10-mi) radius of CPNPP (Figures 2.1-202 and 2.1-203). Granbury has a 2005 estimated population of 7360 while Glen Rose has a population of 2567 (Reference 2.1-228).

Interstate 20, located approximately 45 km (28 mi) northwest, connects the Dallas-Fort Worth Metropolitan area with Abilene. A farm-to-market (FM) road, FM 56, connects the site to U.S. Highway 67 (U.S. 67) and FM 51 (Figure 2.1-202). From Glen Rose, U.S. 67 connects with Cleburne to the east and with Stephenville to the west. FM 51 connects Granbury to Paluxy.

The coordinates of Units 3 and 4 are given below:

Latitude and Longitude NAD83 (degrees/minutes/seconds)

Unit 3	32° 18' 08.9" N	97° 47' 30.1" W
Unit 4	32° 18' 07.5" N	97° 47' 41.8" W

Northing and Easting in Texas Mercator North Central State Plane Projection NAD83 (ft)

	Northing	Easting
Unit 3	6793728	2187352
Unit 4	6793577	2186348

Universal Transverse Mercator Zone 14 NAD83 (Meters)

	Northing	Easting
Unit 3	3574606	613759
Unit 4	3574559	613453

2.1.1.2 Site Area Map

The proposed reactors, auxiliary buildings, turbine buildings, and cooling towers are labeled in Figure 2.1-201. A railroad spur enters the site, but there are no other transportation facilities, commercial, institutional, recreational, or residential structures within the site. The CPNPP site boundary is boldly outlined, and the highways and railroads located within the vicinity are shown in Figure 2.1-202. The site boundary is the same as the property boundary and the restricted area. The CPNPP Units 3 and 4 exclusion area boundary (EAB) extends 0.5 mi from each reactor center point. The total area contained by the site boundary is approximately 3220 ha (7950 ac) of land. Figure 2.1-204 is a U.S. Geological Survey topographic map that shows prominent natural and manmade features. Figure 2.1-205 illustrates the distances from the effluent release boundary (the boundary on which limits for the release of radioactive effluents are based) to the EAB in each of the 22.5-degree segments centered on the 16 cardinal compass points.

2.1.2 Exclusion Area Authority and Control

CP COL 2.1(1) Replace the content of DCD Subsection 2.1.2 with the following.

The property is clearly posted with “no trespassing” signs and all road access points are controlled. The road accessing Squaw Creek is controlled by fences and gates with security codes. The road to the power plant is controlled, once inside the EAB, with security check-points and barriers. The site’s physical security plan contains information on actions to be taken by security force personnel in the event of unauthorized persons crossing the EAB. The population distribution within 0.8 km (0.5 mi) of each reactor center point is zero.

2.1.2.1 Authority

All of the land and water inside the exclusion area is owned and controlled by Luminant, and is in the custody of Luminant. Additionally, Luminant controls all activities within the EAB including exclusion and removal of personnel and property from the area. Some subsurface mineral rights on the CPNPP site are not owned by Luminant; however, deed restrictions prevent mineral owners within the perimeter of the EAB but outside of the confines of SCR from placing vertical drilling rigs below the 240-m (800-ft) contour line. Luminant has absolute authority to control ingress rights for mineral rights exploration in the site. A 26-in crude oil pipeline operated by Sunoco Pipeline L.P. traverses the exclusion area approximately 2275 ft west-southwest of the center point as shown on Figure 2.2-204. This pipeline is also described in Subsection 2.2.2.3. Luminant has granted the pipeline owners easements that retain for Luminant absolute control to determine what type of activities can occur within the EAB, including ingress and egress for the purpose of maintaining the pipelines and their right-of-way.

2.1.2.2 Control of Activities Unrelated to Plant Operation

There are no residences or commercial activities not associated with CPNPP within the CPNPP Units 3 and 4 EAB. There is a company recreational area outside the CPNPP Units 3 and 4 EAB that has limited use but does contain a ballfield, tennis court, pavilion, restroom facilities, play area, and game area. The security firing range is also outside the EAB and is used by all site security personnel and local, state, and some federal law enforcement personnel. Squaw Creek Reservoir (SCR), located within the site boundary, is open to members of the public via controlled access for recreational uses, such as boating and fishing. A maximum limit of 100 boats on SCR is expected at any given time, not including special events. No public highways or railroads traverse the exclusion area.

2.1.2.3 Arrangements for Traffic Control

Arrangements with Somervell and Hood counties for control of traffic on-site in the event of an emergency are not required because no publicly used transportation routes cross the EAB. The SCR is open to the public via controlled access for recreational activities such as boating and fishing. Squaw Creek park area personnel, Luminant employees, and contractors perform actions in the event of an emergency to clear the lake area if needed. Site security personnel take actions to control traffic and unnecessary personnel on-site in the event of an emergency. Local, state, and federal law enforcement personnel in the area are trained to, and participate in, controlling of all off-site roads, population, and properties. Luminant has no authority for handling the public off-site but the emergency organization does make recommendations to the local officials in matters of emergency response.

2.1.2.4 Abandonment or Relocation of Roads

No public roads cross the exclusion area (Figure 2.1-202). Therefore, no abandonment or relocation of roads is necessary (Figure 2.1-205) (Reference 2.1-207).

2.1.3 Population Distribution

CP COL 2.1(1) Replace the content of DCD Subsection 2.1.3 with the following.

The population distribution surrounding the CPNPP site, up to an 80-km (50-mi) radius, is estimated based upon the most recent U.S. Census Bureau decennial census data (Reference 2.1-226). The population distribution is estimated in nine concentric bands at 0 – 2 km (1.24 mi), 2 – 4 km (2.5 mi), 4 – 6 km (3.7 mi), 6 – 8 km (5 mi), 8 – 10 km (6.2 mi), 10 – 16 km (10 mi), 16 – 40 km (25 mi), 40 – 60 km (37 mi), and 60 – 80 km (50 mi) from the center point between CPNPP Units 3 and 4. Population sectors out to 16 km (10 mi) are shown in Figure 2.1-206 and population sectors out to 80 km (50 mi) are shown in Figure 2.1-207. These bands are subdivided into 16 directional sectors, each centered on one of the 16 compass points and consisting of 22.5 degrees.

The population projections are derived from county estimates that were based on the cohort-component method (Reference 2.1-223). The counties that were used for the population projections are listed in Table 2.1-201. Using linear or polynomial regression, an equation was derived for each county. The equation was used in conjunction with the 2000 census data to produce a growth ratio. Ratios were calculated for each county and for each year, then weighted by area and summed into sectors. The ratio set was then used to produce a sector level population projection ratio set for the 80 km (50 mi) region. The census population numbers were then sorted into the radial grid. In instances where census blocks were divided by sector boundary lines, the population was weighted by area to produce proportionate data values. These values were summed and multiplied by their projection ratio to produce the final population radial grid maps (Figure 2.1-206, Figure 2.1-207).

2.1.3.1 Population within 10 Mi

Figure 2.1-208 illustrates the portion of the study area within 16 km (10 mi) of the site center point. The population sector map for the 16k (10 mi) radius is shown in Figure 2.1-206.

Permanent population is projected to 40 years beyond the 2016 construction completion date for the reactors. Table 2.1-202 shows the projected permanent population for each sector out to 16 km (10 mi), for the years 2007, 2016, 2026, 2036, 2046, and 2056. Population for all the sectors in the 16-km (10-mi) radius for each projected year is shown in the Cumulative Totals field of Table 2.1-202. The method for population projection is described in Subsection 2.1.3.

2.1.3.2 Population between 10 and 50 Mi

Figure 2.1-207 shows the regional population within 80 km (50 mi) of the site center point. The map contains the radial grid with 2007 estimated population and counties. The CPNPP region includes all or part of the counties listed in Table 2.1-201. The distances defining the sectors are 16 km (10 mi), 40 km (25 mi), 60 km (37 mi), and 80 km (50 mi). Fort Worth is the largest city within 80 km (50 mi), with a 2006 estimated population of 653,320 (Reference 2.1-229)(Reference 2.1-228). Smaller cities within the 80 km (50 mi) area include North Richland Hills, with a 2006 estimated population of 62,306; Mansfield, with an estimated population of 41,564; Haltom City, with an estimated population of 39,987; Burleson, with an estimated population of 31,660; Cleburne, with an estimated population of 29,689; Watauga, with an estimated population of 23,685; Weatherford, with an estimated population of 24,630; and Benbrook, with an estimated population of 22,307. The locations of these cities are shown in Figure 2.1-203. Several cities have 2006 estimated populations between 10,000 and 20,000. These include Azle, Forest Hill, Mineral Wells, Saginaw, Stephenville, and White Settlement. Many other small towns, cities, and urban areas with populations less than 10,000 are distributed within the 80 km (50 mi) area (Reference 2.1-228)(Reference 2.1-229).

Permanent population is projected to 40 years beyond the 2016 construction completion date for the reactors. Table 2.1-203 shows the projected permanent population for each sector in the 16 – 80 km (10 – 50 mi) radius for the years 2007, 2016, 2026, 2036, 2046 and 2056. The total number of people in the 16 – 80-km (10 – 50-mi) radius is shown in the Cumulative Totals field of Table 2.1-203 for each projected year.

2.1.3.3 Transient Population

Though relatively rural in nature, the region surrounding CPNPP has numerous tourist attractions that contribute moderate levels of transient population. Within a 10-km (6-mi) radius of the site, the largest tourist attraction is Dinosaur Valley State Park, with over 235,000 annual visitors.

The CPNPP region encompasses a large portion of the Fort Worth metropolitan area. Numerous attractions in Fort Worth combine to generate an estimated total of 10 million visitors each year.

Transient data are gathered through personal communications with businesses, companies, and local chambers of commerce within the region. This method for collecting transient data provides a more accurate assessment of people visiting the area and a much more precise location of transient contributors. Contributors to transient population in the CPNPP region are shown in Table 2.1-204.

Transient population is projected to 40 years beyond the 2016 construction completion date for the reactors. Table 2.1-205 contains the projected transient population for each sector and projections for the years 2007, 2016, 2026, 2036, 2046, and 2056 for the non-zero sectors. The sectors that have zero values are not illustrated in this table. Peak visitor numbers are provided when available. If the annual numbers are the only available data, then the average number of visitors per day is calculated from the total and taken as the peak. These peak and derived peak numbers are presented in the projected transient population.

2.1.3.3.1 Transient Population to 16 Km (10 Mi)

There are numerous facilities within the 16-km (10-mi) radius that host outdoor activities. These include Oakdale Park, Tres Rios River Ranch, the Texas Amphitheater, the Brazos Drive-In Theatre, and the Glen Lake Methodist Camp and Retreat Center. These facilities combined have approximately 438,000 visitors per year.

Several events during the year draw a large number of visitors, the most notable being the Annual 4th of July Celebration held in Granbury, which attracts approximately 50,000 visitors. Table 2.1-206 lists other events held in Granbury and Glen Rose, in addition to events elsewhere in the CPNPP region.

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

2.1.3.3.2 Transient Population 16 – 80 Km (10 – 50 Mi)

Excluding the Fort Worth metropolitan area, the majority of transients within the range of the 16 – 80 km (10 – 50 mi) area is attributed to parks and lodging. These include Cleburne State Park, Lake Mineral Wells State Park, Lake Whitney State Park, Meridian State Park, Rough Creek Lodge, and Riverbend Retreat Center. The four state parks and two resorts host more than 450,000 visitors (including day and overnight stay visitors) per year, while the Fossil Rim Wildlife Center attracts an additional 100,000 visitors (Reference 2.1-222).

The City of Fort Worth lies on the northeast periphery of the 80-km (50-mi) radius. There are several large attractions in the metro area, which in combination host nearly 10 million visitors per year. The Will Rogers Memorial Center hosts over two million visitors, with the largest event being the Southwestern Exposition Livestock Show and Rodeo in the spring. The Fort Worth Museum of Science and History, the Fort Worth Botanical Gardens, and the Fort Worth Convention Center each attract close to one million visitors per year. Other attractions include the Fort Worth Zoo, the Bass Performance Hall, the Kimball Art Museum, and the Casa Manana Dinner Theater.

Numerous events are held in Fort Worth in addition to the Southwestern Exposition Livestock Show and Rodeo. Two of the most prominent are the Main Street Arts Festival, held in downtown Fort Worth in April, and Mayfest, held in Trinity Park in May (Reference 2.1-217)(Reference 2.1-218). Events in the CPNPP region are listed in Table 2.1-206.

The nearest commercial airport is located approximately 16 km (10 mi) north of CPNPP in Granbury. In 2007, Granbury Municipal reported an average of 73 operations per day. Of those operations, almost two-thirds were local general aviation (Reference 2.1-202). Fort Worth is home to the region’s largest airport, Fort Worth Meacham International. In 2007, Fort Worth Meacham International reported an average of 271 operations per day. Of those operations, just over half were transient general aviation and approximately one-third were local general aviation (Reference 2.1-201). Passenger numbers are not publicly available for either airport.

There are no passenger trains that run within a 16-km (10-mi) radius of the CPNPP site. However, three passenger trains operate within the region. Amtrak’s Texas Eagle route passes through Fort Worth and Cleburne connecting Chicago, Illinois to San Antonio, Texas while the Heartland Flyer route travels between Fort Worth and Oklahoma City, Oklahoma. The Fort Worth and Cleburne stations have a combined annual usage of just under 83,600 people (Reference 2.1-204)(Reference 2.1-215). In addition, the Trinity Railway Express connects downtown Fort Worth to Dallas and served 2.16 million passengers in fiscal year 2004 (Reference 2.1-208).

2.1.3.3.2.1 Recreational Transients

Hunting and fishing in the portion of Texas included in the region are important recreational pastimes. The number of licenses issued in the region for the 2006 license year was 33,086 for hunting; 60,657 for fishing; and 38,972 for combined hunting and fishing.

Squaw Creek Reservoir (SCR), located within the site boundary, is open to members of the public via controlled access for recreational uses, such as boating and fishing. A maximum limit of 100 boats on SCR is expected at any given time, not including special events.

A portion of Lake Granbury falls within the vicinity of the CPNPP site. Lake Granbury has seven public use areas that provide opportunities for swimming, picnicking, and camping (Reference 2.1-205). Lake Granbury is also popular with boaters, with a peak season average of 290 boaters daily (Reference 2.1-206). Additionally, the Granbury Riverboat offers seven weekly cruises on the lake, with peak attendance of up to 150 passengers per cruise.

Two major campgrounds that host events in the vicinity of CPNPP are Oakdale Park and Tres Rios River Ranch, which combine to host 250,000 visitors annually (Reference 2.1-222). Some events held at Oakdale Park include Bluegrass Jam Sessions held once a month during the winter, the Texas State Mountain and Hammer Dulcimer Festival held in May, and the Annual Fall Woodcarving Show and Sale held in October and November (Reference 2.1-221). Events at Tres Rios River Ranch include the Boy Scout Camp in June, the Vietnam Vet Rally in September, and the Tommy Alverson Family Gathering in October (Reference 2.1-224). Events at other locations in the region are listed in Table 2.1-206.

Five golf courses are located within 16 km (10 mi) of the CPNPP site: Squaw Valley Golf Course, Pecan Plantation Country Club, Nutcracker Golf Club, Granbury Country Club, and Harbor Lakes Golf Course. Visitor numbers were not available for the Nutcracker Golf Course or Granbury Country Club, but the other three courses attract approximately 103,000 people each year.

There are five parks run by the Texas Parks and Wildlife Department located within the 80-km (50-mi) radius: Cleburne State Park, Dinosaur Valley State Park, Lake Mineral Wells State Park, Lake Whitney State Park, and Meridian State Park. These five parks account for over 643,000 visitors annually. Additionally, Acton State Historical Park is also located in the region, but no visitor numbers are kept for the site. Peak season extends from March through November.

2.1.3.3.2.2 Seasonal Population

Many of the attractions in the vicinity of the CPNPP site are based on outdoor activities. The peak times for these attractions, and the highest visitor numbers, occur from the spring to mid-fall. The lowest levels occur during the winter

months. Additionally, holiday weekends have higher visitor numbers than regular weekends.

2.1.3.3.2.3 Transient Workforce

Temporary workers for construction of the new plant are expected to be accommodated in Somervell and Hood counties where approximately 472 rental properties were available in 2000 (Reference 2.1-225)(Reference 2.1-227). At its peak, the temporary workforce for construction is expected to be 4300 workers, most of which are expected to be migrants to the vicinity.

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

There are 13 two-year and four-year colleges and universities within the CPNPP region. Total enrollment for these schools is more than 95,000 students (Reference 2.1-219). The two-year and four-year colleges and universities in the region are typically near peak capacity for the majority of the year, excluding the summer months (mid-May through mid-August). The majority of transients within the 80 km (50 mi) region are recreational in nature. Therefore, when educational institutions are at their lowest levels during the summer months, the overall transient population within the 80 km (50 mi) region is still at its highest level.

There are 18 major hospitals and medical centers within 80 km (50 mi) of CPNPP. These medical facilities have a combined capacity of 2687 staffed beds and discharge more than 131,000 patients per year. The two closest major medical facilities to the CPNPP site are Glen Rose Medical Center and Lake Granbury Medical Center. These two facilities account for 16 beds, 720 discharges, and 59 beds, 1998 discharges, respectively. The largest medical facility within the region is the Harris Methodist Fort Worth Hospital in Fort Worth, with 536 beds and more than 37,000 patient discharges annually (Reference 2.1-216)(Reference 2.1-203).

There are 48 nursing homes in the region of CPNPP, not including Fort Worth. These facilities combine for a total capacity of more than 5000 beds. The closest facility to the CPNPP site is Cherokee Rose Manor in Glen Rose, with a 102-bed capacity. The largest facility is the West Side Campus of Care in White Settlement, with a 240-bed capacity (Reference 2.1-220)(Reference 2.1-214).

Combining the college, hospital, and nursing home populations, the total number of people using the special facilities in the region is approximately 231,000 people. Special facility transients are not included in the total transients.

2.1.3.3.3 Total Permanent and Transient Populations

Table 2.1-204 shows contributors to the transient population in the CPNPP region. The peak daily transient population for the CPNPP region in 2007, not including special facility transients, is projected to be approximately 340,080 people. The

estimated permanent population for 2007 in the CPNPP region is approximately 1.5 million. The total population within the CPNPP region is projected to be approximately 1.9 million.

2.1.3.4 Low-Population Zone

For the CPNPP site, the low-population zone (LPZ) is defined as a 3.2-km (2-mi) radius from the site center point. Using this radius, no portion of Glen Rose is incorporated into the LPZ, as shown in Figure 2.1-209.

According to the U.S. Census Bureau 2000 data, there are 352 people living within the LPZ, primarily south of the site towards the Town of Glen Rose. The distribution of people within the LPZ in the 16 cardinal directions is provided in Table 2.1-207. There is one major contributor to the transient population in this area, the CPNPP Visitor Center, which attracts approximately 10,000 people annually (Reference 2.1-222). The LPZ is serviced by FM 56, which is routed through the western portion of the LPZ as shown in Figure 2.1-209. There are no facilities within the LPZ that require special consideration such as hospitals, prisons, jails, or any other facilities that involve confined populations. Industrial facilities within 5 mi are discussed in Subsection 2.2.2.

The CPNPP workforce is estimated at 1000 people for Units 1 and 2 and 420 people for Units 3 and 4, including operational, security, administrative, and contract workers. The 1420 person increase in population caused by the workforce results in an increase in the total daily population density within the LPZ from 11 people per sq km (28 people per sq mi) to 54 people per sq km (141 people per sq mi).

At the projected end of reactor operation (2056), the permanent population of the LPZ is expected to be 603, a density value of 19 people per sq km (48 people per sq mi). Combining this number with the estimated number of CPNPP employees, the total population is 2023 people, and the LPZ population density increases to 62 people per sq km (161 people per sq mi).

2.1.3.5 Population Center

The nearest population center, as defined by 10 CFR 100.3, is the urban area of Cleburne, with a 2000 population of 36,345 (Figure 2.1-203). Cleburne’s urban border, as defined by the U.S. Census Bureau, is situated 39 km (24 mi) to the east of the CPNPP Units 3 and 4 center point. Using county projection equations and projecting to the end of licensing (2056), Granbury becomes the closest population center. Granbury’s urban border is located 10.1 km (6.3 mi) from the CPNPP Units 3 and 4 center point. This distance is greater than 1-1/3 times the distance from the reactor center point to the boundary of the LPZ, as required by NUREG-0800.

The transient population is not considered in these calculations because 10 CFR 100.3 defines a population center as “the distance from the reactor to the

nearest boundary of a densely populated center containing more than about 25,000 residents." Transient populations by nature are not considered to be a part of the resident population.

2.1.3.6 Population Density

The projected permanent population of the CPNPP region is added to the projected transient population producing the total population. These values are plotted as a function of distance from the center point on Figure 2.1-210 and Figure 2.1-211 for the first year of operation (2016) and about five years after the first year of operation (2021), respectively. Illustrated on Figure 2.1-210 and Figure 2.1-211 is the cumulative population that would result from a uniform population density of 190 people per sq km (500 people per sq mi). The figures show that the total population density for both 2016 and 2021 does not exceed 190 people per sq km (500 people per sq mi).

The projected permanent population for 2016 is approximately 1.8 million and the projected transient population is 387,631. Transient population is projected using a ratio generated from transient sector population divided by the U.S. Census 2000 population. The projected permanent population for both 2016 and 2056 are multiplied by this ratio to calculate projected transient population. Thus, the projected total population within an 80-km (50-mi) radius in 2016 is approximately 2.2 million. The total population density for the startup year is 106 people per sq km (274 people per sq mi).

The projected total population within an 80-km (50-mi) radius in 2021, about five years after the first year of operations for the plant, is approximately 2.5 million. This is the sum of the projected permanent population (2,012,825 people) and the projected transient population (440,453 people). The total population density is projected to be 121 people per sq km (312 people per sq mi).

2.1.4 Combined Licence Information

Replace the content of DCD Subsection 2.1.4 with the following.

CP COL 2.1(1)
STD COL 2.1(1)

2.1(1) *Geography and demography*

This COL item is addressed in Subsections 2.1.1, 2.1.2, and 2.1.3 and the associated tables and figures.

2.1.5 References

CP SUP 2.1(1) Add the following references after the last reference in DCD Subsection 2.1.5.

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CP COL 2.1(1)

Table 2.1-201
Counties Entirely or Partially Located within the 80-km (50-mi) Radius

Bosque	Ellis	Jack	Somervell
Comanche	Erath	Johnson	Stephens
Coryell	Hamilton	McLennan	Tarrant
Dallas	Hill	Palo Pinto	Wise
Eastland	Hood	Parker	

CP COL 2.1(1)

Table 2.1-202 (Sheet 1 of 5)
Projected Permanent Population for Each Sector 0 – 16 km (0 – 10 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 0-2 (km)	2-4 (km)	4-6 (km)	6-8 (km)	8-10 (km)	10-16 (km)	0-16 (km)
North							
2007	0	16	51	154	337	9395	9953
2016	0	18	59	179	390	10,884	11,530
2026	0	21	67	206	450	12,540	13,284
2036	0	24	76	233	509	14,195	15,037
2046	0	27	85	260	568	15,850	16,790
2056	0	29	94	287	628	17,506	18,544
NNE							
2007	1	18	39	113	220	6379	6770
2016	1	21	45	131	255	7391	7844
2026	1	24	52	151	293	8515	9036
2036	1	26	59	171	332	9639	10,228
2046	1	29	66	191	371	10,763	11,421
2056	1	32	73	210	409	11,887	12,612
NE							
2007	0	15	112	161	359	2296	2943
2016	0	17	130	186	416	2660	3409
2026	0	19	150	214	479	3065	3927
2036	0	21	170	243	542	3469	4445
2046	0	23	190	271	605	3874	4963
2056	0	25	209	299	668	4279	5480
ENE							
2007	0	2	36	84	271	2566	2959
2016	0	2	40	95	311	2970	3418
2026	0	3	45	108	355	3418	3929
2036	0	3	49	121	399	3867	4439
2046	0	3	54	133	443	4315	4948
2056	0	3	58	146	488	4763	5458

CP COL 2.1(1)

Table 2.1-202 (Sheet 2 of 5)
Projected Permanent Population for Each Sector 0 – 16 km (0 – 10 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 0-2 (km)	2-4 (km)	4-6 (km)	6-8 (km)	8-10 (km)	10-16 (km)	0-16 (km)
EAST							
2007	0	5	131	29	54	161	380
2016	0	6	145	32	60	177	420
2026	0	6	159	35	66	195	461
2036	0	7	174	39	72	213	505
2046	0	8	188	42	78	232	548
2056	0	8	203	45	84	250	590
ESE							
2007	0	23	57	111	247	495	933
2016	0	25	62	123	272	545	1026
2026	0	27	69	135	299	600	1131
2036	0	30	75	147	327	655	1234
2046	0	33	81	160	355	710	1339
2056	0	35	88	172	382	765	1442
SE							
2007	0	71	89	135	316	304	915
2016	0	79	98	148	348	335	1008
2026	0	87	108	163	383	369	1110
2036	0	95	117	178	419	403	1212
2046	0	102	127	193	454	437	1313
2056	0	110	137	208	489	471	1415
SSE							
2007	0	140	109	799	1516	598	3162
2016	0	154	120	879	1668	658	3479
2026	0	169	132	968	1837	725	3831
2036	0	185	144	1057	2006	791	4183
2046	0	200	156	1146	2175	858	4535
2056	0	216	168	1235	2344	925	4888

CP COL 2.1(1)

Table 2.1-202 (Sheet 3 of 5)
Projected Permanent Population for Each Sector 0 – 16 km (0 – 10 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 0-2 (km)	2-4 (km)	4-6 (km)	6-8 (km)	8-10 (km)	10-16 (km)	0-16 (km)
SOUTH							
2007	8	80	24	124	68	365	669
2016	8	88	26	136	75	401	734
2026	9	97	29	150	83	442	810
2036	10	106	32	163	91	483	885
2046	11	115	35	177	98	523	959
2056	12	124	37	191	106	564	1034
SSW							
2007	29	67	20	25	40	193	374
2016	32	74	22	27	44	213	412
2026	35	81	25	30	48	234	453
2036	38	89	27	33	52	256	495
2046	41	96	29	36	57	277	536
2056	44	104	32	38	61	299	578
SW							
2007	28	51	31	44	42	92	288
2016	31	56	35	48	46	101	317
2026	34	62	38	53	51	112	350
2036	37	68	42	58	55	122	382
2046	40	73	45	63	60	132	413
2056	43	79	49	67	65	143	446
WSW							
2007	39	31	40	23	44	73	250
2016	43	34	45	26	50	83	281
2026	47	37	50	29	56	94	313
2036	52	41	54	32	62	105	346
2046	56	44	59	36	69	115	379
2056	61	48	64	39	75	126	413

CP COL 2.1(1)

Table 2.1-202 (Sheet 4 of 5)
Projected Permanent Population for Each Sector 0 – 16 km (0 – 10 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 0-2 (km)	2-4 (km)	4-6 (km)	6-8 (km)	8-10 (km)	10-16 (km)	0-16 (km)
WEST							
2007	12	12	49	101	45	119	338
2016	14	14	57	117	52	138	392
2026	15	16	65	135	60	159	450
2036	16	17	74	153	68	180	508
2046	18	19	83	170	76	201	567
2056	19	21	91	188	83	222	624
WNW							
2007	1	5	22	68	77	216	389
2016	1	6	26	79	89	250	451
2026	1	7	29	91	102	288	518
2036	1	8	33	103	116	326	587
2046	1	9	37	115	130	364	656
2056	1	10	41	127	143	402	724
NW							
2007	1	2	6	4	27	985	1025
2016	1	3	7	4	32	1141	1188
2026	1	3	8	5	37	1315	1369
2036	1	4	9	5	41	1488	1548
2046	1	4	10	6	46	1662	1729
2056	1	4	11	7	51	1835	1909
NNW							
2007	0	4	16	63	169	851	1103
2016	0	4	18	73	196	986	1277
2026	0	5	21	85	226	1136	1473
2036	0	6	24	96	256	1286	1668
2046	0	6	26	107	285	1436	1860
2056	0	7	29	118	315	1585	2054

CP COL 2.1(1)

Table 2.1-202 (Sheet 5 of 5)
Projected Permanent Population for Each Sector 0 – 16 km (0 – 10 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 0-2 (km)	2-4 (km)	4-6 (km)	6-8 (km)	8-10 (km)	10-16 (km)	0-16 (km)
Totals							
2007	119	542	832	2038	3832	25,088	32,451
2016	131	601	935	2283	4304	28,932	37,186
2026	143	665	1047	2558	4825	33,207	42,445
2036	156	730	1159	2832	5347	37,478	47,702
2046	169	791	1271	3106	5870	41,749	52,956
2056	182	855	1384	3377	6391	46,022	58,211

Cumulative Totals	0-2 (km)	0-4 (km)	0-6 (km)	0-8 (km)	0-10 (km)	0-16 (km)
2007	119	661	1493	3531	7363	32,451
2016	131	732	1667	3950	8254	37,186
2026	143	808	1855	4413	9238	42,445
2036	156	886	2045	4877	10,224	47,702
2046	169	960	2231	5337	11,207	52,956
2056	182	1037	2421	5798	12,189	58,211

Based on 2000 Census data.
(Reference 2.1-226)

CP COL 2.1(1)

Table 2.1-203 (Sheet 1 of 5) Projected Permanent Population for Each Sector 16 – 80 km (10 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year		Sector 16-40 (km)	40-60 (km)	60-80 (km)	16-80 (km)
North					
	2007	11,320	37,256	17,904	66,480
	2016	13,082	42,981	20,702	76,765
	2026	15,040	49,342	23,811	88,193
	2036	16,997	55,702	26,920	99,619
	2046	18,955	62,063	30,028	111,046
	2056	20,913	68,424	33,137	122,474
NNE					
	2007	7586	61,636	91,401	160,623
	2016	8777	70,856	104,610	184,243
	2026	10,099	81,100	119,287	210,486
	2036	11,422	91,345	133,964	236,731
	2046	12,745	101,589	148,641	262,975
	2056	14,067	111,834	163,318	289,219
NE					
	2007	5896	207,161	646,328	859,385
	2016	6963	237,503	736,399	980,865
	2026	8149	271,217	836,478	1,115,844
	2036	9335	304,930	936,557	1,250,822
	2046	10,521	338,644	1,036,636	1,385,801
	2056	11,707	372,358	1,136,715	1,520,780
ENE					
	2007	11,865	69,338	142,365	223,568
	2016	14,123	82,491	167,494	264,108
	2026	16,632	97,106	195,416	309,154
	2036	19,141	111,721	223,337	354,199
	2046	21,650	126,336	251,259	399,245
	2056	24,160	140,950	279,180	444,290

CP COL 2.1(1)

Table 2.1-203 (Sheet 2 of 5)
Projected Permanent Population for Each Sector 16 – 80 km (10 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year		Sector 16-40 (km)	40-60 (km)	60-80 (km)	16-80 (km)
East					
	2007	27,428	15,290	9326	52,044
	2016	32,648	18,041	11,060	61,749
	2026	38,447	21,097	12,987	72,531
	2036	44,246	24,154	14,914	83,314
	2046	50,045	27,211	16,840	94,096
	2056	55,845	30,267	18,767	104,879
ESE					
	2007	975	3951	13,732	18,658
	2016	1129	4398	15,293	20,820
	2026	1301	4894	17,026	23,221
	2036	1472	5391	18,760	25,623
	2046	1644	5888	20,493	28,025
	2056	1815	6384	22,227	30,426
SE					
	2007	1154	8043	6591	15,788
	2016	1249	8816	7258	17,323
	2026	1355	9676	7999	19,030
	2036	1461	10,535	8740	20,736
	2046	1566	11,394	9481	22,441
	2056	1672	12,254	10,222	24,148
SSE					
	2007	1061	2866	7218	11,145
	2016	1145	3092	7792	12,029
	2026	1238	3342	8430	13,010
	2036	1331	3593	9069	13,993
	2046	1424	3844	9707	14,975
	2056	1517	4094	10,345	15,956

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Table 2.1-203 (Sheet 3 of 5) Projected Permanent Population for Each Sector 16 – 80 km (10 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 16-40 (km)	40-60 (km)	60-80 (km)	16-80 (km)
South				
2007	1673	933	2547	5153
2016	1808	1000	2776	5584
2026	1958	1074	3022	6054
2036	2108	1147	3262	6517
2046	2258	1220	3493	6971
2056	2408	1291	3718	7417
SSW				
2007	688	2050	4478	7216
2016	748	2132	4639	7519
2026	814	2211	4788	7813
2036	880	2276	4906	8062
2046	946	2329	4991	8266
2056	1012	2368	5045	8425
SW				
2007	1172	1360	1492	4024
2016	1291	1471	1541	4303
2026	1424	1590	1580	4594
2036	1557	1706	1601	4864
2046	1689	1819	1605	5113
2056	1822	1927	1592	5341
WSW				
2007	5206	21,732	5543	32,481
2016	5738	23,951	5796	35,485
2026	6329	26,417	6024	38,770
2036	6919	28,883	6196	41,998
2046	7510	31,348	6313	45,171
2056	8101	33,814	6374	48,289

CP COL 2.1(1)

Table 2.1-203 (Sheet 4 of 5)
Projected Permanent Population for Each Sector 16 – 80 km (10 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year		Sector 16-40 (km)	40-60 (km)	60-80 (km)	16-80 (km)
West					
	2007	1566	3388	996	5950
	2016	1728	3734	1035	6497
	2026	1908	4118	1068	7094
	2036	2087	4503	1090	7680
	2046	2267	4887	1100	8254
	2056	2447	5271	1100	8818
WNW					
	2007	1236	853	1777	3866
	2016	1374	936	1890	4200
	2026	1527	1027	2009	4563
	2036	1680	1118	2120	4918
	2046	1833	1210	2224	5267
	2056	1986	1301	2320	5607
NW					
	2007	1805	1949	1703	5457
	2016	2061	2104	1834	5999
	2026	2345	2277	1980	6602
	2036	2629	2449	2126	7204
	2046	2914	2622	2272	7808
	2056	3198	2794	2418	8410
NNW					
	2007	4307	7022	23,143	34,472
	2016	4979	8013	25,718	38,710
	2026	5726	9115	28,580	43,421
	2036	6474	10,216	31,441	48,131
	2046	7221	11,317	34,303	52,841
	2056	7969	12,419	37,165	57,553

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Table 2.1-203 (Sheet 5 of 5)
Projected Permanent Population for Each Sector 16 – 80 km (10 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Direction / Year	Sector 16-40 (km)	40-60 (km)	60-80 (km)	16-80 (km)
Totals				
2007	84,938	444,828	976,544	1,506,310
2016	98,843	511,519	1,115,837	1,726,199
2026	114,292	585,603	1,270,485	1,970,380
2036	129,739	659,669	1,425,003	2,214,411
2046	145,188	733,721	1,579,386	2,458,295
2056	160,639	807,750	1,733,643	2,702,032

Cumulative Totals	16-40 (km)	16-60 (km)	16-80 (km)
2007	84,938	529,766	1,506,310
2016	98,843	610,362	1,726,199
2026	114,292	699,895	1,970,380
2036	129,739	789,408	2,214,411
2046	145,188	878,909	2,458,295
2056	160,639	968,389	2,702,032

Based on 2000 Census data.
(Reference 2.1-226)

CP COL 2.1(1)

Table 2.1-204 (Sheet 1 of 2)
Contributors to Transient Population within 80 km (50 mi)

Facility Name	Average Daily Transients	Peak Daily Transients
Will Rogers Memorial Center	12,458	41,667
Trinity Railway Express	5918	---
Billy Bob's Texas	5288	---
Fort Worth Cats Baseball	4167	---
Casa Manana Dinner Theater	---	3718
Fort Worth Convention Center	3014	3801
Fort Worth Museum of Science and History	2901	---
Fort Worth Botanical Gardens	2740	---
Fort Worth Zoo	2714	---
Bass Performance Hall	2135	---
Kimball Art Museum	971	---
Brazos Drive-In Theater	962	---
Fort Worth Herd	767	---
Hamilton Roping Arena	750	---
Modern Art Museum	649	---
Dinosaur Valley State Park	644	---
Fort Worth Nature Center and Refuge	551	---
Oakdale Park	548	---
Glen Rose Expo Center	545	2000
Lake Whitney State Park	332	---
Amon Carter Museum	325	---
Lake Granbury Boating	---	290
Lake Mineral Wells State Park	284	---
National Cowgirl Museum and Hall of Fame	276	---
Meridian State Park	274	---
Fossil Rim Wildlife Center	274	---
Stockyards Museum	272	---
Bureau of Engraving and Printing Visitors Center	255	---
Texas Cowboy Hall of Fame	247	---
Creation Evidence Museum	231	---
Cleburne State Park	229	---
Fort Worth Amtrak Texas Eagle	224	---
Texas Amphitheatre	164	---
Tres Rios River Ranch	137	---
Granbury Riverboat	136	---

CP COL 2.1(1)

Table 2.1-204 (Sheet 2 of 2)
Contributors to Transient Population within 80 km (50 mi)

Facility Name	Average Daily Transients	Peak Daily Transients
Pecan Plantation County Club	123	---
Squaw Valley Golf Course	99	300
Weatherford Rodeo Arena	96	5000
Hidden Oaks Golf Course	93	---
Texas Civil War Museum	82	---
Glen Lake Methodist Camp and Retreat Center	77	---
Riverbend Retreat Center	63	---
Harbor Lakes Golf Course	60	---
Rough Creek Lodge and Resort	55	---
DeCordova Bend Golf Course	51	---
Shooting Gallery Gun Range	50	---
Hood County Jail and Historical Museum	34	---
CPNPP Visitor Center	27	---
Somervell County Historical Museum	24	---
Pier 144 Marina and RV Park	21	---
Chandler’s Gun Shop and Shooting Range	8	---
Cleburne Amtrak Texas Eagle	5	---
Starr Hollow Golf Course	3	20
The Windmill Farm and Bed and Breakfast	3	150
Trickle Creek Cabins	2	---
Hideaway Ranch and Retreat	1	---

(Reference 2.1-204)

(Reference 2.1-206)

(Reference 2.1-222)

(Reference 2.1-208)

CP COL 2.1(1)

Table 2.1-205
Projected Transient Population for Each Sector 0 – 80 km (0 – 50 mi) for Years 2007, 2016, 2026, 2036, 2046, and 2056

Distance (km)	Direction	2007	2016	2026	2036	2046	2056
2	WSW	30	33	36	39	42	46
6	SE	2900	3191	3514	3837	4160	4483
6	SSW	706	776	855	934	1012	1091
8	SE	2191	2411	2655	2899	3143	3387
8	S	253	278	307	335	363	391
10	SE	563	620	682	745	808	871
10	SSE	1722	1895	2087	2279	2471	2663
16	N	29,639	34,339	39,561	44,784	50,006	55,228
16	NNE	60	69	80	90	101	111
16	NE	208	242	278	315	352	388
16	SE	69	76	84	91	99	107
16	S	300	330	364	397	431	464
16	NW	169	196	226	255	285	315
40	N	136	157	180	204	227	251
40	NNE	107	124	143	162	181	199
40	NE	80	95	111	127	144	160
40	E	11,634	13,848	16,308	18,768	21,228	23,687
40	SSW	270	294	320	346	372	398
40	SW	1	1	1	1	2	2
40	WSW	5580	6150	6783	7416	8050	8683
40	NW	22	26	29	33	36	40
40	NNW	6	7	8	9	9	10
60	N	45,423	52,403	60,158	67,913	75,668	83,423
60	NNE	92	106	122	137	152	168
60	NE	2215	2539	2899	3260	3620	3981
60	ENE	5680	6757	7955	9152	10,349	11,546
60	SE	11,135	12,205	13,395	14,585	15,775	16,964
60	SSE	715	771	834	896	959	1022
80	N	114	131	151	171	191	210
80	NNE	898	1028	1172	1316	1460	1604
80	NE	210,974	240,374	273,042	305,710	338,377	371,045
80	SSE	5321	5744	6215	6685	7155	7626
80	SSW	1750	1813	1871	1917	1950	1971
80	NNW	11,256	12,508	13,900	15,292	16,684	18,075

CP COL 2.1(1)

Table 2.1-206 (Sheet 1 of 2)
Events in the CPNPP Region

Event	Location	Dates for 2007	Total Visitors
Honeyfest 2007	Burleson	May 12	10,000
Antique Alley and Yard Sale Spring	Cleburne	Third weekend of April	30,000
Antique Alley and Yard Sale Fall	Cleburne	Third weekend of September	25,000
Octoberfest and Kaleidoscope Festival	Clifton	October 20	5000
Cowtown Marathon	Fort Worth	February 24	12,620
Jazz by the Boulevard Music and Arts Festival	Fort Worth	September 16-17	50,000
Main Street Arts Festival	Fort Worth	April 19-22	450,000
Mayfest	Fort Worth	May 4-7	300,000
Crown Plaza Invitational Golf Tournament	Fort Worth	May 24-27	175,000
Fort Worth Southwestern Exposition Livestock Show and Rodeo	Fort Worth	Jan. 12 – Feb. 4	1,000,000
Red Steagall Cowboy Gathering	Fort Worth	October 27-29	45,000
Texas Forts Muster	Fort Worth	April 28-29	30,000
Willie Nelson & Friends 4th of July	Fort Worth	July 4	50,000
PRCA Rodeo	Glen Rose	March	6000
Tommy Alverson Family Gathering	Glen Rose	October 5	7500
Annual 4th of July Celebration	Granbury	July 3-4	50,000
Brazos River Musicfest	Granbury	March 24	5000
Country Christmas Celebration	Granbury	November 23	7000
General Granbury's Birthday	Granbury	March 24	7000
Harvest Moon Festival	Granbury	October 20-21	5000
Thunder over Texas Christian Bike Rally and Car Show	Granbury	August 31	7000
Dove Festival	Hamilton	Labor Day Weekend	5000
Crazy Water Festival	Mineral Wells	October 8	10,000
Texas Music Festival	Stephenville	April 17-21	20,000
Christmas on the Square	Weatherford	December	5000

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Table 2.1-206 (Sheet 2 of 2)
Events in the CPNPP Region

Event	Location	Dates for 2007	Total Visitors
First Monday Weekends	Weatherford	Monthly	8,000
Parker County Peach Festival	Weatherford	July 9	40,000
PRCA Rodeo	Weatherford	June 14	20,000
AMA Pro/Am National Motocross	Whitney	March 6-11	10,000
Pioneer Days	Whitney	October	10,000
West Shores Fire Dept. Fish Fry	Whitney	Labor Day Weekend	5000

(Reference 2.1-209)

(Reference 2.1-210)

(Reference 2.1-211)

(Reference 2.1-212)

(Reference 2.1-217)

(Reference 2.1-218)

CP COL 2.1(1)

Table 2.1-207
Population Distribution in the Low-Population Zone

	0-1 (mi)	1-2 (mi)	0-2 (mi)
N	0	6	6
NNE	0	9	9
NE	0	6	6
ENE	0	0	0
E	0	0	0
ESE	0	2	2
SE	0	15	15
SSE	0	23	23
S	0	66	66
SSW	10	74	84
SW	9	55	64
WSW	19	37	56
W	8	9	17
WNW	0	3	3
NW	0	1	1
NNW	0	0	0
Total	46	306	352

(Reference 2.1-226)

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.

CP COL 2.2(1) Replace the content of DCD Section 2.2 with the following.

The Comanche Peak Nuclear Power Plant (CPNPP) is located in Hood and Somervell counties, Texas. Hood County is located north of Somervell County. The two counties are bounded by Parker County to the north, Johnson County to the east, Bosque County to the south, Erath County to the west, and Palo Pinto County to the northwest, as seen in Figure 2.1-203.

The CPNPP site is accessible by road and rail. Interstate 20 (I-20) connects the Dallas-Fort Worth metropolitan area with Abilene, and its closest portion to the site is located approximately 28 mile (mi) northwest (Reference 2.2-201). U.S. Highway 377 (US 377) runs southwest from the City of Fort Worth to Stephenville passing through Granbury. U.S. Highway 67 (US 67) connects Cleburne to Stephenville after passing through Glen Rose. The site is accessible by rail via a rail spur that runs from the CPNPP site to an intersection with the main line in Tolar, Texas. The Tolar line is owned by Fort Worth and Western Railroad and is located approximately 9.5 mi northwest of the center point between CPNPP Units 3 and 4.

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

2.2.1 Locations and Routes

CP COL 2.2(1) Replace the content of DCD Subsection 2.2.1 with the following.

Within a 5-mi radius of the CPNPP site, there is one railroad, four farm-to-market roads, one state highway, and one federal highway, all with commercial traffic (Reference 2.2-201). Not including CPNPP Units 1 and 2, there are eight industrial facilities including two electric generation plants within 5 mi of the site center point (Reference 2.2-202). There are no public airports within 5 mi of the site center point (Reference 2.2-213). Specifically, the following transportation routes and industrial facilities are shown in Figure 2.2-201.

- IESI Somervell County Transfer Station
- Wolf Hollow 1, LP
- Glen Rose Medical Center
- Cleburne Propane

- Wheeler Branch Reservoir and Water Treatment Facility
- DeCordova Compressor Station
- Glen Rose Wastewater Treatment Plant (WWTP)
- Texas Department of Transportation Maintenance Station
- Farm to Market Road 56 (FM 56)
- Farm to Market Road 205 (FM 205)
- Farm to Market Road 200 (FM 200)
- Farm to Market Road 51 (FM 51)
- Texas State Highway 144 (SH 144)
- U.S. Highway 67 (US 67)
- CPNPP railroad spur

There are no significant manufacturing plants, chemical plants, or refineries located within 5 mi of CPNPP.

Aboveground and underground storage tanks are located within 5 mi of CPNPP. State regulations for tank registrations are consistent with federal regulations and require the registration of underground storage tanks used for gasoline, diesel, used oil, and jet fuel as well as hazardous material storage tanks containing substances such as acetone or methyl ethyl ketone. Agricultural tanks with a storage capacity greater than 1100 gallons (gal) must be registered. Underground storage tanks that do not contain liquid substances do not need to be registered. Aboveground storage tanks containing petroleum products with a capacity greater than 1100 gal must be registered. (Reference 2.2-212)

Banks Information Solutions Inc. (BIS) provided a list of petroleum storage tanks from a database search of petroleum storage tanks registered by the Texas Department of Environmental Quality (TCEQ). According to the BIS report, there are six petroleum storage tanks within 5 mi of the CPNPP center point. The IESI Somervell County Transfer Station has one aboveground petroleum storage tank to the southeast containing 2000 gal of diesel. To the east of the site, Martha A. Newkirk has three underground storage tanks, all temporarily out of use as of February 2007. Also to the south, the Somervell County Maintenance Department has two tanks with a capacity of 4000 gal each. One tank contains diesel while the other tank contains gasoline (Reference 2.2-202). The capacity and contents of the storage tanks are described in Table 2.2-201, and the locations are shown in Figure 2.2-201.

In addition to the registered petroleum storage tanks above, there are also underground and aboveground storage tanks at the three power plants located within the 5-mi radius. These tanks are described in Subsections 2.2.2.2.7, 2.2.2.2.8, and 2.2.2.2.9.

Mining and quarry operations, drilling operations, and wells are discussed in Subsection 2.2.2.2.11. Oil and gas pipelines are discussed in Subsection 2.2.2.3. Military bases and missile sites are discussed in Subsection 2.2.2.2.12. Evaluation of explosions postulated to occur on transportation routes near CPNPP is addressed in Subsection 2.2.3.1.1.1.

2.2.2 Descriptions

CP COL 2.2(1) Replace the content of DCD Subsection 2.2.2 with the following.

The industries within 5 mi of CPNPP are located to the south and east. Figure 2.2-201 shows the location of these industries. A brief description of several major industrial facilities is listed below.

2.2.2.1 Description of Industrial Facilities

Seven industrial facilities, excluding electric generation stations, are located within 5 mi of CPNPP. Table 2.2-202 displays the industrial facilities within or near the 5 mi radius of CPNPP, their primary function or major products, and the number of persons employed. None of these facilities produce major products. Descriptions of these facilities are detailed in Subsections 2.2.2.2.1, 2.2.2.2.2, 2.2.2.2.3, 2.2.2.2.4, 2.2.2.2.5, 2.2.2.2.6, and 2.2.2.2.10. Subsections 2.2.2.2.7, 2.2.2.2.8, and 2.2.2.2.9 provide detailed information on the electrical generation stations closest to the CPNPP site, Wolf Hollow 1, LP, DeCordova SES, and the existing CPNPP Units 1 and 2. Aside from CPNPP Units 1 and 2, no nuclear generation plants are located within 50 mi of CPNPP.

2.2.2.2 Description of Products and Materials

This subsection provides descriptions of the products and materials regularly manufactured, stored, used, or transported in the vicinity of CPNPP.

2.2.2.2.1 IESI Somervell County Transfer Station

This site is a waste transfer station for Somervell County. The facility is located 4.2 mi south-southeast of the site. No hazardous materials are stored on-site, with the exception of an oil dump. The oil dump has a capacity of 200 gal and is emptied periodically. Waste is consolidated and shipped to licensed and controlled landfills in Weatherford or Fort Worth.

2.2.2.2.2 Glen Rose WWTP

The Glen Rose WWTP is run by the City of Glen Rose and treats approximately 320,000 gallons per day (gpd). The facility is located 4.2 mi southeast of the CPNPP Units 3 and 4 center point. On-site, there are ten 150-pound (lb) cylinders of chlorine used to treat the water. The cylinders are kept in a locked cage and are transported to and from the facility by truck. Table 2.2-203 contains the Occupational and Safety Health Administration (OSHA) permissible exposure limits for the hazardous materials stored on-site.

2.2.2.2.3 Glen Rose Medical Center

The Glen Rose Medical Center is a combined hospital and nursing home facility that provides medical services to the area. The facility is located 5 mi southeast of the site. No toxic chemicals are stored on-site, with the exception of some water treatment and cleaning supplies. Bio-hazardous materials are produced and shipped out once every two weeks by licensed transport. Liquid oxygen is stored in a 1000-lb tank that is refilled by truck as needed.

2.2.2.2.4 Cleburne Propane

The facility is located 3.9 mi east-southeast of the center point of CPNPP Units 3 and 4. Three propane storage tanks are located on-site: two 14,500-gal tanks and one 18,000-gal tank. The propane is transported to the site via semi trucks traveling south from Granbury on SH 144, with an average of two to three deliveries a week during the winter and three to four deliveries a month during the summer. The tanks are stored aboveground, mounted on concrete saddles. The facility also has three Bobtail trucks with propane tanks attached that are used to make local residential and commercial deliveries. The capacity of the Bobtail truck propane tanks ranges from 2600 to 2800 gal. Table 2.2-203 lists the OSHA permissible exposure limits for the hazardous materials.

2.2.2.2.5 DeCordova Compressor Station

The DeCordova Compressor Station is a natural gas compressor station operated by Enterprise and is located 3.1 mi northeast of the CPNPP Units 3 and 4 center point. The station has a volume of 700 to 800 cubic feet per day (cu ft/day) and is operational most of the time. The station has an automatic emergency shutdown if a drop in pressure is detected. Approximately 100 gal of lube oils are stored in a tank on-site. The tank is periodically refilled by a lube oil truck.

2.2.2.2.6 Texas Department of Transportation Maintenance Station

The Texas Department of Transportation Maintenance Station is located 4.9 mi south of the CPNPP Units 3 and 4 center point. The station's hazardous materials are listed in Table 2.2-204 while the OSHA permissible exposure levels are listed in Table 2.2-203.

2.2.2.2.7 Wolf Hollow 1, LP

Wolf Hollow 1, LP is a 730-megawatt (MW) gas-fired combined-cycle power plant located 4.2 mi northeast of the CPNPP site (Reference 2.2-211). Hazardous materials stored on the Wolf Hollow 1, LP site are listed in Table 2.2-205. The OSHA permissible exposure limits for the reported toxic materials are in Tables 2.2-203 and 2.2-206.

At this time no information is available concerning on-site storage tanks. An inquiry on the TCEQ database was performed and no on-site storage tanks were reported for this facility.

2.2.2.2.8 DeCordova SES

The DeCordova SES is a conventional gas/oil steam generating plant with four additional natural gas combustion turbines. The plant is located 9.35 mi northeast of the center point of CPNPP Units 3 and 4. Hazardous materials stored on-site are listed in Table 2.2-207. The OSHA permissible exposure limits for the reported toxic materials are in Table 2.2-203.

DeCordova SES has 13 aboveground storage tanks. The contents of the storage tanks are described in Table 2.2-208.

2.2.2.2.9 Comanche Peak Nuclear Power Plant

The existing CPNPP Units 1 and 2 are located within the CPNPP site boundary. The hazardous chemicals located on-site are listed in Table 2.2-209 while the OSHA permissible exposure limits are listed in Tables 2.2-203, 2.2-206, and 2.2-210. There are 22 aboveground storage tanks and four underground storage tanks on-site. These tanks hold petroleum products, gases, and other chemicals. The contents of the storage tanks are described in Table 2.2-211.

2.2.2.2.10 Wheeler Branch Reservoir and Water Treatment Facility

The Wheeler Branch Reservoir was completed in 2007 and is located 3.2 mi southeast of the CPNPP Units 3 and 4 center point. The reservoir has a surface area of 180 acres (ac) and a storage capacity of 4118 acre-feet (ac-ft). Plans are in place for a water treatment plant to process the 2000 ac-ft of water available each year for municipal use. The water treatment plant consists of the plant, ancillary facilities, and treated water distribution and storage facilities. The water treatment plant is expected to be constructed in 2010. It is anticipated that 1000 gallons of sodium hypochlorite at 12.5 percent are stored on-site for use in water treatment.

2.2.2.2.11 Mining and Quarrying Activities

There are no coal or lignite mines within the vicinity of CPNPP (Reference 2.2-208). There are 37 regular producing gas wells and two injection wells within 5 mi

of CPNPP. The closest producing gas well to CPNPP is located 1.2 mi northwest, while the closest permitted location is 1.2 mi to the north-northeast. Drilling activities are assumed to occur exclusively at existing or permitted well locations. The nearest plugged gas well is 0.9 mi west of the CPNPP Units 3 and 4 center point. There are no oil wells within 5 mi of CPNPP (Reference 2.2-209). Figure 2.2-202 shows the surface location of gas wells within 5 mi of CPNPP.

2.2.2.2.12 Military Facilities

There are no military facilities within 5 mi of the CPNPP site center point. The closest operating military facility is the Naval Air Station (NAS) Fort Worth, Joint Reserve Base (JRB) located approximately 36 mi northeast of the site. In 1993, NAS Dallas was relocated to the previous site of Carswell Air Force Base (AFB) under the Base Realignment and Closure Commission's decisions. The base was designated NAS Forth Worth, JRB at Carswell. The base occupies nearly all of the facilities established at Carswell AFB, the base of an air force bomber/tanker installation that was selected for closure in 1991. The base currently houses a wide array of fighter/attack and airlift units from reserve components of the U.S. Marine Corps, Navy, and Air Force. As a joint defense facility, the base plays a major role in training and preparing air crews and aviation ground support personnel as well as maintaining reservist readiness for impending deployment to various combat theaters. (Reference 2.2-203)

2.2.2.3 Description of Pipelines

There are eight major pipelines within 5 mi of the center point of CPNPP Units 3 and 4 as shown in Figure 2.2-204 (Reference 2.2-209). None of the pipelines operate at higher-than-normal pressure, and there are no plans for the pipelines to carry other materials besides the present products. Atmos Energy operates three natural gas pipelines: one 36-inch (in) pipeline passing through the northern portion of the site; one 6.63-in pipeline crossing the northern and western portions of the site; and one 16-in pipeline located northeast of the site. These pipelines are made of steel and were installed in 1972, 1974, and 1989, respectively. The maximum allowable operating pressure (MAOP) of the 36-in and 16-in pipelines is 960 pounds per square inch (psi), while the MAOP of the 6.63-in pipeline is 500 psi. The depths of the pipelines and location of isolation valves were not available.

Quicksilver Resources operates a 20-in natural gas pipeline to the northeast of the site and a 12-in natural gas pipeline to the east of the site. The pipelines are two years old or less and are buried at a minimum depth of 36 in below ground surface (bgs). The MAOP of both pipelines is 1050 psi. Both pipelines are made of steel. An isolation valve is located at the juncture of the two pipelines, with the next closest located at the Quicksilver plant south of Pecan Plantation. Valves are typically located every 7 mi along the pipelines.

Sunoco Pipeline, LP operates a 26-in crude oil pipeline that crosses the western and southern portions of the site. The pipeline has a MAOP of 750 psi. The

pipeline is made of steel and was constructed in the 1970s. The pipeline is located at a depth of 4 feet (ft) bgs. There are no valves within 5 mi of CPNPP, and the nearest station associated with the pipeline is over 30 mi away.

Enterprise operates two natural gas pipelines: one 30-in pipeline that passes through the northern portion of the site and one 14-in pipeline located northeast of the site. The 30-in pipeline was installed in 1971 and is buried 36 in bgs. The pipeline has a MAOP of 1050 psi. Both pipelines are made of steel. Isolation valves are located at the DeCordova Compressor Station, 3.1 mi northeast of the CPNPP Units 3 and 4 center point.

In addition to these major pipelines, there are numerous lines delivering natural gas to residential, commercial, and industrial units. These lines are operated by Meg Texas Gas Services LP or Peveler Pipeline LP and have diameters ranging from 2.38 to 16 in. A 36-in pipeline operated by Energy Transfer Partners (ETP) was installed in 2008. The steel pipeline is located 100 ft north of the existing Atmos Energy 36-in pipeline near the northern end of Squaw Creek Reservoir (SCR) and has a MAOP of 1480 psi. The pipeline is buried at a depth of 48 in bgs, except for the portion that is buried under SCR.

2.2.2.4 Description of Waterways

The only waterway near CPNPP is SCR, which is available to the public for recreational use via controlled access. Boating and fishing are permitted with a maximum of 100 boats at any given time, not including special events. There is no commercial traffic on SCR. There are no navigable rivers within 5 mi of the site. The intake structure for CPNPP Units 3 and 4 is not located on SCR (Subsection 2.2.3.1.5).

2.2.2.5 Description of Highways

The nearest highway with commercial traffic is FM 56, passing 1.4 mi west-southwest of the center point of CPNPP Units 3 and 4. The nearest federal highway with commercial traffic is US 67, passing about 4.4 mi to the south at its closest point. In addition to US 67, segments of SH 144 are located within a 5-mi radius of the center point of CPNPP Units 3 and 4.

Material registered with the federal government as a hazardous material must follow designated non-radiological hazardous materials (NRHM) routes around high population areas. In the CPNPP region, only Tarrant County has designated NRHM routes, centered in the city of Fort Worth (Reference 2.2-215). The amount of explosives shipped along the public roads within 5 mi of the facility is unknown. There are no federal, state, or local agencies that are required by law to keep records of transportation of hazardous materials; therefore, no data are available.

Estimated annual average daily traffic (AADT) counts in 2004 indicate the following (Reference 2.2-217):

- 3020 vehicles travel on FM 56 between mile 4.2 and 5.6 (west of the site).
- 11,780 vehicles travel on US 67 at mile 1.0, located in Glen Rose east of the intersection with FM 56, while 11,730 vehicles travel US 67 west of the intersection.
- 10,570 vehicles travel on SH 144 to the south of Granbury, while 6030 vehicles travel SH 144 north of the site.

2.2.2.6 Description of Railroads

The Fort Worth, Western Railroad Company owns and operates a railroad line that runs through the city of Tolar approximately 9.5 mi northwest of CPNPP. This line is the nearest main line to CPNPP. It covers the distance between Fort Worth and Brownwood. The nearest public transportation railway is the Amtrak Texas Eagle Route that passes through Cleburne 24 mi east of CPNPP. (Reference 2.2-216)

An average of two trains per day use the Tolar route. The railroad has a 50-ft right-of-way. No radiological material is transported on this line, but four to five cars of hazardous materials are transported each month.

However, these rail harzardous materials shipments are outside the 5 mi radius of CPNPP Units 3 and 4. As a result, these potential hazardous materials were not evaluated for CPNPP Units 3 and 4. See Subsection 2.2.3 for a discussion of potential hazardous materials accidents that were evaluated.

2.2.2.7 Description of Airports and Airways

This subsection provides descriptions of the nearby airports and regional airways.

2.2.2.7.1 Airports

There are no commercial airports within 5 mi of CPNPP (Reference 2.2-213). The nearest public airport is located approximately 10 mi north of CPNPP in Granbury. Granbury Municipal Airport has two runways located on a single asphalt stretch, with a length and width of 3603 ft and 60 ft, respectively. Runway 14 has a heading of 144 degrees magnetic (150 degrees true north), while Runway 32 has a heading of 324 degrees magnetic (150 degrees true north). The facility is a home base of operations for 82 single-engine aircraft, six multi-engine aircraft, and two helicopters. In 2007, Granbury Municipal Airport reported an average of 73 operations per day. Of those operations, 67 percent are local general aviation, 33 percent are transient general aviation, and none are military operations. (Reference 2.2-214)

There are several modifications and repairs planned for Granbury Municipal Airport. Improvements include widening and resurfacing the existing taxiways, and building an additional runway parallel to Loop 567. All runways are intended

to be upgraded to 30,000-lb pavement strength to accommodate the growing demand for business and corporate jet traffic from the Fort Worth/Dallas area (Reference 2.2-204). There have been no fatal aircraft accidents in the 5-mi radius of CPNPP in the last 20 yr. There have been four nonfatal accidents associated with Granbury in the last 10 years. (Reference 2.2-205)

Granbury Municipal Airport is the only public airport within 10 mi of the site. The reported average operations of 73 per day is well below the conservative threshold of $500D^2$ operations per year, where the variable D represents the distance in miles from the sites. There are no airports within the region that exceed the $1000D^2$ criterion.

Below are some predominant airports of interest outside 10 miles that do not exeed the 1000 D^2 criterion:

Cleburne Municipal Airport is a public, noncommercial airport located 29 mi east of the site. As of 2007, the airport had approximately 32,850 aircraft operations per year (Reference 2.2-233). There have been no fatal airplane accidents in the Cleburne area in the last 10 years. However, four nonfatal accidents have been reported during the same time period. (Reference 2.2-235)

Fort Worth Spinks Airport is a public, noncommercial airport located 33 mi northeast of the site. As of 2006, the airport had approximately 58,400 aircraft operations per year (Reference 2.2-235). There have been no fatal accidents in the Burleson area in the last 10 years. There have been two nonfatal accidents during the same time period (Reference 2.2-237).

Fort Worth Meacham International Airport is a public airport located 44 mi northeast of the site. As of 2007, the airport reported approximately 98,915 operations per year (Reference 2.2-234). There have been two fatal accidents associated with Fort Worth in the last 10 years. An additional 30 nonfatal accidents took place in the Fort Worth area during the same time frame (Reference 2.2-233).

Arlington Municipal Airport is a public, noncommercial airport located 48 mi northeast of the site. As of 2006, the airport reported approximately 151,475 operations per year (Reference 2.2-236). There have been no fatal accidents associated with the Arlington area in the last 10 years. Three nonfatal accidents took place during the same time frame (Reference 2.2-232).

2.2.2.7.2 Airways

There are no airways that pass within 5 mi of CPNPP as shown in Figure 2.2-203. The centerlines of two low-altitude flight lines pass within 10 mi of CPNPP. These routes, also known as Victor air routes, are primarily flown by general aviation aircraft. The routes generally have a width of 8 nautical mi, and occupy the airspace between 18,000 ft and the floor of controlled airspace (700 ft to 1200 ft).

Victor air route V18-94 tracks in an east-west manner and passes 9.7 mi south of the site at its closest point. Victor air route V17-18-568 tracks in a northwest-southeast manner and passes 6.6 mi to the west of the site at its nearest point.

The nearest high altitude flight line is J131, which tracks in a northeast-southwest manner and passes 6.9 mi southeast of the site at its nearest point. The second closest flight line is J23, which passes 15.7 mi northwest of the site. High-altitude airways are primarily used by commercial air carriers, the military, and high-performance general aviation aircraft. These routes also have a width of 8 nautical mi and are flown from 18,000 ft to the top of controlled airspace, 45,000 ft. All flights above 18,000 ft are required to be instrument flight rules (IFR) flights. All altitudes and routes are assigned by air traffic controllers.

One military training route, Victor air route VR-158, passes within 10 mi of the CPNPP site. This air route is used by T-38C aircraft for training purposes only, with 300 – 400 annual sorties or aircraft deployments. The route is located 7.8 mi southwest at its closest point. Flights on this route are between 500 ft and 5000 ft aboveground, with a width of 10 nautical miles. A second military training route, IR-139, passes 11 mi to the west at its closest point. This air route is used by F16, F5, and T38 aircraft with 10 annual sorties. Flights along this route are between 100 ft to 6000 ft with a width of 10 nautical miles.

One military operations area (MOA) is located within 50 mi of CPNPP. The Brownwood 1 East MOA is located 33 mi southwest of CPNPP at its closest point. The MOA is under continual use and has an effective altitude of 7000 ft.

2.2.2.8 Projections of Industrial Growth

There are no industrial parks within 5 mi of the CPNPP. No sizeable industrial growth is expected in the Glen Rose area. In Granbury, industrial expansion focuses on the property around the airport. Plans are in place for a 400-ac industrial park in the area around the airport. There are no known plans for expansion at the industrial facilities described in Subsection 2.2.2.2.

2.2.3 Evaluation of Potential Accidents

CP SUP 2.2(2) Add the following paragraph at the end of DCD Subsection 2.2.3.

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

plant as well as events and conditions that may occur outside the nuclear power plant.

2.2.3.1 Determination of Design Basis Events

CP COL 2.2(1) Add the following subsections after DCD Subsection 2.2.3.

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

2.2.3.1.1 Explosions

This subsection addresses potential explosion hazards from nearby transportation routes, and nearby industrial facilities. Nearby pipelines and gas wells are evaluated in Subsections 2.2.3.1.2.3 and 2.2.3.1.2.4.

2.2.3.1.1.1 Transportation Routes

Accidents were postulated for the nearby highways identified in Subsection 2.2.2.5. The nearest commercial traffic is FM 56, which passes approximately 1.4 mi west-southwest of the nearest safety-related structure of CPNPP Units 3 and 4. The accident of concern along FM 56 is one that results in the detonation of a highly explosive cargo carried by a truck. Based on Regulatory Guide 1.91, it is necessary to demonstrate that such an explosion on the highway does not result in a peak positive incident overpressure that exceeds 1 pounds per square inch (psi) at the critical structures on the CPNPP Units 3 and 4 site. The maximum probable hazardous cargo for a single highway truck is presented in terms of equivalent trinitrotoluene (TNT). Regulatory Guide 1.91 states the maximum probable hazardous solid cargo for a single highway truck is 50,000 lb. The TNT equivalency is based on The Departments of The Army, The Navy, and The Air Force TNT equivalency equation (Reference 2.2-220).

The methodology presented in Regulatory Guide 1.91 establishes the safe distance beyond which no damage would be expected (i.e., a peak positive incident overpressure of less than 1 psi at the critical structures on the CPNPP Units 3 and 4 site) from a truck explosion along FM 56 at its closest point. An evaluation performed for materials with a TNT equivalency of 2.24 and using the maximum cargo for two trucks determined the safe distance to be 0.52 mi. There is considerable margin between the required safe distance and the actual distance to the nearest safety-related structure (1.4 mi). The TNT equivalency

value of 2.24 is almost double the U.S. Department of Defense Explosive Safety Board value of 1.14 for HBX-3 (Reference 2.2-210), an explosive used primarily in underwater demolition and missile warheads. It is unlikely that two trucks carrying the maximum hazardous cargo, traveling in the same area at the same time would simultaneously explode. The assumption of two trucks provides an added degree of conservatism. Note that this assumption bounds the explosive energy of commonly transported materials such as gasoline and propane. This conservative approach was taken because there are no restrictions on the type or quantity of materials that can be transported on the highway. The effects of blast-generated missiles would be less than those associated with the blast overpressure levels considered in Regulatory Guide 1.91. Because the overpressure criteria of the guide are not exceeded, the effects of blast-generated missiles are not considered.

There are no navigable waterways used for commercial shipping within 5 mi of the CPNPP Units 3 and 4 site, and there are no main railroad lines within 5 mi of CPNPP Units 3 and 4, as discussed in Subsection 2.2.2.6. Figure 2.2-201 shows a spur of the main railroad line that goes past CPNPP Units 3 and 4 and ends at CPNPP Units 1 and 2. This spur is used to transport materials to and from the site and is not used for commercial transportation of chemicals and commodities. Thus, this spur of the mainline is not considered to be a hazard to CPNPP Units 3 and 4.

2.2.3.1.1.2 Nearby Industrial Facilities

Subsection 2.2.2.1 identifies the following facilities located within 5 mi of CPNPP Units 3 and 4, along with any potential hazardous material stored at those locations: the IESI Somervell County Transfer Station; Wolf Hollow 1, LP; the Glen Rose Medical Center; the Glen Rose WWTP; the Texas Department of Transportation Maintenance Station; and Cleburne Propane. Subsection 2.2.1 identifies six registered petroleum storage tanks within 5 mi of the CPNPP Units 3 and 4 site. The contents, capacities, and locations of the tanks relative to CPNPP Units 3 and 4 are summarized in Table 2.2-201.

The IESI Somervell County Transfer Station does not store any significant amount of hazardous materials. Though Wolf Hollow 1, LP does store some flammable or explosive chemicals, the quantity is too small to pose a hazard at CPNPP Units 3 and 4. Although quantities of hazardous materials were not available for Wolf Hollow, materials were screened out based upon their ability to form an explosive vapor at ambient conditions. Materials that did not screen out due to flashpoint were then assessed based upon maximum available quantities from commercial vendors, whether they were registered petroleum tanks, or expected quantities at this type of facility. The DeCordova SES does not house any chemicals that may pose a fire, explosion, or a vapor cloud risk to CPNPP Units 3 and 4. The Glen Rose Medical Center and the Glen Rose WWTP do not contain any flammable or explosive materials. There are no hazardous materials stored in significant enough quantity at the Texas Department of Transportation Maintenance Station to pose a threat to CPNPP Units 3 and 4.

Five registered underground storage tanks are located within 5 mi of the center point of CPNPP Units 3 and 4, three at Martha A. Newkirk and two at Somervell County Maintenance Department. Underground storage tanks do not represent a fire or explosion hazard. Any fuel that leaks from the tanks will be absorbed into the ground and will not be exposed to the atmosphere. No evaporation occurs and consequently no flammable vapor cloud can be formed because the fuels are not exposed to the atmosphere, eliminating the need for investigation. The one registered aboveground tank within 5 mi of the site is located at the IESI fleet refueling facility located approximately 4 mi south-southeast of the center point of CPNPP Units 3 and 4. This tank contains diesel fuel and is not considered to be volatile enough to represent a hazard at the CPNPP Units 3 and 4 site.

The CPNPP Units 1 and 2 on-site storage tanks listed in Table 2.2-211 were evaluated with respect to potential explosion hazards at CPNPP Units 3 and 4. It was concluded that these storage tanks meet the safe standoff distance requirements of Regulatory Guide 1.91. Of the tanks listed in Table 2.2-211, the tank that represents the greatest explosion hazard for CPNPP Units 3 and 4 is the propylene tank at the Bulk Gas Storage Facility. The CPNPP Units 1 and 2 Bulk Gas Storage Facility is located 1450 ft from the nearest safety-related structure at CPNPP Units 3 and 4. Based on the methodology of Regulatory Guide 1.91, the safe standoff distance for the propylene tank is 1174 ft for an unconfined vapor explosion. For a confined vapor explosion, the safe standoff distance is 581 ft.

Cleburne Propane is located 3.6 mi east-southeast of the nearest CPNPP Units 3 and 4 safety-related structures. The total amount of propane stored on-site among the four tanks and trucks is approximately 56,400 gal. Assuming this aggregate amount of propane is detonated, the safe standoff distance for a confined vapor explosion was determined, per the methodology of Regulatory Guide 1.91, to be 0.28 mi, and the safe standoff distance for an unconfined vapor explosion was determined to be 0.54 mi. The results for the confined and unconfined local vapor explosion are less than the actual standoff distance of 3.6 mi. Therefore, the postulated propane explosion at Cleburne Propane does not generate an overpressure above 1 psi at CPNPP Units 3 and 4.

2.2.3.1.1.3 On-site Explosion Hazards

Gas explosions from on-site sources outside containment at CPNPP Units 3 and 4 are not credible sources of missile generation per DCD Subsection 3.5.1. The chemicals used for the Makeup Water Treatment System are not flammable or explosive.

2.2.3.1.1.4 Gas Wells - Explosives

One technique used to control wellhead fires is the use of explosives to remove the oxygen from the air and thereby suffocate the fire. Potential wellhead fires in the Barnett Shale formation do not have sufficient flow rates to warrant the use of explosives to extinguish them. For the wells in this area, the wellhead fire fighters would use a water spray to extinguish the fire and then proceed to cap the

wellhead while working under the water spray. Thus, the impact on the CPNPP site from explosives to control wellhead fires need not be considered.

2.2.3.1.2 Flammable Vapor Clouds (Delayed Ignition)

The potential for detonation and deflagrations in a plume resulting from release of the commodities from a transportation accident was evaluated, as well as a potential release from nearby facilities. These evaluations assumed dispersion downwind toward CPNPP Units 3 and 4, with a delayed ignition. For each commodity of interest, the vapor dispersion was determined based on a wind speed of 1 meter per second (m/sec), a Stability Class of F, and a 105°F ambient air temperature. These meteorological conditions were chosen to maximize the vaporization rate of the commodity of interest while limiting the downwind dispersion. The Areal Locations of Hazardous Atmospheres (ALOHA) code (Reference 2.2-219) was used to evaluate the dispersion and detonation of the vapor clouds.

2.2.3.1.2.1 Transportation Routes

As discussed in Subsection 2.2.2.5, the closest highway to CPNPP Units 3 and 4 is FM 56. For the evaluation of the potential effects of accidents on FM 56, a single tanker truck volume of 9600 gal was assumed along with assumed rupture sizes of 4.5 square meters (m^2) and 1 m^2 located at the bottom of the tank. The release rates, puddle formation, and evaporation rates were calculated by the ALOHA code. The two hole sizes were analyzed in order to perform an evaluation of the effect of the hole size on the results so as to demonstrate the larger hole is bounding. The 1 m^2 rupture is a large rupture size. The 4.5 m^2 rupture is the largest size allowed by ALOHA based on the geometry of the tank. Because almost any commodity can be transported along the highways, various commodities were assumed. Gasoline and propane were analyzed because these commodities are commonly transported. Other less popular commodities (acetylene, ethylacetylene, ethylene oxide, propylene oxide) were analyzed. These commodities were determined to have a high enough reactivity to result in a vapor cloud explosion when the cloud is ignited by a spark or a flame. These evaluations determined that for all cases there is a negligible overpressure at the site resulting from ignition of a vapor cloud, and the concentrations remain below the lower explosive limit at CPNPP Units 3 and 4.

2.2.3.1.2.2 Industrial Facilities

According to Table 2.2-212, there are five possible sources that may release propane into the environment from Cleburne Propane (four tanks and three trucks). As described in Subsection 2.2.2.2.4, Cleburne Propane is located 3.9 mi east-southeast of the center point of CPNPP Units 3 and 4. Of these sources, the largest volume of propane is housed in an 18,000-gal tank. Large rupture sizes of 5 m^2 and 1 m^2 were examined for this facility. The release rates were calculated by the ALOHA code. The evaluation determined that there is a negligible

overpressure in the area of CPNPP Units 3 and 4 resulting from a delayed ignition of a vapor cloud, and the concentrations at the CPNPP Units 3 and 4 site are negligible.

2.2.3.1.2.3 Pipeline

Table 2.2-213 provides detailed information on the pipelines that were evaluated. These pipelines bound the potential effects to CPNPP Units 3 and 4. For the natural gas pipelines, the gas releases were calculated using the ALOHA code assuming each pipeline was connected to an infinite source so that gas escapes from the broken end of the pipeline at a constant rate for an indefinite period of time. The longest pipeline allowed by ALOHA is 6.2 mi. Thus, a pipe length of 6.2 mi is used for those pipelines where the nearest block valve or compressor station is farther away. The actual distance to the nearest block valve or compressor station is used if it is less than 6.2 mi. A constant pressure was maintained, and a single-ended rupture was assumed. The pipes were modeled as smooth pipes to minimize the frictional resistance. A pipe length of 6.2 mi was used to model the Atmos pipelines, because the nearest compressor station is approximately 20 mi from CPNPP Units 3 and 4. The pipe length used for the Enterprise Products pipeline evaluation is 5 mi, the approximate distance to the nearest compressor from the site. The ETP pipeline was also modeled with the maximum length of 6.2 mi allowed by the ALOHA code.

The release rates calculated by the ALOHA code are very conservative compared to the standard gas flow equations. Should a release from the Atmos, Enterprise, or ETP pipelines result in a detonation, resulting overpressure at the nearest safety-related structure is negligible. Also, concentrations at the CPNPP Units 3 and 4 control room intakes remain well below the lower explosive limits (LEL). The ETP pipeline represents the bounding natural gas pipeline accident as this pipeline has the largest size and maximum operating pressure. The ALOHA results demonstrate that there is a negligible overpressure in the area of CPNPP Units 3 and 4 resulting from ignition of the gas cloud and that the concentration of the natural gas at the CPNPP Units 3 and 4 site remains below 2260 parts per million (ppm), which is well below the lower flammability limit of 44,000 ppm.

For the Sunoco crude oil pipeline, both large breaks and small breaks were analyzed. For the large break, a maximum break flow of 47 cubic feet per second (cfs) was assumed. This flow was calculated based on the Darcy equation. The assumed friction length is 30 mi, the distance to the nearest compressor station. This flow rate is assumed for a 1-minute duration, the time to detect and isolate the large break. The flow is discharged directly into the atmosphere, and assumes the break occurs at the shortest distance between the pipe and the nearest safety-related structure (0.36 mi). The location of the pipeline relative to the CPNPP Units 3 and 4 is shown in Figure 2.2-204. The resulting overpressure at the nearest safety-related structure is 0.274 psi, which is much less than the 1 psi acceptance criteria. The vapor concentration at the CPNPP Units 3 and 4 control room intake is less than 8600 ppm, which is less than the LEL of 13,000 ppm.

For the small breaks, a leak rate of 0.62 cfs was assumed for a period of 32 hours (hr). This rate is based on the largest undetectable leak and the longest period of time the spillage would go unobserved. It is assumed that the entire amount of leaked crude oil is caught in the three retaining ponds. No absorption or evaporation is assumed as the crude oil makes its way to the retaining ponds. The distance from the closest retaining pond to a safety-related structure is 0.29 mi. An analysis of a vapor cloud formation from evaporation on this pond with a delayed ignition determined the peak overpressure to be 0.171 psi at the nearest safety-related structure. The concentration at the CPNPP Units 3 and 4 control room intakes is below 8680 ppm, which is below the LEL of 13,000 ppm. The Sunoco crude oil pipeline does not represent an explosion or flammable vapor cloud hazard at CPNPP Units 3 and 4.

2.2.3.1.2.4 Gas Wells

Wellhead blowouts are not considered credible in the Barnett Shale formation. A true blowout occurs when, during the drilling process, an unexpectedly high pressure pocket of gas or oil is entered that has a flow rate on the order of 10 to 20 million cu ft/day. With a natural gas well in the Barnett Shale, the driving gas pressure is not sufficient to move the column of fracing water that is in the casing when the well is stimulated. So the traditional wellhead blowout is not credible. However, once the well is in production mode, should the natural gas pipeline be breached (e.g., a bulldozer backing over it), the following is expected: <2 million cu ft/day of natural gas, <500 barrels of produced water/day, and <10 barrels of condensate. Condensate is heavy crude oil. Produced water is the salt water that is expected from the well that also contains natural gas, various hydrocarbon products, and some fracing water.

The closest functioning natural gas well, owned and operated by XTO Energy Inc., is 1.2 mi from the center point of CPNPP Units 3 and 4. For the purposes of evaluating the consequences of breaching a well, a gas release rate of 15.6 million cu ft/day was assumed. This rate bounds the largest absolute open flow potential for all the wells within 5 mi of CPNPP Units 3 and 4, and accounts for the backflow through the gathering lines. The results show that the maximum concentration at the CPNPP Units 3 and 4 control room intakes is 346 ppm, which is well below the LEL concentration of 44,000 ppm. The maximum overpressure at the closest safety-related structure resulting from ignition of the natural gas cloud is negligible. The analysis shows that, at the assumed release rate, the area of flammability is less than 0.1 mi downwind from a gas well release. The analysis also shows the overpressure from a gas explosion does not exceed 1 psig at a distance less than 0.1 mi from the cloud. It is concluded that the delayed ignition of vapor clouds from nearby transportation routes, pipelines, and facilities does not pose a hazard to CPNPP Units 3 and 4.

2.2.3.1.3 Toxic Chemicals

Events involving the release of toxic chemicals from on-site storage facilities and nearby mobile and stationary sources are considered in this subsection.

Regulatory Guide 1.78 is used for primary guidance on assumptions and criteria for screening out release events that need not be considered in the evaluation of main control room habitability. For releases of hazardous chemicals from stationary sources or from frequently shipped mobile sources in quantities that do not meet the screening criteria, detailed analyses for main control room habitability are discussed in Section 6.4.

2.2.3.1.3.1 Background

Figure 2.2-201 shows the potential stationary industrial sources and mobile sources (barge and river traffic, local highways, and local rail lines) within 5 mi of the CPNPP site. Each of these is discussed and compared to the screening criteria of Regulatory Guide 1.78 in the following sections. Distances from the hazardous chemical location to the nearest main control room (MCR) air inlet were used in the screening analysis.

Regulatory Guide 1.78 establishes the Immediately Dangerous to Life and Health (IDLH) values in National Institute for Safety and Health (NIOSH) "Pocket Guide to Chemical Hazards" as the toxicity value screening criteria for airborne hazardous chemicals. Per Regulatory Guide 1.78, the NIOSH IDLH values were utilized to screen chemicals and to evaluate concentrations of hazardous chemicals to determine their effect on main control room habitability. Quantities of materials were not made available for Wolf Hollow. As a result, only chemicals with NFPA 704 Health Hazard or HMIS Health ratings for three or four materials were considered, all others were screened out. Next, several chemicals were screened out based upon shipping weights, distance from the site, quantities expected to be stored on site, and the ability of the chemical to form a vapor cloud. Of the chemicals remaining, several were screened out based upon not being stored in single volume containers greater than 100,000 lbs. For the remaining chemicals that were not screened out, the masses at Wolf Hollow were determined based upon the mass of those same chemicals located at DeCordova with an increase of 25 percent. This was based upon similar facilities and similar material quantities. Using these masses, the final screening was performed in accordance with RG 1.78, Appendix A.

The possible stationary and mobile sources of hazardous chemicals, as described in Subsection 2.2.2, were initially screened as potential toxicity hazards based on the properties of the chemicals housed at the facility or in the case of mobile sources that may transverse the route. Only chemicals with NFPA 704 Health Hazard or HMIS Health ratings of three or four (highly or extremely toxic, respectively) were considered as potential toxicity threats, unless otherwise specified in Regulatory Guide 1.78 or NUREG/CR-6624.

The main control room habitability threats that could not initially be eliminated based on material properties or distance from the site were further investigated to determine if sufficient quantities of a chemical were housed at that location to warrant a detailed habitability analysis. Determination of the quantity of material

that warranted a detailed main control room habitability analysis is based on the methodology of Regulatory Guide 1.78.

2.2.3.1.3.2 Source Evaluation

The following subsections provide descriptions of the release sources.

2.2.3.1.3.2.1 Mobile Sources

Of the three mobile sources (road, railroad, and waterway), only roadways are within 5 mi of the site; neither railroads nor waterways need be considered further based on the distance criteria prescribed in Regulatory Guide 1.78.

Roadway FM 56 poses the largest potential mobile risk to the CPNPP Units 3 and 4 main control rooms due to postulated hazardous chemical releases. FM 56 serves as the bounding case because it is closest to the site (1.4 mi to the nearest MCR inlet) among the three roadways within 5 mi, and any registered hazardous material is permitted to travel this roadway. Based on a postulated chlorine release, the quantity of hazardous material that may transverse FM 56 is greater than the acceptable quantity as identified in Regulatory Guide 1.78. The frequency of a hazardous chemical release on roads was also examined. Results show the total frequency for a road-based hazardous material release is higher than the 1.0E-6 screening frequency of Regulatory Guide 1.78. Therefore, a more detailed main control room habitability analysis is necessary for roadway transportation. Table 2.2-214 summarizes the chemical, quantity, and distance to the nearest CPNPP Units 3 and 4 MCR inlet to be considered for the main control room habitability analysis in Section 6.4.

2.2.3.1.3.2.2 Stationary Sources

The fixed facilities that could not be initially screened out based on the chemicals stored at the facility are: Wolf Hollow I, LP; Cleburne Propane; DeCordova SES; and Glen Rose WWTP.

The hazardous chemicals housed at Glen Rose WWTP and Cleburne Propane are not sufficiently large to warrant a detailed habitability analysis based on the methodology in Regulatory Guide 1.78. DeCordova SES houses 15,294 lb of sodium hydroxide and 45,981 lb of sulfuric acid these quantities were evaluated based upon a distance of 3.7 mi from the nearest MCR inlet. This is conservative as the actual distance to DeCordova is 9.35 miles, which could have eliminated DeCordova from consideration in accordance with RG 1.75. Wolf Hollow I, LP houses sodium hydroxide and sulfuric acid in sufficient quantities to warrant a more detailed main control room habitability analysis. Those quantities are 19,118 lb and 57,477 lb, respectively, at 3.9 mi from the nearest MCR inlet.

Sunoco Pipeline, LP operates a pipeline which carries crude oil. This pipeline was the only pipeline that was not initially screened out based on the toxicity of the substance being transported. Crude oil may contain significant amounts of

hydrogen sulfide, which is a toxic chemical. A postulated pipeline release may contain sufficient quantities of hydrogen sulfide to warrant a more detailed main control room habitability analysis. The postulated release of hydrogen sulfide is 1716 lb at 0.33 mi from the nearest MCR inlet.

As noted in Subsection 2.2.3.1.1.2, five registered underground storage tanks are located within 5 mi of the center of CPNPP Units 3 and 4 site. Underground storage tanks are not treated as a location for significant airborne chemical release because any chemical release is mitigated by the ground and consequently its release rate and mass release to the atmosphere is significantly reduced. There is one aboveground tank within 5 mi of the site; it is located at IESI Somervell County Transfer Station and houses diesel fuel. Diesel fuel is not considered a toxic threat based on its chemical NFPA 704 health hazard rating of zero.

Chemicals on-site at CPNPP Units 1 and 2 are screened based on their distance (within 0.3 mi) to the closer MCR inlet of CPNPP Units 3 and 4 and the quantity of 100 lb. These criteria for on-site chemicals follow Regulatory Guide 1.78 guidance. The buildings, chemicals, quantities, and distance to the nearest CPNPP Units 3 and 4 MCR inlet that meet this criteria and house toxic chemicals, thereby requiring further analysis, are summarized in Table 2.2-214.

There are several chemicals currently planned to be on-site for CPNPP Units 3 and 4 to be used for water treatment. Those chemicals are morpholine, dimethylamine, hydrazine, ammonia, and sulfuric acid. Based on preliminary site plans, the bulk storage for the morpholine, dimethylamine, hydrazine, and ammonia will be located outside the turbine building, which could be approximately 330 ft from the main control room air intake. The day tanks are planned to be located inside the turbine building. Based on preliminary site plans, the storage for the sulfuric acid is near the main cooling towers, approximately 1200 ft to the main control room air intake. Because these chemicals are in quantities greater than 100 lb and potentially less than 0.3 mi from the main control room intake, a main control room habitability evaluation is required for these five chemicals.

Table 2.2-214 summarizes the chemicals that do not meet the Regulatory Guide 1.78 screening criteria, and the quantity and distance to the nearest CPNPP Units 3 and 4 MCR inlet to be considered for the main control room habitability analysis in Section 6.4.

2.2.3.1.4 Fires

Fires originating from accidents at any of the facilities or transportation routes discussed previously would not endanger the safe operation of the station because of the distance between potential accident locations and CPNPP Units 3 and 4. The location of CPNPP Units 3 and 4 is at least 0.25 mi away from any potential accident location.

The nuclear island is situated sufficiently clear of trees and brush. The distance exceeds the minimum fuel modification area requirements of 30 ft, per NFPA-1144 (Reference 2.2-221). There is no threat from brush or forest fires.

Fire and smoke from accidents at nearby homes, industrial facilities, transportation routes, or from area forest or brush fires, do not jeopardize the safe operation of the plant due to the distance of potential fires from the plant (Figure 2.2-201). Smoke detectors are located in the main control room outside air intakes and are used to automatically switch the main control room heating, ventilating, and air conditioning (HVAC) system from the normal operating mode to the emergency mode upon detection of smoke (DCD Subsection 9.4.1.2.2). Any potential heavy smoke problems at the MCR air intakes would not affect the plant operators.

A potential gas well fire was analyzed using the ALOHA code. The fire is modeled as a jet fire with a burn rate of 3.3E4 pounds per min. This flow rate bounds the maximum absolute open flow potential of the wells within 5 mi of CPNPP Units 3 and 4. The assumed distance is 1.2 mi from the center point of CPNPP Units 3 and 4, and is based on the location of the closest currently operating well. The resulting heat flux from a gas well fire on the closest safety-related structure is less than 0.02 kilowatts (kW) per m^2. The analysis shows that the heat flux decreases to 2.0 kW/m^2 at 219 yd (<0.15 mi) from the jet fire. This heat flux is sufficiently low as to not result in exceeding any of the thermal acceptance criteria of the structures.

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

2.2.3.1.5 Collision with Intake Structure

As discussed in Subsection 2.2.2.4, the only waterway near CPNPP is SCR, which is available to the public for recreational use via controlled access. The intake structure of CPNPP Units 3 and 4 is not located on SCR; SCR usage does not pose a threat to the intake structure. The essential service water system (ESWS) and the circulating water system (CWS) draw water from the intake structure on Lake Granbury to make up for water which has been consumed and discharged as part of the system operations. Figure 2.4.2-201 shows the intake structure is located on Lake Granbury, and Figures 2.2-201 and 2.4.1-203 show that Lake Granbury is more than five miles from the site. DCD Subsection 10.4.5.1.1 states the CWS does not have a safety-related function and has no safety design basis. As discussed in Subsection 9.2.1.3, the ESWS is supplied with water from the ultimate heat sink (UHS) and returns water to the UHS. The UHS is designed to assure sufficient cooling water inventory to mitigate the consequences of a design basis accident for a minimum of 30 days without makeup. Thus, potential consequences of collisions with the intake structure

would not be serious enough to affect the safety of the plant to the extent that the guidelines in 10 CFR 100 could be exceeded.

2.2.3.1.6 Liquid Spills

As discussed in Subsection 2.2.2.4, the only waterway near CPNPP is SCR, which is available to the public for recreational use via controlled access. The intake structure for CPNPP Units 3 and 4 is not located on SCR; release of liquids into SCR would not affect operation of the plant. The essential service water system (ESWS) and the circulating water system (CWS) draw water from the intake structure on Lake Granbury to make up for water which has been consumed and discharged as part of the system operations. Figures 2.2-201, 2.4.2-201, and 2.4.1-203 show the intake structure is located on Lake Granbury, which is more than five miles from the site. DCD Subsection 10.4.5.1.1 states the CWS does not have a safety-related function and has no safety design basis. As discussed in Subsection 9.2.1.3, the ESWS is supplied with water from the ultimate heat sink (UHS) and returns water to the UHS. The UHS is designed to assure sufficient cooling water inventory to mitigate the consequences of a design basis accident for a minimum of 30 days without makeup.

The accidental release of petroleum products into Lake Granbury, the most likely material released, would not affect operation of the plant. The normal water level in Lake Granbury is El. 693.00 ft, with the pump intake screen at 656.00 ft. Liquids with a specific gravity less than unity, such as petroleum products, would float on the surface of the lake and are not likely to be drawn into the makeup water system. Liquids with a specific gravity greater than unity would disperse and be diluted before reaching the pump intake. Thus, potential consequences of liquid spills in the vicinity of the intake structure would not be serious enough to affect the safety of the plant to the extent that the guidelines in 10 CFR 100 could be exceeded.

2.2.3.1.7 Radiological Release

The impact of CPNPP Unit 1 or 2 radiological releases on CPNPP Units 3 and 4 has been evaluated. This evaluation considered the release of radioactive material from CPNPP Units 1 and 2 due to normal operations and unanticipated events. For normal releases, the CPNPP Units 1 and 2 radiation monitoring program limits the maximum airborne radioactivity levels for normal and anticipated operational occurrences to within the limits of 10 CFR 20, Appendix B. The potential doses to CPNPP Units 3 and 4 personnel due to normal or anticipated releases from CPNPP Unit 1 or 2 are acceptable because these releases would be within the Appendix B limits.

For design basis events, the potential effects from CPNPP Units 1 or 2 radiological releases on the CPNPP Units 3 and 4 main control room personnel was found to be bounded by the CPNPP Units 1 or 2 main control room accident doses due to the greater atmospheric dispersion for CPNPP Units 3 or 4. Following a limiting design basis accident at CPNPP Units 1 or 2, any non-

essential CPNPP Unit 3 or 4 personnel would be evacuated in accordance with the Emergency Plan.

2.2.3.2 Effects of Design Basis Events

Potential design basis events associated with accidents at nearby facilities and transportation routes have been analyzed and the effects of these events on the safety-related components of the plant are insignificant as discussed in Subsection 2.2.3.1.

Many gas wells currently exist around the site, the closest being 1.2 mi from the center point of CPNPP Units 3 and 4. Future wells may be closer to the site as a result of further development of the Barnett Shale. Subsection 2.2.3.1.4 showed that the resulting heat flux from a fire satisfies the acceptance criteria at distances greater than 0.15 mi from the wellhead (approximately 660 ft). Thus, wells should be located no closer than 0.15 mi from any safety-related structure.

2.2.4 Combined License Information

CP COL 2.2(1) Replace the content of DCD Subsection 2.2.4 with the following.

> ***2.2(1)*** *Description of nearby facilities, establishment of hazards, and determination of accidents.*
>
> *This COL item is addressed in* Subsections 2.2.2 *and* 2.2.3 *and the associated tables and figures.*

CP SUP 2.2(2) Add the following new subsection after DCD Subsection 2.2.4.

2.2.5 References

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CP COL 2.2(1)

Table 2.2-201
Registered Petroleum Storage Tanks Within 5-Mi Radius of CPNPP

Site	Address	UST / AST	Distance from Center point of CPNPP Units 3 and 4 (mi)	Number of Tanks	Combined Capacity (gal)	Tank 1 Capacity (gal)	Tank 1 Contents	Tank 2 Capacity (gal)	Tank 2 Contents
IESI Somervell Co. Transfer Station	1591 N. FM 56	AST	4.2	1	2,000	2,000	Diesel	N/A	N/A
Martha A. Newkirk	4800 N. Hwy. 144	UST	3.3	3	0[(a)]	N/A	N/A	N/A	N/A
Somervell Co. Maintenance Dept.	28646 Hwy. 72	UST	3.8	2	8,000	4,000	Gasoline	4,000	Diesel

a) Tanks temporarily out of use

(Reference 2.2-202)

CP COL 2.2(1)

Table 2.2-202
Industrial Facilities near CPNPP

Name of Facility	Primary Function	Employees
Wolf Hollow 1, LP	Electric generation plan	
Cleburne Propane	Propane supply	3
DeCordova SES	Electric generation plant	23
Glen Rose Medical Center	Medical services	280
Glen Rose WWTP	Wastewater treatment	1
IESI Somervell Co. Transfer Station	Waste transfer facility	2
DeCordova Compressor Station	Natural gas compression	Unmanned
Texas Department of Transportation Maintenance Station	Street maintenance	22

CP COL 2.2(1)

Table 2.2-203 (Sheet 1 of 4)
OSHA Permissible Exposure Limit (PEL) Z-1 Table for Industrial Facilities Within 5 mi of CPNPP

Facility	Substance	Limit (ppm)	Limit (mg/m^3)
Glen Rose WWTP	Chlorine	(C) 1	(C) 3
Cleburne Propane	Propane	1000	1800
TxDOT	Ammonia	50	35
Maintenance Facility	Gypsum (respirable fraction)		5
	Isopropylamine	5	12
	Limestone (total dust)		15
	Portland cement (respirable fraction)		5
	Sulfuric acid		1
Wolf Hollow 1, LP	Ammonia	50	35
	Carbon dioxide	5000	9000
	Cyclohexane	300	1050
	Diacetone alcohol	50	240
	Dichlorodifluoromethane	1000	4950
	Ethanolamine	3	6
	Ethyl benzene	100	435
	Heptane	500	2000
	n-Hexane	500	1800
	Hydroquinone		2
	Isopropyl alcohol	400	980
	Methyl alcohol	200	260
	Mineral oil mist		5
	Naphthalene	10	50
	Phosphoric acid		1
	Propane	1000	1800
	Propylene oxide	100	240
	Sec-Butyl alcohol	150	450
	Sodium hydroxide		2
	Sulfuric acid		1
	Xylenes	100	435

CP COL 2.2(1)

**Table 2.2-203 (Sheet 2 of 4)
OSHA Permissible Exposure Limit (PEL) Z-1 Table for
Industrial Facilities Within 5 mi of CPNPP**

Facility	Substance	Limit (ppm)	Limit (mg/m^3)
DeCordova SES	Ammonia	50	35
	Carbon		3.5
	Carbon dioxide	5000	9000
	Ethylene glycol	(C) 0.2	(C) 1
	Hydrogen chloride	(C) 5	(C) 7
	Mineral oil mist		5
	Sodium hydroxide		2
	Sulfuric acid		1
CPNPP Units 1 and 2	Acetic acid	10	25
	Acetone (2-Propanone)	1000	2400
	Antimony		0.5
	Arsenic, inorganic compounds		0.01
	Barium (soluble compounds)		0.5
	1,2-benzenedicarboxylic acid dibutyl ester (dibutyl phthalate)		5
	n-Butyl-acetate	150	710
	n-Butyl alcohol	100	300
	2-Butoxyethanol	50	240
	Calcium carbonate (respirable fraction)		5
	Calcium oxide		5
	Carbon		3.5
	Carbon dioxide	5000	9000
	Carbon monoxide	50	55
	Chromium metal and insol. salts (as Cr)		0.5
	Copper (fume)		1
	Copper (total dust)		1
	Cryolite (as F)		2.5
	Diacetone alcohol	50	240
	Dichlorodifluoromethane	1000	4950

CP COL 2.2(1)

Table 2.2-203 (Sheet 3 of 4)
OSHA Permissible Exposure Limit (PEL) Z-1 Table for Industrial Facilities Within 5 mi of CPNPP

Facility	Substance	Limit (ppm)	Limit (mg/m^3)
	Dimethylamine	10	18
	Dolomite (respirable fraction)		5
	2-Ethoxyethanol	200	740
	Ethyl acetate	400	1400
	Ethyl alcohol (ethanol)	1000	1900
	Ethyl benzene	100	435
	Fluorspar (as F)		2.5
	Glycerin (total dust)		15
	Glycerin (respirable fraction)		5
	Graphite (synthetic)		15
	Gypsum (respirable fraction)		5
	Hydrazine	1	1.3
	Hydrochloric acid	(C) 5	(C) 7
	Iron oxide (fume)		10
	Isopropyl alcohol	400	980
	Lead		0.05
	Limestone (respirable fraction)		5
	Magnesium oxide fume		15
	Manganese compounds		(C) 5
	Methyl alcohol	200	260
	Methyl n-amyl ketone	100	465
	Methyl ethyl ketone (2-Butanone)	200	590
	Methyl isobutyl ketone (hexone)	100	410
	Mineral oil mist		5
	Molybdenum (soluble)		5
	Molybdenum (insoluble)		15
	Morpholine	20	70
	Naphtha	100	400

CP COL 2.2(1)

Table 2.2-203 (Sheet 4 of 4)
OSHA Permissible Exposure Limit (PEL) Z-1 Table for Industrial Facilities Within 5 mi of CPNPP

Facility	Substance	Limit (ppm)	Limit (mg/m^3)
	Naphthalene	10	50
	Nickel		1
	Nitric acid	2	5
	Phosphoric acid		1
	Portland cement (respirable fraction)		5
	Propane	1000	1800
	2-Propanol	400	983
	Sec-Butyl alcohol	150	450
	Silicon (respirable fraction)		5
	Silicon (total dust)		15
	Sodium fluoride		2.5
	Sodium hydroxide		2
	Starch		5
	Sulfuric acid		1
	Tin, inorganic compounds		2
	Titanium dioxide		15
	Triphenyl phosphate		3
	1,3,5-Trimethylbenzene	25	
	1,2,4-Trimethylbenzene (pseudocumene)	25	
	Vanadium (fume)		(C) 0.1
	Xylenes	100	435
	Zinc oxide fume		5
	Zinc oxide (respirable fraction)		5

(Reference 2.2-206), (Reference 2.2-222), (Reference 2.2-223), (Reference 2.2-224), (Reference 2.2-225)

A (C) designation denotes a ceiling limit.

CP COL 2.2(1)

Table 2.2-204
Hazardous Materials at Texas DOT Maintenance Station

Chemical Inventory	Amount On-site
Round-Up Pro Herbicide	290 gal
Outrider Herbicide	10.3 gal
Logan Asphalt Patching Material	4750 lb
Quikrete Cement	29,300 lb

CP COL 2.2(1)

Table 2.2-205
Hazardous Materials at Wolf Hollow 1, LP*

Chemical Inventory
1,1 Dichloro-1-fluroethane, isopropyl alcohol
Benzene
Ethyl cyanoacrylate, hydroquinone
Carbon dioxide
Methylene chloride, methyl alcohol, propylene oxide
Phosphoric acid
Heptane, mineral spirits
Isopropyl alcohol
Light aliphatic naptha
Sodium hydroxide (Caustic soda)
Ethanol amine & HCL (Rea L 1254)
Sulfuric acid
Petroleum solvent
Industrial gear oil
Distillates, hydrotreated heavy paraffinic
Gasoline
Petroleum distillates
Diesel
Aerokroil, petroleum based oil

*Quantities of chemicals were not available from Wolf Hollow. Subsection 2.2.3.1.1.2 and 2.2.3.1.3.1 discuss the screening criteria used in establishing what hazardous materials were used in the Explosion Hazards Analysis and Control Room Habitability Analysis, respectively.

CP COL 2.2(1)

Table 2.2-206
OSHA Permissible Exposure Limit (PEL) Z-2 Table for Industrial Facilities Within 5 mi of CPNPP

Facility	Substance	Time Weighted Average (8-hr shift)	Acceptable Ceiling Concentration	Acceptable Maximum Peak above Acceptable Ceiling Concentration (8-hr shift)	
				Concentration	Maximum Duration (min)
Wolf Hollow 1, LP	Benzene	10 ppm	25 ppm	50 ppm	10
	Methylene chloride	25 ppm			
	Toluene	200 ppm	300 ppm	500 ppm	10
CPNPP Units 1 and 2	Benzene	10 ppm	25 ppm	50 ppm	10
	Methylene chloride	25 ppm			
	Mercury		1 mg/10m^3		
	Toluene	200 ppm	300 ppm	500 ppm	10

(Reference 2.2-207), (Reference 2.2-227)

CP COL 2.2(1)

Table 2.2-207
Hazardous Materials at DeCordova SES

Chemical Inventory	Amount On-Site
Amberlite IR-122 resin	1815 gal
Amberlite IRA-402 resin	5820 gal
Ammonia	4000 lb
Ammonium hydroxide	8 gal
Carbon dioxide (liquid)	22,000 lb
Caustic soda (50%)	1200 gal
Dixichlor	2000 gal
Dowtherm SR-1 heat transfer	1063 gal
Ferric sulfate	3500 lb
Sulfuric acid solutions	1 gal
Teresstic 32	3895 gal
Shell Turbo T oil 32	12,660 gal
Carbon	1870 gal
Lead acid battery	67,585 lb
Sulfuric acid 66 BE	200 gal
Slimicide C-70	6186 gal
Sulfuric acid	3000 gal
Nitrogen	10,262 lb
Depositrol BL5301	2000 gal
Tax exempt low sulfur diesel 2	12,604,000 gal

CP COL 2.2(1)

Table 2.2-208
On-Site Storage Tanks at DeCordova SES

Chemical	Number of Tanks	Maximum Amount Stored On-Site
Ammonia	1	4000 lb
Carbon dioxide (refrigerated liquid)	1	22,000 lb
Caustic soda	1	1200 gal
Carbon	1	1870 gal
Depositrol BL5301	2	2000 gal
Dixichlor	2	2000 gal
Nitrogen	1	10,262 lb
Tax exempt low sulfur diesel 2	2	12,604,000 gal
Slimicide C-70	1	6186 gal
Sulfuric acid	1	3000 gal

CP COL 2.2(1)

Table 2.2-209 (Sheet 1 of 3)
Hazardous Materials at CPNPP Units 1 and 2

Chemical Inventory	Amount On-Site
Acetone	4.8 gal[(a)]
Acetic acid	1.5 gal
Aerokroil	21.5 gal[(a)]
Amershield	902 gal
Armorseal Rexthane I Urethane Floor Coating	660 gal
Arcaloy 300 Series AC-DC Stainless Steel Covered Electrodes	670 lb
Arcaloy Bare Stainless Steel Welding Wires	2448 lb
ATF Dexron II	61.3 gal
B-12 Chemtool Carburetor Choke Cleaner	5 gal
Bleach Sodium Hypochlorite 5.25%	123.8 gal
Broma 10-Minute Dry Decorative Enamel	556.5 lb[(b)]
Bromine	6000 gal
Butanone	56 gal
Calcium carbonate	1 lb
Carboline 3359	48 gal
Carboline 890 Part A Leaded	214 gal
Carbon	575 lb
Carbon dioxide (refrigerated liquid)	6200 lb
Carbon monoxide	235 lb
Caustic soda	48,000 lb
Certified Hydrazine 35%	155 gal
Chevron 1000 THF	15 gal
Copper	0.6 gal
Diesel fuel	439,951 gal
Dimethylamine, 40 WT.	461 gal
Dixichlor	24,424 gal
Dow Corning 321 Dry Film Lubricant	253 gal
Electrical grade silicon	1.56 gal

CP COL 2.2(1)

Table 2.2-209 (Sheet 2 of 3)
Hazardous Materials at CPNPP Units 1 and 2

Chemical Inventory	Amount On-Site
Electrolyte battery fluid acid	8 gal
Ethanol Spec Ind 200P 4084	167 gal
Fiberglass insulation	10,000 lb
Exxon Diesel 2	8000 gal
Exxon Varsol 1 fluid	211 gal
Formula 409 All Purpose Cleaner	115.3 gal[a]
Fyrquel 150	15.1 gal
Fyrquel 220	20 gal
Fyrquel EHC	1675 gal
Glycerine	4.8 gal
Glyptal 1201A	1.1 gal
Gulf Propane	1200 lb
Hydrazine 35% RR-44	690 gal
Hydrochloric acid	0.34 gal
Isopropyl alcohol	173 gal
Lead acid battery	459,460 lb
Lime Off	84 gal
Liquefied petroleum gas	6 lb
LPS 3 Heavy-Duty Rust Inhibitor	38.3 lb
L-Tec Solid Steel Welding Electrodes	140 lb
Magnaflux SKD-S2 Spotcheck Developer	275 gal
Masterflow 928 Grout	125 lb
Mercury	100 lb
Meropa 150, 220, 320, 460, 680	368 gal
Methyl alcohol	11.5 gal
Methyl ethyl ketone	1 gal
Morpholine	222 gal
MS-260 Safezone Cleaner	140 lb[b]
Krylon mineral spirits	45 gal

CP COL 2.2(1)

Table 2.2-209 (Sheet 3 of 3)
Hazardous Materials at CPNPP Units 1 and 2

Chemical Inventory	Amount On-Site
Neutrasorb acid neutralizer	107 lb
Nitric acid	12.9 gal
Oxygen	13,236 lb
Phosphoric acid	222 gal
Premium RB Grease	38.5 lb[(b)]
Propylene, Liquefield petroleum gas, LPG	4000 gal
Quikrete Concrete Mix	240 lb
SCAV-OX® 35% Hydrazine Solution	785 gal
Siltemp CH, SR, ST, WT, TR, BR, & S	30 lb
SKC-S Spotcheck Cleaner/Remover	123 lb
Sulfuric acid	168,000 lb
Texaco super unleaded gasoline	20 gal
Techalloy	1040 lb
Techalloy AWS ER80S-B2L	340 lb
Techalloy AWS ER502	210 lb
Thinner #33	222 gal
Toluene	1 gal
Unleaded gasoline	2532 gal
USG Sheetrock All Purpose Joint Compound Ready Mixed	10 gal
Wax Off II Floor Stripper	79.5 gal
Xylenes	72 gal
Z.R.C. Cold Galvanizing Compound (Aerosol)	6.9 gal

a) Amount excludes lb

b) Amount excludes gal

CP COL 2.2(1)

Table 2.2-210
OSHA Permissible Exposure Limit (PEL) Z-3 Mineral Dusts for Industrial Facilities Within 5 mi of CPNPP

Substance	mppcf	mg/m^3
Amorphous Silica	20	80 + $\%SiO_2$
Crystalline Silica (respirable fraction)		0.1
Graphite (Natural)	15	
Mica	20	
Portland cement	50	
Talc (not containing asbestos)	20	

(Reference 2.2-218), (Reference 2.2-228)

CP COL 2.2(1)

Table 2.2-211
On-Site Storage Tanks at CPNPP Units 1 and 2

Chemical	Type of Tank	Amount Stored
Bromine	AST	6000 gal
Bulab 7016	AST	6000 gal
Bulab 6040	AST	6315 gal
Carbon dioxide (liquid)	AST	6000 lb
Caustic soda	AST	48,000 lb
Compressed hydrogen	AST	638 lb
CMS buffer solution pH 10 (blue)	AST	500 gal
Diesel fuel	AST	1000 gal
Diesel fuel	AST	6000 gal
Diesel fuel	AST	32,000 gal
Diesel fuel	UST	100,000 gal
Diesel fuel	UST	100,000 gal
Diesel fuel	UST	100,000 gal
Diesel fuel	UST	100,000 gal
Dixichlor	AST	18,424 gal
Dixichlor	AST	6000 gal
Ferric sulfate solution	AST	48,000 lb
Fyrquel EHC	AST	1630 gal
Liquid nitrogen	AST	167,260 lb
Propylene, Liquefied petroleum gas	AST	4000 gal
Nitrogen	AST	4475 lb
Oxygen	AST	3561 lb
Sulfuric acid	AST	48,000 lb
Sulfuric acid	AST	1250 gal
Thruguard	AST	4400 gal
Unleaded gasoline	AST	2500 gal

CP COL 2.2(1)

Table 2.2-212
Cleburne Propane Storage Tanks

Material	Means of Storage	Volume (gal)
Propane	Tank	14,500
Propane	Tank	14,500
Propane	Tank	18,000
Propane	Truck	2800 per truck (three)
Propane	Tank	1000

CP COL 2.2(1)

Table 2.2-213
Pipelines Near CPNPP Units 3 and 4

Name of Pipeline Owner	Diam. (ft)	Distance to the Center point of CPNPP Units 3 and 4 (mi)	Direction to CPNPP	Distance to the Closest Safety Related Structure (mi)	Maximum Pressure (psi)
Atmos Pipeline - Texas	3.0 ft	2.2	N	2.1	960
Energy Transfer Partners (ETP)	3.0 ft	2.22	N	2.1	1480
Sunoco Pipeline, LP	2.2 ft	0.5	WSW	0.36	750
Enterprise Products Operating, LP	2.5 ft	2.2	N	2.1	1050
Cowtown Pipeline, LP	1.67 ft	2.2	N	2.1	1050
Atmos Pipeline - Texas	0.55 ft	0.7	WSW	0.56	500

CP COL 2.2(1)

Table 2.2-214
Toxic Chemicals that do not Meet the Regulatory Guide 1.78 Screening Criteria[a]

Hazardous Chemical Location	Chemicals	Quantity	Distance to the Nearest Units 3 and 4 MCR Inlet	IDLH	Calculated Maximum Concentration in Control Room
Roadway FM 56	Chlorine	42,500 lb	1.4 mi	1.0E+01 ppm	5.7 ppm
DeCordova SES	Sodium hydroxide	15,294 lb	3.7 mi[b]	10 mg/m^3	Not Analyzed[c]
	Sulfuric acid	45,981 lb		15 mg/m^3	1.9E-4 mg/m^3
Wolf Hollow 1, LP	Sodium hydroxide	19,118 lb	3.9 mi	10 mg/m^3	Not Analyzed[c]
	Sulfuric acid	57,477 lb		15 mg/m^3	2.0E-4 mg/m^3
Sunoco Pipeline, LP	Hydrogen sulfide	1716 lb	0.33 ml	1.0E+02 ppm	4.17 ppm
CPNPP Units 1 and 2, Waste Management Bldg.	Sulfuric acid	1250 gal (19,159 lb)	733 ft	15 mg/m^3	1.75E-03 mg/m^3
CPNPP Units 1 and 2, Bulk Gas Storage	Liquefied petroleum gas	4000 gal	1400 ft	2.10E+03 ppm	3.63E+01 ppm
	Carbon dioxide	6000 lb		4.0E+04 ppm	1.46E+01 ppm
CPNPP Units 3 and 4, Water Treatment Chemicals	Morpholine	10,000 gal	<300 ft	1.4E+03 ppm	3.49E-01 ppm
	Dimethylamine, 40%	5000 gal	<300 ft	5.00E+02 ppm	1.65E+01 ppm
	Hydrazine	1000 gal	<300 ft	5.0E+01 ppm	9.29E-02 ppm
	Ammonia	1000 gal	<300 ft	3.0E+02 ppm	2.70E+01 ppm
	Sulfuric acid	10,000 gal	<1200 ft	15 mg/m^3	6.19E-03 mg/m^3
CPNPP Units 3 and 4, Chiller Refrigeration	Refrigerant (R-134a used as typical)	< 2570 lbs at a vapor density of 9.369 lbs/m^3	104 ft [d] 123 ft [e]	Asphyxiant	(f)

a) These chemicals do not meet the Regulatory Guide 1.78 screening criteria. They are further evaluated for control room habitability in Section 6.4.

b) Evaluations were completed using 3.7 miles. Actual distance is 9.35 miles, as shown in Subsection 2.2.2.2.8. Therefore, the results of these evaluations are conservative.

c) This chemical does not readily disperse; therefore, it was not analyzed.

d) Straight line from the closest essential chiller unit to the control room door entrance.

e) Straight line from the closest non-essential chiller unit to the control room door entrance.

f) Resulting oxygen concentration for entire refrigerant quantity added to control room is greater than the OSHA 29 CFR1915.12(a)(2) confined space lower limit of 19.5%.

2.3 METEOROLOGY

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

CP SUP 2.3(1) Add the following paragraph after the paragraph in DCD Section 2.3.

This section provides a description of the meteorology of the site and its surrounding areas. Table 2.0-1R gives a comparison of the key Comanche Peak Nuclear Power Plant (CPNPP) site meteorological characteristics with the DCD design parameters.

2.3.1 Regional Climatology

CP COL 2.3(1) Replace the content of DCD Subsection 2.3.1 with the following.

This subsection describes the general climate of the region with respect to types of air masses, synoptic features (high- and low-pressure systems and frontal systems), general airflow patterns (wind direction and speed), temperature, humidity, precipitation (rain, snow, sleet, and freezing rain), potential influences from regional topography, and relationships between synoptic-scale atmospheric processes and local (site) meteorological conditions.

2.3.1.1 General Climate

From the hot, dry desert of Far West Texas and the blue northers that blast the Llano Estacado to the humid, rainy pine forests of East Texas and the hurricanes that sweep across the Gulf Coast, Texas' climate is as varied as its landscape. That variability is a result of the interactions between Texas' unique geographic location and the movements of seasonal air masses, such as arctic fronts, the jet stream, subtropical west winds, tropical storms, and a subtropical high pressure system known as the Bermuda High (Figure 2.3-201). (Reference 2.3-201) The location of Texas with relation to the North American continent, the warm Gulf of Mexico, and the not-far-distant Pacific Ocean guarantees a constant exchange of settled and unstable weather. The state's varied physiography, from the forests of the east and the Coastal Plain in the south to the elevated plateaus and basins in the north and west, also brings a wide variety of weather on almost any day of the year. Because of its expansive and topographically diverse nature, Texas offers continental, marine, and mountain-type climates. West of the Caprock on the High Plains, a continental climate, marked by cold winters and low humidity, predominates. In the Trans-Pecos, a form of mountain climate is found. The eastern two-thirds of Texas, on the other hand, have a humid, subtropical climate that is occasionally interrupted by intrusions of cold air from the north. Though variations in climate across Texas are considerable, they are nonetheless gradual. (Reference 2.3-202)

The State of Texas lies within both "cool" and "warm" parts of the Temperate Zone of the northern hemisphere. Texas has three major climatic types, which are classified as Continental, Mountain, and Modified Marine. There are no distinct boundaries which divide these climate types, but the approximate area of Texas that each encompasses is indicated on Figure 2.3-202 by the broad stippled lines. (Reference 2.3-203)

A Continental Steppe climate is prevalent in the Texas High Plains. This climate type is typical of interiors of continents and is characterized by large variations in the magnitude of ranges in daily temperature extremes, low relative humidity, and irregularly-spaced rainfall of moderate amounts. The main feature of this climate in Texas is semi-arid with mild winters. (Reference 2.3-203)

The Mountain climate is dominant in the Guadalupe, Davis and Chisos Mountains of the Trans-Pecos region of Texas. The characteristics of this climate are cooler temperatures, lower relative humidity, orographic precipitation anomalies and less dense air. The mountain climate is contrasted by the Subtropical Arid climate of the surrounding lowlands. (Reference 2.3-203)

Most of the state, climatologically, has a Modified Marine climate which is classified and named "Subtropical," with four subheadings. A marine climate is caused by the predominant onshore flow of tropical maritime air from the Gulf of Mexico. The onshore flow is modified by a decrease in moisture content from east to west and by intermittent seasonal intrusions of continental air. The four subheadings of Subtropical-Humid, Subhumid, Semi-arid and Arid account for the changes in moisture content of the northward flow of Gulf of Mexico air across the state. (Reference 2.3-203)

The climatic descriptions of the regions delineated on Figure 2.3-202 are given below:

- The eastern third of Texas has a Subtropical Humid climate that is most noted for warm summers.
- The central third of Texas has a Subtropical Subhumid climate characterized by hot summers and dry winters.
- The broad swath of Texas from the mid-Rio Grande Valley to the Pecos Valley has a Subtropical Steppe climate and is typified by semi-arid to arid conditions.
- The High Plains region of West Texas features a Continental Steppe climate with large variations in daily temperature extremes, low relative humidity, and irregularly-spaced rainfall of moderate amounts.
- The basin and plateau region of the Trans-Pecos features a Subtropical Arid climate that is marked by summertime precipitation anomalies of the mountain relief.

- A Mountain type climate is common in the higher elevations of the Guadalupe, Davis and Chisos mountains.

The variation of climate types in Texas is caused by the physical influences of the state being located (1) downwind from mountain ranges to the west, (2) proximate to the Gulf of Mexico and the southern Great Plains, (3) west of the center of the Bermuda high pressure cell, (4) at a relatively low latitude, and by (5) the changes in land elevation from the high plains and mountains to the coastal plains. These influences on the weather-particularly on the moisture content of the air-define climate and are evident by comparing the changes of contour patterns that are illustrated on the monthly series of maps in the following paragraphs (Reference 2.3-203).

Far West Texas has a climate more similar to New Mexico than to the rest of Texas. This region of far west Texas is also referred to as the Trans-Pecos region and is represented by Division five on Figure 2.3-203. Winters are cold and generally dry, except for rain and snow that fall mostly in the higher elevations. Summer is the rainy season, and moisture from both the Gulf of Mexico and the Gulf of California contribute to afternoon thunderstorms. Annual precipitation depends on elevation more than location; the dry grasslands near Marfa do not resemble the rest of the state but have become part of the public consciousness of the Texas natural environment. (Reference 2.3-204)

Other parts of Texas have neither the topographic relief nor the wide variations of climate of Far West Texas. The terrain changes steadily and continuously from one end of the state to the other, the terrain is interrupted only by such features as the Caprock Escarpment (in the Panhandle) and the Balcones Escarpment (along the southern and eastern margins of the Hill Country). The terrain descends from northwest to southeast, drained into the Gulf of Mexico by a series of parallel rivers. (Reference 2.3-204)

The climate changes are even more gradual than the terrain. Annual mean temperatures are coolest to the north and warmest to the south (see temperature and precipitation graphs on Figure 2.3-203). Annual mean precipitation is heaviest in the east and lightest in the west. Precipitation changes are more substantial than the temperature changes, as the near-desert in the west gradually gives way to annual accumulations close to 60 in along the Louisiana border. (Reference 2.3-204)

With subtle variations in climate and terrain, sub-regions of the State of Texas are often more clearly delimited by changes in vegetation or terrain character. Because many transitions are gradual, categorizations are somewhat arbitrary (Reference 2.3-204). The ten climate divisions identified by the NCDC group the regions according to similarity of overall climatic characteristics (Figure 2.3-203). The Trans-Pecos region was discussed above. The other nine divisions are described below.

The 10 climate divisions represent regions with similar climatic characteristics, such as vegetation, temperature, humidity, rainfall, and seasonal weather changes. Data collected at locations throughout the state are averaged within the divisions in order to make maps such as the one in Figure 2.3-203. These divisions are commonly used to report climatic information, such as precipitation, temperature, and drought indices (Reference 2.3-201). The Texas High Plains (Climatic Division 1 on Figure 2.3-203) occupies most of the Texas Panhandle and is defined on the north and west by the state boundaries and on the east by the Caprock Escarpment. The High Plains are about as flat as the coastal plains of Texas. The major cities of the High Plains are Amarillo, Lubbock, and on the margin of the Trans-Pecos, the neighboring cities of Midland and Odessa. Much of the High Plains is underlain by the Ogalalla Aquifer, which supplies a large but dwindling water supply to the area's irrigated agriculture. The High Plains are divided in two by a valley carved by the Canadian River, and a branch of the Red River, which has created the dramatic Palo Duro Canyon. Elsewhere, the High Plains are pockmarked with shallow, intermittent lakes and an occasional district where sand dunes have been set in motion by the wind (Reference 2.3-204).

The Low Rolling Plains (Climatic Division two on Figure 2.3-203) are largely rangeland, consisting of grasslands interspersed with forests of mesquite, a short, invasive tree with sweet-smelling wood but sparse shade. They lie east of the High Plains and include the cities of Abilene and Wichita Falls. While surface waterways are much more numerous than in the High Plains, lakes are much less frequent, as the land drops steadily toward the east. Many soils are quite red, and the runoff from this area helps give the Red River its name. This area has the greatest frequency of tornadoes in Texas (Reference 2.3-204).

The Cross Timbers (Climatic Division three on Figure 2.3-203) are also a mixture of grasslands and forest, although the forest includes oak and other species besides mesquite. The greater biological diversity among trees is attributable to higher precipitation totals and slightly warmer temperatures, along with soil variations. Like the Low Rolling Plains, the Cross Timbers slope mainly from west to east. Most lakes are man-made. Fort Worth and Temple are prominent cities on its eastern edge, while Austin, the state capital, sits at the intersection of the Cross Timbers, the Blackland Prairies, and the Edwards Plateau (Reference 2.3-204).

The Edwards Plateau (Climatic Division six on Figure 2.3-203) lies south of the High Plains, Low Rolling Plains and Cross Timbers, and east of the Trans Pecos. Its southern margin is the Balcones Escarpment, and the region includes both the relatively flat plateau area as well as the high-relief plateau margin where some of the most rugged terrain in Texas (known as the Hill Country) is located. The area is underlain by limestone formations, and many dramatic caves are located here. The vegetation varies from grasslands in the west to forests in the east, with pockets of maple and cypress hundreds of mi from their normal ecosystems. The most prominent city is San Angelo, but its eastern margin abuts Austin and San Antonio and various bedroom communities have developed, attracting people from the cities with its scenic ruggedness and slightly cooler summertime

temperatures. Various spring-fed rivers and streams originate along the Balcones Escarpment (Reference 2.3-204).

The Blackland Prairies (Climatic Division seven on Figure 2.3-203) are defined by several strips of rich, black soil that extend from San Antonio to Dallas and beyond and eastward to the Piney Woods. Most of the Blackland Prairies are occupied by farming operations, but in the 18th and 19th centuries the prairies formed easy corridors for long-distance travel from west to east. Now, the primary transportation corridor is along the western edge of the Blackland Prairies, along Interstate 35 and a string of major cities from San Antonio to Austin, Temple, Waco, Dallas and Sherman. The prairies are generally flat or rolling, and are devoted primarily to non-irrigated agriculture (Reference 2.3-204). The Post Oak Savannah lies mainly east of the Blackland Prairies, but is interlaced with the Prairies in a few areas. The Savannah was a fire-driven ecosystem, with oak trees underlain by grasslands. Now the territory consists of a mosaic of oak woods, tree-studded fields, and open grazing areas, with farming confined mainly to sediment-filled river valleys. The Post Oak Savannah includes Bryan/College Station, home of Texas A and M University (Reference 2.3-204).

The Piney Woods (Climatic Division four on Figure 2.3-203) are the westernmost portion of the mixed evergreen-deciduous forest belt that stretches westward across the Deep South from the Carolinas. The wide variety of trees is dominated by pine and oak, resting on fast-draining, sandy soils. Occasional cleared grasslands are outnumbered by productive forests, both public and private. In the interior of this region is the Big Thicket, a combination of uplands and lowlands with a rich diversity of plant species. In the Piney Woods, tall pines, prickly pear cactus, and palmetto exist side by side. Population centers include Longview, Tyler and Texarkana (Reference 2.3-204).

The Gulf Coastal Plain (Climatic Division eight on Figure 2.3-203) is primarily a combination of prairies and marshes. Behind the barrier beach is a set of lagoons and estuaries that form a rich habitat for migratory and resident birds, including a major wintering area for the endangered whooping crane. While tornadoes and floods are the primary weather hazards in the rest of the state, the Gulf Coastal Plain is most vulnerable to hurricanes. Major cities along the coastal plain include Houston, Beaumont, Victoria, Corpus Christi and, on a barrier island, Galveston (Reference 2.3-204).

The South Texas Plains (Climatic Division nine on Figure 2.3-203) are largely arid and treeless. The largest ranch in Texas, the King Ranch, is here. Widespread areas are covered with dense thickets of subtropical brush. San Antonio is along the northern margin of this region, while Laredo is in its southwestern corner (Reference 2.3-204).

The Lower Rio Grande Valley (Climatic Division 10 on Figure 2.3-203) is the smallest geographical area described. It consists of alluvial plains that are under widespread irrigated agriculture. The salt and freshwater marshes and other plant communities host a wide variety of tropical and temperate species of birds, many

of which pass through the area en route between North and Central America. The area, often known simply as “the Valley”, is a popular wintering area for residents of the central United States. Primary cities include Brownsville and McAllen (Reference 2.3-204).

The climate of Texas is determined by geographical features external to the state. To the southeast, the Gulf of Mexico provides a source of warm, moist air throughout the year. During the summer, the entire state comes under the influence of the Gulf of Mexico, as southeasterly and southerly winds settle into place. Air approaching Texas from the Gulf of Mexico may have a long history of being over the tropical waters of the Caribbean and the Atlantic, or it may recently have moved offshore from the southeast United States. The latter circumstance leads to air that is more polluted and in summertime is hazier. Tropical Atlantic air is relatively clean and visibility tends to be excellent despite the high humidity. Except for the Trans-Pecos, most of the water that falls as precipitation in Texas has entered the state from the Gulf coast (Reference 2.3-204).

The second climate maker is the Mexican High Plain, or Altiplanicie Mexicana. This arid, high-altitude plateau region extends northward from Mexico City nearly to the U.S. border. Rarely does the air flow originating from the Mexican High Plain reach ground level in Texas except in the Trans-Pecos region, but it influences the weather throughout the state. When surface winds in Texas are from the south or southeast, winds 10,000 ft aboveground are normally from the southwest. Thus, low-level air from the Gulf of Mexico is overlaid with warmer, drier air from the Mexican High Plain. Close to the Mexican border, this warm air ‘caps’ the humid Gulf of Mexico air, preventing thunderstorm activity and trapping the humid air close to the ground. As the air masses precede north, particularly during the spring and fall, they progressively move beneath cooler air aloft. While the humid low-level air becomes more unstable, it still cannot convect because of the capping inversion. Eventually, if a frontal system or other disturbance causes larger-scale ascent, the Mexican High Plain air can cool enough to eliminate the cap, suddenly allowing vigorous thunderstorm activity to take place. The combination of the Gulf of Mexico and the Altiplanicie Mexicana makes Texas and the southern Great Plains the worldwide hot spot for severe convection and tornadoes (Reference 2.3-204).

The third climate maker is the Rocky Mountains. Arizona, New Mexico and west Texas form one of two relative gaps in the Rocky Mountain Cordillera; the other is along the U.S.- Canadian border. Westerly winds often blow through this gap, but the Rockies form a broad barrier to westerlies for the rest of the state. In the eastern half of Texas, the least likely wind direction is from the west. The Rockies also block air from moving across them from the east. In particular, cold air masses that reach the United States from the north cannot easily spread westward and instead are funneled southward parallel to the mountains. Such cold air reaches farther south into Texas and beyond than anywhere else on the continent. Nevertheless, it is rare for bitterly cold air to reach the Lower Rio Grande Valley, allowing grapefruit to be one of the area’s largest cash crops (Reference 2.3-204).

Precipitation is not evenly distributed over the state, and variations in precipitation at any one locale from year to year are apt to be pronounced. The mean annual precipitation varies from a statewide maximum of 59.20 in at Orange, in the lower Sabine River valley of East Texas, to a minimum of 7.82 in at El Paso, at the western tip of the state (Reference 2.3-202). The annual average precipitation map for Texas is shown on Figure 2.3-204. This figure shows the decrease in precipitation going from East to West. The mean annual rainfall distribution correlates roughly with longitude and varies little from north to south across Texas. Generally, annual precipitation decreases about an inch for each 15-mi displacement from east to west. West Texas is the driest region in the state, with an average annual region wide precipitation of 11.65 in, while the Upper Coast (45.93 in) and East Texas (44.02 in) are the wettest. At most locations rainfall for any single month will vary appreciably from the norm. Likewise, the number of days with precipitation usually is significantly abnormal. Moreover, the number of "rain days" follows the general trend of rainfall totals in that seasonal frequencies of rain days are lowest when rainfall totals are lowest. The mean number of days in January with at least 0.1 in of precipitation varies from seven in East Texas to one or fewer in the Trans-Pecos; in July rain days normally are as numerous in the mountainous Trans-Pecos as in East Texas and along the upper coast. Particularly in the western half of Texas, one or two rainstorms often account for nearly all of a month's rainfall. The wet season does not occur at the same time of year in all parts of Texas. Intense and prolific thunderstorms, often moving in "squall lines," roam much of Texas in the late spring; Central, North, and East Texas receive their maximum rainfall in May. The warmest time of year is also the wettest for the High Plains and Trans-Pecos; nearly three-fourths of the total annual precipitation in these regions occurs from May to October. Tropical weather disturbances ensure that the late summer and early autumn are the two wettest periods for the part of Texas within 100 mi of the Gulf of Mexico (Reference 2.3-202). The annual average precipitation for each of the ten Texas climate divisions for the period 1895 through 2005 is shown in Figure 2.3-205. This figure also shows the percent deviation from the annual average for each of the ten divisions. The annual average for climate division three, which includes the CPNPP site, is 34.3 in.

Winter is the driest time of the year in nearly all of Texas. The exception is East Texas, where rainfall typically is the least substantial in July and August. December or January is normally the driest month on the High and Low Rolling Plains, as well as on the Edwards Plateau. The dry season peaks somewhat later farther east in north central and south central Texas, while on the coastal plains February is the driest month. Early spring (March – April) is normally very dry in the Trans-Pecos; in fact, in this semiarid region, rainless spells often last several weeks at a time, and two or even three months can elapse without significant rain. Because much of the annual rainfall occurs quickly, excessive runoff often leads to flooding. The broad, flat valleys in the eastern half of Texas, where mean annual rainfall exceeds 25 to 30 in, sustain comparatively slow runoff. When rain is heavy, these valleys store vast amounts of water before slowly releasing it into the streams. The resulting flat-crested, slow-moving flood in the lower basins causes protracted periods of inundation. By contrast, in the western half of Texas,

where ground and tree cover is sparse and stream slopes are typically quite steep, high-intensity rains produce rapid runoff that frequently leads to flash flooding. The area along the Balcones Escarpment (from Austin south to San Antonio, then west to Del Rio) is one of the nation's three most flash-flood-prone regions (Reference 2.3-202).

Snowfall occurs at least once every winter in the northern half of Texas, although accumulations rarely are substantial except in the High Plains. Snow is not uncommon in the mountainous areas of the Trans-Pecos, though heavy snows (five in or more) come only once every two or three winters. More often than not, snow falling in the southern half of the state melts and does not stick to the surface; snow stays on the ground only once or twice in every decade. Snowfall rarely is observed before early November and hardly ever occurs after mid-April. Where it is not uncommon, snow is almost always heaviest in either January or February. Mean seasonal snowfall is 15-18 inch in the Texas Panhandle and 4 – 8 in elsewhere in the High and Low Rolling Plains (Reference 2.3-202).

Temperatures vary considerably among the ten climatic regions of Texas. Few or no areas of Texas escape freezing weather in any winter. On the other hand, the heat of summer is intense everywhere. Whereas precipitation varies longitudinally across Texas, mean annual temperature varies latitudinally. On a year-around basis, readings are the coolest in the extreme north and warmest in the far south. In mid-winter the mean daily minimum temperature varies between the upper teens in the northern periphery of the Panhandle and the low fifties in the lower Rio Grande valley; afternoon highs range from the upper forties in the extreme north to near seventy in the far south. Conversely, summer lows in the Panhandle average in the low sixties and, in the lower Valley, in the middle to upper seventies; daytime highs reach into the low nineties in both regions. All-time temperature extremes in Texas include: -23° F at Tulia (1899) and Seminole (1933) and 120°F at Seymour (1936) and Monahans (1994) (Reference 2.3-202). Other Texas weather records are given in Table 2.3-201. The annual average maximum daily temperature map for Texas is shown on Figure 2.3-206 based on data from 1971 through 2000. This figure shows an annual average maximum of 76°F near the CPNPP site. Extended periods (more than one or two days) of subfreezing highs are rare, even in the far north. However, parts of the Panhandle generally have subfreezing temperatures for many successive winter nights. The mean number of days with freezing temperatures in the northern High Plains is 120. In this region the first autumn freeze ordinarily occurs at the end of October, and the last freeze in spring takes place in mid-April. The "freeze-free" season lengthens with distance north-to-south down the state. The mean number of days with freezes is forty to fifty-five in north central Texas and twenty to twenty-five in south central Texas. In some years the temperature never reaches the freeze level in the Valley. Even when it does, it almost always remains below 32° F for only four to six hours or less, usually around sunrise (Reference 2.3-202).

All of the Texas coastline is subject to the threat of hurricanes and lesser tropical storms during the summer and autumn. Vulnerability reaches a maximum during August and September, the height of the hurricane season in the Gulf of Mexico

and Caribbean Sea. Hurricanes strike the Texas coast an average of one every three yr. Inland, hurricanes cause damage due to high winds, including tornadoes, and flooding from excessive rainfall. Persons along the coast must also contend with storm tides (Reference 2.3-202).

Although tornadoes can occur anytime, most of them materialize during April, May, and June. In a normal year, about 130 tornadoes are sighted in Texas, 30 percent of which occur in May. On average, about 200 people are hurt and a dozen are killed annually by the tornadoes. Tornadoes are most likely to occur along and south of the Red River between Lubbock and Dallas; they are least likely in the Trans-Pecos. Thunderstorms occur in every month of the year, though least in winter. With an average of sixty thunderstorm days a year, East Texas is most susceptible to the severe localized phenomena fostered by the storm (hail, high winds, flash flooding). The mean annual number of thunderstorm days diminishes from east to west across Texas; the Trans-Pecos has only about forty such days each year. The lower Valley has fewer still (thirty). The peak hail frequency statewide is in May. Most hailstorms are short-lived, however, because the macroscale weather systems (such as squall lines) that generate hail move rapidly. Hailstones are usually largest in the High Plains, where hail the size of tennis balls-even baseballs-is not uncommon in the summer. Sunshine is most abundant in the extreme west, where El Paso receives an average of 80 percent of the total possible sunshine annually. Cloud cover is most prevalent along the coast, especially in the Upper Coast, where the mean annual sunshine amounts to only about 60 percent of possible sunshine hours (Reference 2.3-202).

Weather stations in the region surrounding the CPNPP site are shown on Figure 2.3-207. The closest weather stations to the CPNPP site are: Dublin, Glen Rose, Cleburne, Benbrook, Dallas Fort Worth Airport, Dallas Love Field Airport, Mineral Wells Airport, Weatherford, and Stephenville. Based on data for the period 1971 – 2000 for Dallas Fort Worth Airport, Dallas Love Field Airport, Mineral Wells Airport, and Glen Rose the mean daily maximum temperature is 77.6°F and the mean daily minimum temperature is 54°F. The lowest daily minimum is -15°F and the highest daily maximum temperature is 115°F. The annual average precipitation is 34.6 in. Monthly data from these stations are given in Tables 2.3-202 through 2.3-205. From data collected at the Dallas Fort Worth Airport, the Mineral Wells Airport, and the CPNPP site the typical wind direction for the region is from 147 degrees (Figures 2.3-208 through 2.3-210), the average wind speed is 10.5 mph. The frequency of snowfall in this region is so low that the average annual snowfall is near zero. The frequency of sleet and freezing rain is discussed in the following sections along with the regional dewpoint/relative humidity.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 General

Meteorological data are presented in this subsection for severe weather phenomena such as hurricanes, tornadoes, thunderstorms, lightning, hail, high air

pollution, and ice storms. Also presented are the meteorological data used for evaluating the performance of the ultimate heat sink and design basis tornado parameters.

The interplay between synoptic scale phenomena and topography is small in the region surrounding the site. The effect of terrain features on synoptic scale flow can readily be ascertained when a larger area, which takes in the high country of West Texas and Eastern New Mexico, is included; i.e., the principal effect is that the high country forms a natural barrier to the flow of air. Consequently, moist tropical air from the Gulf of Mexico and air from the arctic or polar sources, which flows uninhibited through the site region, is effectively blocked from the areas to the west of the mountains. The net result is wide fluctuations in rainfall, humidity, and annual sunshine over the larger area. Severe weather in the region is usually associated with heavy thunderstorms (including tornadoes) and tropical cyclones. Property damage occurs from flooding and high winds. Damaging hail also occasionally occurs in the site region (Reference 2.3-205).

Extreme weather calculations for CPNPP were conducted over the maximum data span available. Certified climatological data obtained from the U.S. National Climatic Data Center was used for the severe weather phenomena evaluations. This data selection supports accurate severe weather phenomena projections for the area in the vicinity of CPNPP site. This extensive historic data record provides the historical climatic trends and severe natural phenomena to be included in the site characterization.

Dry-bulb, coincident wet-bulb, and non-coincident wet-bulb temperatures represent significant site characteristics because this data is used in demonstrating that the US-APWR DCD site parameters are bounding (i.e., more conservative). The CPNPP site characteristic temperatures were developed by considering both 100-year return temperatures and 0 percent exceedance temperatures. These values were calculated using a 30-year sequential hourly meteorological data set for the Dallas/Fort Worth Airport National Weather Service station. The difference between the CPNPP site characteristics and the DCD site parameters, used for design, provide additional margin to the selected CPNPP site characteristics. This margin accounts for variations due to limitations in the accuracy, quantity, and period of time in which the historical data have been accumulated.

General predictions on global or U.S. climatic changes expected during the period of reactor operation are uncertain and are currently only applicable on a macroclimatic scale. Because the maximum data span available (i. e., representative of the microclimate near the CPNPP site) was used in the severe weather analysis, accurate severe weather phenomena projections are provided based on historic data. Projection of future climatological conditions at the CPNPP site are speculative at best, based on current understanding and modeling of global climate change.

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 such changes are difficult to predict reliably. Even the most reliable climate change models are not capable of accurately predicting design basis extremes in weather patterns.

A discussion of speculations about climate change would not resolve current metrological and geophysical modeling inadequacies. Discussion of changes in average global trends will not result in data that can be reviewed on a site-specific basis with any degree of accuracy or reliability. It is relatively easy to demonstrate that an increase in the average value of temperature (or precipitation) at a given location is much more likely to be a result of numerous increases in temperatures (or precipitation) in the "normal range" rather than increases in extreme values, because a change in a select number of extreme values will essentially have no measurable effect on longer term average values. Therefore, the information presented in this subsection of the FSAR is focused on the extreme meteorological conditions that will facilitate a plant design that will operate within these safety margins throughout the projected plant life of 40 to 60 years. This is accomplished by identifying historical extremes and projecting, in a scientifically defensible manner, the potential effects weather will have on the safety and operation of CPNPP Units 3 and 4.

2.3.1.2.2 Hurricanes

Hurricanes and tropical storms are among the most devastating naturally occurring hazards in the United States. A tropical cyclone is defined as a low-pressure area of closed circulation winds that originates over tropical waters. A tropical cyclone begins as a tropical depression with wind speeds below 39 mph. As it intensifies, a tropical cyclone may develop into a tropical storm with wind speeds between 39 mph and 74 mph. When wind speeds go beyond 74 mph, the tropical storm is known as a hurricane. The Gulf of Mexico and the Atlantic Coast areas are the most susceptible to tropical cyclones (Reference 2.3-224).

Based on data from NOAA Technical Memorandum NWS SR-206 (Reference 2.3-206) and data for 2004 – 2006 from the National Hurricane Center (Reference 2.3-234), the number of tropical storms and hurricanes affecting Texas from the period of 1899 through 2006 was 39. The storms that have affected Texas are listed in Table 2.3-206 along with the date and storm category. Based on these data, the storm return period is 2.8 yr as shown in Table 2.3-207. This table also provides the Saffir/Simpson storm category definitions and gives a breakdown of storms by month and storm category. There have been no category five storms and only six category four storms affecting Texas. August and September have the most storms with approximately 60 percent of the storms occurring in these months. Figure 2.3-211 gives the tropical cyclone frequency and intensity along the U.S. coastline based on data from 1871 through 1998. This figure shows a

relative Texas peak in frequency at Galveston. Using this peak, the frequency of tropical storms and hurricanes at Galveston is approximately 43 per 100 yr or a return period of 2.3 yr. Considering hurricanes only the return period increases to four yr. For major hurricanes, the return period is 12.5 yr. These results are in good general agreement with the data from SR-206 given in Table 2.3-207. Figure 2.3-212 gives the number of hurricanes as a function of wind speed. As expected, the hurricane frequency decreases with wind speed. For a wind speed of 125 knots (144 mph) the return period is given as 10 yr.

The number of tropical storms passing within 50 statute mi of the CPNPP site are listed on Table 2.3-208 and shown on Figure 2.3-213. These data, obtained from the NOAA Coastal Services Center, show that only one hurricane, in 1900, passed within 50 mi of the site during the period 1851 – 2006. There appears to be a connection between hurricane frequency and El Nino and La Nina events. El Nino events tend to suppress the formation of hurricanes by steering the subtropical jet stream into the hurricanes' path and shearing off the tops of the storms before they develop into full intensity. During La Nina episodes, the jet stream moves north, and hurricanes tend to more easily evolve without interference. The tropical cyclone season for Texas extends from June to October; storms are more frequent in August and September, and rarely occur after the first of October. The average frequency of tropical cyclones with hurricane force winds, i.e., winds greater than 74 mph, that affected Texas during the period 1899 – 2002 is approximately one every three yr (Reference 2.3-207).

After a hurricane or tropical storm makes landfall, it begins to break apart, although remnants of the storm can continue moving inland. These remnants have been known to bring heavy precipitation, high winds, and tornadoes to locations near the CPNPP site. For instance, a remnant of the September 1900 Hurricane that devastated Galveston made its way into north central Texas, where it produced heavy rains. In 1934, a tropical disturbance moved inland along the middle Texas coast and eventually found its way to Kaufman County, where it caused damage from straight-line winds. In 1981, the remnants of Pacific Hurricane Norma came across north central Texas, bringing torrential rain (10-13 in between Denton and Bridgeport) and a few weak tornadoes (Reference 2.3-224). In 1995, the remnants of Tropical Storm Dean brought heavy rain to Hood and Somervell counties and 6 to 10 in of rain fell near Glen Rose (Table 2.3-205).

Tropical cyclones including hurricanes lose strength rapidly as they move inland, and the greatest concern is potential damage from winds or flooding due to excessive rainfall. Figure 2.3-214 shows the decay of tropical cyclone winds after landfall. As seen, only the fastest moving storms will maintain any significant wind speed by the time they reach the CPNPP site. From this figure, a tropical cyclone with 86 mph winds traveling at 18 mph will have dissipated to less than 40 mph at the CPNPP site.

In a paper by Kaplan and Demaria, the decay of tropical cyclone winds after landfall was evaluated. The wind speed after landfall is given by the following inland wind decay model:

$$V(t) = V_b + (RV_o - V_b)e^{-\alpha t} - C$$

Where:
V(t) is the wind speed as a function of time,
V_b is 26.7 kt,
R is 0.9,
α is 0.095 hr^{-1},
t is the time after landfall, and
C is a correction factor to account for the inland distance. Where:

$$C = m\left[ln\left(\frac{D}{D_0}\right)\right] + b$$

Where:
D in the inland distance in kilometers,
D_o is 1 km,
$m = c_1 * t(t_0 - t)$,
$b = d_1 * t(t_0 - t)$,
c_1 = 0.0109 kt/hr^2,
d_1 = -0.0503 kt/hr^2, and
t_o = 50 hr.

Assuming a maximum landfall wind speed or 208 kt (~240 mph), a translational velocity of 16 kt (18.4 mph), and a distance of 400 miles from the CPNPP site to Galveston, gives a maximum possible wind speed of 61 mph at the CPNPP site. This should be considered as the upper bound of possible hurricane wind speed at the CPNPP site.

The Probable Maximum Hurricane (PMH) is discussed in CPNPP UFSAR Subsection 2.3.1.2.2. For the CPNPP site, the PMH sustained (10-minute average) wind speed at 30 ft aboveground is 81 mph (Reference 2.3-205).

2.3.1.2.3 Tornadoes

During the period January 1, 1950 through July 31, 2006, 158 tornadoes (mean annual frequency of 2.8/yr) occurred within Somervell County and the surrounding counties (Bosque, Erath, Hood, and Johnson) (Reference 2.3-225). It should be noted that statistical data on severe local storms, tornadoes particularly, are highly dependent on human observation. For example, as population density increases, the number of tornado occurrences observed and accurately reported generally increases. However, tornadoes that cross county lines may be counted twice due to this increase in reporting.

The probability that a tornado will occur at the CPNPP site is low. Records show that in a 56-yr period (1950 – 2006) there were three tornadoes reported in Somervell County, the location of the site (Reference 2.3-225). The data reported by the NOAA's National Environmental Satellite, Data, and Information Service (NESDIS) (Reference 2.3-225) are given in Tables 2.3-209 and 2.3-210. From these data, the average tornado area in Somervell and the surrounding counties, ignoring events with a zero path length, is approximately 0.21 sq mi. Using the principle of geometric probability described by H. C. S. Thom (Reference 2.3-208), a mean tornado path area of 0.21 sq mi, and an average tornado frequency of 2.79 per year for this area (3414 mi^2), the point probability of a tornado striking the plant is $1.7x10^{-4}$/yr. This corresponds to an estimated recurrence interval of 5881 yr.

The tornadoes reported during the years 1950 – 2006 in the vicinity of the site (Bosque, Erath, Hood, and Johnson Counties) are shown in Tables 2.3-209 and 2.3-210. During this period, a total of 158 tornadoes touched down in these counties that have a combined area of 3414 sq mi (Reference 2.3-209). These local tornadoes have a mean path area of 0.21 sq mi excluding tornadoes with a zero length or without a length specified. The site recurrence frequency of tornadoes can be calculated using the point probability method as follows:

Total area of tornado sightings = 3414 sq mi

Average annual frequency = 158 tornadoes/56.58 yr = 2.79 tornadoes/yr

Annual frequency of a tornado striking a particular point P = ([0.21 mi^2/tornado] [2.79 tornadoes/yr]) / 3414 sq mi = 0.00017 yr-1

Mean recurrence interval = 1/P = 5883 yr

This result shows that the frequency of a tornado in the immediate vicinity of the site is low. However, the frequency increases northward until “tornado alley” is entered north of Dallas. Another methodology for determining the tornado wind speed and associated strike probability at the CPNPP site is given in NUREG/CR-4461 (Reference 2.3-210). Based on a 1 degree longitude and latitude box centered on the CPNPP site, the number of tornadoes is 246 between 1950 and 2003. The corresponding expected maximum tornado wind speed and upper limit (95 percentile) of the expected wind speed based on a 2 degree longitude and latitude box centered on the CPNPP site are given below with the associated probabilities.

Probability	Expected maximum tornado wind speed (mph)	Upper limit (95 percent) of the expected tornado wind speed (mph)
10^{-5}	141	146
10^{-6}	178	184
10^{-7}	205	217

In the area north of about 34 degrees north latitude, there is a greater frequency of large tornadoes with wide paths and long trajectories.

Based on the approximately 56-yr period of record from 1950 through 2006, the mean seasonal and annual number of tornado occurrences for the area around the site are (Reference 2.3-225):

Winter	0.14
Spring	1.73
Summer	0.37
Autumn	0.57
Annual	2.81

The design basis tornado parameters used in the design and operation of CPNPP are based on Revision 1 of Regulatory Guide 1.76. For Region I, as described in RG 1.76, the design parameters are listed below:

Translational Speed	46 mph (21 meter/sec)
Rotational Speed	184 mph (82 meters/sec)
Maximum Wind Speed (sum of the translational and rotational speed)	230 mph (103 meters/sec)
Radius of Maximum Rotational Speed	150 ft (45.7 meters)
Maximum Pressure Drop	1.2 psi (83 mb)
Rate of Pressure Drop	0.5 psi/sec (37 mb/sec)

Compliance with Regulatory Guide 1.76 is discussed in Section 1.9. Tornado loadings are discussed in Subsection 3.3.2.

Waterspouts are common along the southeast U.S. coast, especially off southern Florida and the Keys and can happen over seas, bays, and lakes worldwide. However, waterspouts are not expected to occur at the CPNPP site because the only nearby bodies of water are Squaw Creek Reservoir (SCR) and Lake

Granbury. The small size of these lakes does not produce the conditions conducive to waterspouts.

2.3.1.2.4 Thunderstorms

Thunderstorms, from which damaging local weather can develop (tornadoes, hail, high winds, and flooding), occur about 16 days each year based on data from the counties surrounding the site (Reference 2.3-225). The maximum frequency of thunderstorms and high wind events occurs from April to June, while the months from November through February have few thunderstorms. The distributions of thunderstorms and high wind events by county are displayed in Table 2.3-211.

2.3.1.2.5 Lightning

Data on lightning stroke density is becoming more readily available due to the National Lightning Detection Network (NLDN), which has measured cloud to ground lightning for the contiguous United States since 1989. Prior to the availability of these data, isokeraunic maps of thunderstorm days were used to predict the relative incidence of lightning in a particular region. A general rule, based on a large amount of data from around the world, estimates the earth flash mean density to be 1-2 cloud to ground flashes per 10 thunderstorm days per sq km (Reference 2.3-211). The annual mean number of thunderstorm days in the site area is conservatively estimated to be 48 based on interpolation from the isokeraunic map (Reference 2.3-212); therefore it is estimated that the annual lightning stroke density in the CPNPP site area is 25 strikes/sq mi/yr. Other studies gave a ground flash density, (GFD) (strikes/km^2/yr), based on thunderstorm days per year (TSD) as GFD = 0.04 $(TSD)^{1.25}$ = 0.04 $(48)^{1.25}$ = 5 strikes/km^2/yr or 13 strikes/mi^2/yr (Reference 2.3-213).

Recent studies based on data from the National Lightning Detection Network (NLDN) (Reference 2.3-214) indicate that the above strike densities are upper bounds for the CPNPP site. Mean annual flash density given in Huffines and Orville (Reference 2.3-214) for 1989 – 96 is 3 to 5 strikes/km^2/yr or 13 strikes/mi^2/yr in North Central Texas.

2.3.1.2.6 Hail

Almost all localities in Texas occasionally experience damage from hail. While the most commonly reported hailstones are 1/2 to 3/4 inch in diameter, hailstones 3 to 3-1/2 inch in diameter are reported in Texas several times a year. (Reference 2.3-205)

During the period January 1, 1950 through March 31, 2007 there were 707 reports of large hail (3/4 in diameter or larger) occurrences within the five county area (Somervell, Bosque, Erath, Hood, and Johnson) around the site (Reference 2.3-225). This gives a mean annual frequency of 12.3 hailstorms per year for this

area. Fortunately, recurrence of damaging hail at a specific location is very infrequent.

The total number of large-hail occurrences (3/4 in diameter or larger) for the five county area around the CPNPP site is given in Table 2.3-212. The average number per year for this area is also provided. Damaging hailstorms are most frequent during April, May, and June, the period of severe-thunderstorm activity.

2.3.1.2.7 Air Pollution Potential

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

The newly promulgated EPA 8-hour ozone standard (62 FR 36, July 18, 1997) is 0.08 ppm in accordance with 40 CFR 50.10 (Reference 2.3-226). Somervell County is in attainment for all criteria pollutants (carbon monoxide, lead, nitrogen dioxide, particulate matter ([PM_{10}, particulate matter less than 10 micron], [$PM_{2.5}$, particulate matter less than 2.5 micron]), ozone, and sulfur oxides. There are nine counties (or parts of counties) north and northeast of Somervell County that are in non-attainment with the 8-hour ozone standard (Reference 2.3-227). Texas non-attainment areas are shown on Figure 2.3-381. Currently designated (as of March 2, 2006) non-attainment areas in this region of Texas for the criteria pollutants are as follows:

TEXAS (Region VI)

Dallas - Fort Worth, TX (Moderate)

Collin Co [(a)] [(b)]

Dallas Co [(a)] [(b)]

Denton Co [(a)] [(b)]

Ellis Co

Johnson Co

Kaufman Co

Parker Co

Rockwall Co

Tarrant Co [(a)] [(b)]

a) area has whole or part county or counties in a previous 1-hr Ozone nonattainment area (as of June 15, 2005) no longer subject to the 1-hour standard

b) area has whole or part county or counties in a CO, PM-10, or PM-2.5 nonattainment or maintenance area or previous 1-hr Ozone nonattainment or maintenance area (as of June 15, 2005)

The ventilation rate is a significant consideration in the dispersion of pollutants. Higher ventilation rates are better for dispersing pollution than lower ventilation rates. The atmospheric ventilation rate is numerically equal to the product of the mixing height and the wind speed within the mixing layer (Reference 2.3-228).

Conditions in the region generally favor turbulent mixing. Two conditions which reduce mixing, increasing the air pollution potential, are surface inversions and stable air layers aloft. The surface inversion is generally a short-term effect and surface heating on most days creates a uniform mixing layer by mid-afternoon. On the other hand, if warming caused by subsiding air occurs, the second condition, namely a subsidence inversion, may result. Because both conditions usually occur in conjunction with light winds, the air pollution potential is amplified (Reference 2.3-205).

Holzworth (Reference 2.3-215) has computed mean morning and afternoon mixing depths and corresponding wind speeds for several stations in Texas and plots of morning and afternoon mixing heights and wind speeds. The data from these plots are given in Table 2.3-213 for the CPNPP vicinity. There is considerable variation in mixing depths among Texas stations; but the mixing depths all display similar seasonal variation, the depth being greatest during the warm months and shallowest during the cold months. Holzworth also provides isopleths of the total number of forecast-days of high air pollution potential in five yr. Figure 2.3-215 shows that the number of high air pollution days in five yr for this region is zero.

Mixing height data for Stephenville are given in Tables 2.3-214 and 2.3-215. Table 2.3-214 gives the seasonal morning and afternoon mixing heights. This table shows that there is reasonable agreement with the earlier data provided by Holzworth. Comparison with the Holzworth data indicates that the morning mixing heights at Stephenville are higher in winter and lower in summer. The Stephenville afternoon mixing heights are highest in the spring and summer, which generally agrees with the Holzworth data. The mean morning and afternoon ventilation rates for Stephenville is given in Table 2.3-215. Mixing height data were also obtained from the Ventilation Climate Information System (Reference 2.3-232) and are presented in Table 2.3-216 on a monthly basis along with the wind speed, and ventilation index. These data indicate that stable periods with light wind conditions are generally of short duration in the region. Based on data from 1981 through 1989, the Ventilation Climate Information System provides the daily and

annual variability of the mixing height on a monthly basis. These data are provided in Figures 2.3-216 through Figure 2.3-239. The monthly morning mixing height variability is given on Figure 2.3-240 and the afternoon on Figure 2.3-241. The average monthly morning and afternoon mixing heights are shown on Figure 2.3-242.

Based on a 50-yr period of record (1948 – 1998), Wang and Angell (Reference 2.3-216) tabulated the number of times stagnating anticyclones persisted for four or more days. Occurrences of stagnation were determined primarily on the basis of a surface pressure-gradient analysis. In the general area of the site, the mean duration was five days and the mean annual frequency was five percent of the days annually (Figure 2.3-243). The mean annual days of stagnation was 20 and there were four cases per year exceeding four days duration (Figure 2.3-244). The number of air stagnation days was highest in July and August, with six days each (Figure 2.3-245). The other months subject to air stagnation (June, September, and October) had two, four, and three stagnation days, respectively (Figure 2.3-245). The air stagnation trend for this general area is negative (Figure 2.3-246) over the 50-yr period of record.

2.3.1.2.8 Precipitation

Historic precipitation data covering the period of 1971 through 2000 for the Dallas Fort Worth Airport, Dallas Love Field, Mineral Wells Airport, and the Glen Rose weather stations are given in Tables 2.3-202 through 2.3-205. The annual average and maximum 24-hour rainfall for these stations are given in Table 2.3-347.

The maximum 24-hr rainfall for Glen Rose was associated with Tropical Storm Dean.

Maximum rainfall, estimated by statistical analysis of regional precipitation data, is given in Table 2.3-217 for return periods of one to 100 yr and for rainfall durations of from five minutes to ten days. These data were taken from NOAA Technical Memorandum NWS Hydro-35 (Reference 2.3-217), National Weather Service Technical Paper No. 40 (Reference 2.3-218), and National Weather Service Technical Paper No. 49 (Reference 2.3-219). Figure 2.3-247 gives a comparison of the monthly rainfall for representative regional weather stations covering the period of 1971 through 2000. This figure shows that the peak rainfall (~5 in) is in May for all referenced weather stations. A secondary peak (~4 in) occurs in October for these weather stations.

Probable Maximum Precipitation (PMP), sometimes called maximum possible precipitation, for a given area and duration is the depth which can be reached but not exceeded under known meteorological conditions. For the site area, using a 100-yr return period, the PMP for 6, 12, 24, and 48 hours is 6.9, 8.3, 9.5, and 11.0 in, respectively (Table 2.3-217).

Drought is considered by many to be a normal condition in Texas. In every decade of the last century, Texas was a victim of one or more serious droughts. The

drought of the 1930s caused significant declines in rangeland production, which was thought to have never fully recovered to pre-drought conditions. The severe to extreme drought that affected every region of Texas in the early to mid-1950s was the most serious drought to strike Texas in recorded weather history. In fact, the drought reached its worst in the late summer of 1956 in North Central Texas (Reference 2.3-224).

Texas experiences so many droughts in part because of it location along 30 degrees north latitude, a climate transition zone called the Great American Desert (the same latitude where many of the earth's deserts are found). A drought with a duration of three months is likely to occur in some part of the state every nine months. A drought with a duration of six months or longer will likely occur once every 16 months, and a drought with a duration of 12 months is likely somewhere in the state once every three yr. Over the past decade, in addition to the droughts in 1996 and 1998, Texas has also suffered droughts in 2000 and 2002. The duration of droughts in the North Central Texas Climatic Division between 1892 and 1994 ranged from 61 to 73 days. For this purpose, droughts have been arbitrarily defined as when the area has less than 75 percent of the 1931 – 1960 average precipitation (Reference 2.3-224).

The most severe drought of the 20th century in Texas occurred during 1954 – 1956. Fort Worth precipitation records, which illustrate the regional conditions, indicate that the average annual precipitation for this three-yr period was 21.1 in, with only 18.55 in occurring in 1956. Although this period represents the worst drought in Texas, there have been three occurrences of annual precipitation less than 18.55 in during the 81-yr period from 1895 through 1975 at Fort Worth. The extreme minimum annual precipitation recorded, 17.91 in, occurred in 1921 (Reference 2.3-205).

Historic snowfall data covering the period of 1971 through 2000 for the Dallas Fort Worth Airport, Dallas Love Field, Mineral Wells Airport, and the Glen Rose weather station are given in Tables 2.3-202 through 2.3-205. The annual average and maximum 24-hour snowfall for these stations are given below:

	Annual Average Snowfall (in)	Maximum 24-hr Snowfall (in) and Yr
Dallas Fort Worth	2.5	12.1 (1964)
Dallas Love Field	1.7	6.0 (1978)
Mineral Wells	1.8	4.0 (1978)
Glen Rose	1.8	4.5 (1973)

Snowfall records for Dublin for the period 1897 through 2005 are illustrated in Figure 2.3-248. Snowfall records for Weatherford for the period 1896 through 2005 are illustrated in Figure 2.3-249.

Ice storms, precipitation in the form of freezing rain and/or sleet, occur occasionally in the region during the period December through March. Ice storms recorded for the adjoining counties of Bosque, Erath, Hood, Somervell, and Johnson for the period 1950 through 2007 are listed in Table 2.3-218. These data show that the number of ice storms is slightly more than one per year for these counties.

The evaluations of ice thickness from freezing rain performed by American Lifelines Alliance, “Extreme Ice Thicknesses from Freezing Rain”, September 2004, (Reference 2.3-233) indicated that for the site area the ice thickness is one in with a 100-yr return period (Figure 2.3-250). Another study performed by the North Central Texas Council of Governments (NCTCOG) provided estimates of ice thickness with various return periods. Their results, based on a Weibull distribution, are given below (Reference 2.3-224):

Location (year of data)	2-yr estimate (in)	10-yr estimate (in)	50-yr estimate (in)	100-yr estimate (in)
Dallas Love Field (52)	0.35	1.04	3.05	4.86
Dallas Hensley Field (52)	0.38	1.01	2.67	4.07
Grapevine Dam (49)	0.48	1.07	1.67	1.93
Dallas-Fort Worth Int'l AP (28)	0.31	0.76	1.89	2.80
Eagle Mountain Lake (24)	0.33	1.01	3.12	5.06
Benbrook Dam (49)	0.45	0.85	1.24	1.42

The results from this analysis are considerably higher than those reported by the American Lifelines Alliance. This is attributed to the methodology employed by NCTCOG, which used a combination of precipitation and minimum temperatures as a surrogate for winter ice storms or as a measure of potential winter ice storms. Daily precipitation data were used in the analysis if the precipitation equaled or exceeded 0.25 in and the minimum temperature for that day and the previous day were below 33° Fahrenheit. The assumption was that if the minimum temperature were below 33° for the previous and current day, then precipitation would likely occur as ice or freezing rain resulting in a winter ice storm (Reference 2.3-224). This assumption may have resulted in an over-estimate of ice thickness when compared to actual observations. However, these results should provide an upper bound to the actual ice thickness.

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

Texas is not a heavy snow load region. ANSI/ASCE 7-05, "Minimum Design Loads for Buildings and Other Structures," (Reference 2.3-220) identifies that the ground snowload for the CPNPP area is 4 lbf/ft^2 based on a 50-yr recurrence. This is converted to a 100-yr recurrence weight of 4.9 lbf/ft^2 (psf) using a factor of 1.22 (1/0.82) taken from ANSI/ASCE 7-05 Table C7-3. Local snow measurements support this ANSI/ASCE 7-05 value.

To estimate the weight of the 100-yr snowpack at the CPNPP site, the maximum reported snow depths at the Dallas Fort Worth Airport were determined. Table 2.3-202 shows that the greatest snow depth over the 30-yr record is 8 in. The 100-yr recurrence snow depth is 11.2 in using a factor of 1.4 to convert from a 30 yr recurrence interval to 100-yr interval (Reference 2.3-220).

Freshly fallen snow has a snow density (the ratio of the volume of melted water to the original volume of snow) of 0.07 to 0.15, and glacial ice formed from compacted snow has a maximum density of 0.91 (Reference 2.3-221). In the CPNPP site area, snow melts and/or evaporates quickly, usually within 48 hours, and does so before additional snow is added; thus, the water equivalent of the snowpack can be considered equal to the water equivalent of the falling snow as reported hourly during the snowfall. A conservative estimate of the water equivalent of snowpack in the CPNPP site area would be 0.20 in of water per inch of snowpack. Then, the water equivalent of the 100-yr return snowpack would be 11.2 in snowpack x 0.2 in water equivalent/inch snowpack = 2.24 in of water.

Because one cu in of water is approximately 0.0361 pounds in weight, a one in water equivalent snowpack would exert a pressure of 5.20 pounds per sq ft (0.0361 lb/cu in x 144 sq in). For the 100-yr return snowpack, the water equivalent would exert a pressure of 11.7 pounds per sq ft (5.20 lbm/sq ft/in x 2.24 in). This very conservative estimate is approximately twice the value provided in ANSI/ASCE 7-05.

The 100-yr return period snow and ice pack for the area in which the plant is located, in terms of snow load on the ground and water equivalent, is listed below:

- Snow Load = 11.7 lb/ft^2
- Ice Load = 5.06 in * 5.20 lb/ft^2/in = 26.1 lb/ft^2

From Hydrometeorological Report No. 53, NUREG/CR-1486, the 24-hour Probable Maximum Winter Precipitation (PMWP) for a 10 sq-mi area is estimated to be 27 in. The 72-hour PMWP for a 10 sq-mi area is estimated to be 35 in. Assuming a linear relationship between these values gives a 48-hour PMWP of 31 in. Because of the southern location of the site, almost all of this PMWP occurs as liquid. To ensure safety even in the most extreme winter conditions, an assumption was made to combine the 100-year return valves for ice load and snow pack. This yields a maximum extreme winter loading of 37.8 lb/ft^2. As stated in the US-APWR DCD Subsection 3.4.1.2, If PMWP were to occur, US-APWR

safety-related systems and components would not be jeopardized. US-APWR seismic category I building roofs are designed as a drainage system capable of handling the PMWP. The US-APWR DCD also states that seismic category I structures have sloped roofs designed to preclude roof ponding. This is accomplished by channeling rainfall expeditiously off the roof.

2.3.1.2.9 Dust Storms

Blowing dust or sand may occur occasionally in West Texas where strong winds are more frequent and vegetation is sparse. While blowing dust or sand may reduce visibility to less than five mi over an area of thousands of sq mi, dust storms that reduce visibility to one mi or less are quite localized and depend on soil type, soil condition, and vegetation in the immediate area. The NCDC Storm Event database did not report any dust storms in Somervell County between January 1, 1950 and August 31, 2007.

2.3.1.2.10 Ultimate Heat Sink

The performance of the ultimate heat sink is discussed in Subsection 9.2.5. The ambient design air temperatures in Table 2.0-1R are considered in the design of the UHS and are derived based on hourly readings of dry bulb temperature and dew point data from Dallas/Fort Worth Airport (DFW) for the 30-year period from 1977- 2006. Wet bulb temperatures are determined from the NOAA/NCDC data using psychrometric conversion algorithms consistent with the ASHRAE Handbook – Fundamentals (2005). The 1-percent exceedance values for dry bulb temperature and non-coincident wet bulb temperature represent the 99th percentile values (minimum and maximum). The 1-day, 5-day and 30-day worst time periods for the 30-year period were selected from these data. The 0-percent exceedance values (maximum and minimum historical limits) were selected by screening the 30-year hourly temperature records with maximum or minimum dry bulb temperature readings for at least two consecutive hours. Mean coincident wet bulb temperatures represent the average wet bulb values associated with the corresponding dry bulb temperatures at the specified exceedance value. The wet bulb design temperature for the ultimate heat sink was selected to be 80°F in accordance with RG 1.27. The worst 30 day period was selected from the above climatological data between June 1, 1998 and June 30, 1998, with an average wet bulb temperature of 78.0°F. A 2°F margin was added to the maximum average wet bulb temperature for conservatism. The potential for freezing of the ultimate heat sink is remote due to the infrequent occurrence of low temperatures and the short duration of low temperatures.

2.3.1.2.11 Extreme Winds

Estimated extreme winds (fastest mile) for the general area based on the Frechet distribution are:

Return Period (year)	Wind Speed (mi per hr)
2	51
10	61
50	71
100	76

Fastest mile winds are sustained winds, normalized to 30 ft aboveground and include all meteorological phenomena except tornadoes (Reference 2.3-205).

The design basis wind velocity is based on the data from ANSI/ASCE 7-05 (Reference 2.3-220). From Figure 6-1 of ANSI/ASCE 7-05, the 3-second gust wind speed at 33 ft (10 m) aboveground for a 50-year return period for the CPNPP site is 90 mph (40 m/sec). The 3-second gust wind speed for a 100-year return period is 96 mph. The importance factor is 1.15 and the exposure category is C. Wind loadings for the site are discussed in Subsection 3.3.1.

2.3.2 Local Meteorology

CP COL 2.3(1) Replace the content of DCD Subsection 2.3.2 with the following.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

The CPNPP site is located approximately equidistant between Cleburne and Stephenville, Texas, west of the Brazos River. The site elevation is approximately 822 ft mean sea level (msl). The terrain slopes gradually from 300 to 700 ft msl southeast of the site to 1200 to 1800 ft msl northwest of the site (Reference 2.3-205).

2.3.2.1.1 General

In this subsection, the normal and extreme statistics of wind, temperature, water vapor, precipitation, fog, and atmospheric stability are described. Long-term data from proximal weather stations (Figure 2.3-207) have been used to supplement the shorter-term on-site data.

2.3.2.1.2 Surface Winds

Annually, the prevailing surface winds in the region are from the south to southeast while the average wind speed is about 10 mi per hour (mph) based on-site data from 2001-2004 and 2006. As shown on Figures 2.3-208 through 2.3-210, the annual resultant wind vectors for the Dallas Fort Worth Airport, Mineral Wells, and CPNPP are 149°, 138°, and 153°, respectively. The annual

average wind speeds for Dallas Fort Worth Airport, Mineral Wells, and CPNPP are 10.3, 9.0, and 9.8 mi per hour, respectively. In winter there is a secondary wind direction maximum from the north to northwest due to frequent outbreaks of polar air masses (Figures 2.3-274 and 2.3-306).

Percentage frequencies of surface wind direction, by wind speed, at the Dallas Fort Worth Airport for the yr 1997 – 2006 are shown on a monthly and annual basis in Tables 2.3-220 through 2.3-232. According to the annual table, surface wind directions at the Dallas Fort Worth Airport are from the southeast, south-southeast, and south 43 percent of the time. These directions predominate during the individual months also, but to a lesser extent during November through March. The annual average wind speed (shown in Table 2.3-232) is 10.3 mi per hour. The maximum average wind speed (12.7 mph) occurs in the spring, while the minimum (8.2 mph) occurs in the fall.

Percentage frequencies of surface wind direction, by wind speed, at the Mineral Wells Airport for the yr 2001 – 2006 are shown on a monthly and annual basis in Tables 2.3-233 through 2.3-245. According to the annual table, Table 2.3-245, surface wind directions at the Mineral Wells Airport are from the southeast, south-southeast, south, and south-southwest 50 percent of the time. These directions predominate during the individual months also, but to a lesser extent during November through March. The annual average wind speed (shown in Table 2.3-245) is 8.81 mi per hour. The maximum average monthly wind speed (10.73 mph) occurs in the spring, while the minimum (7.32 mph) occurs in the late summer.

Monthly and annual on-site wind frequency distributions for CPNPP using data measured at the 10-meter level are included in Tables 2.3-246 through 2.3-258. Similar to the off-site distribution, the surface wind is from the southeast, south-southeast, south, and south-southwest 51 percent of the time. The annual average wind speed is also similar on-site, averaging 9.8 mi per hour. The maximum average wind speed (11.3 mph) occurs in the spring, while the minimum (8.0 mph) occurs in the late summer.

Monthly and annual on-site wind frequency distributions for CPNPP using data measured at the 60-meter level are included in Tables 2.3-259 through 2.3-271. Similar to the off-site distribution, the surface wind is from the southeast, south-southeast, south, and south-southwest 52 percent of the time. The annual wind speed averages 12.6 mi per hour. The maximum average wind speed (14.8 mph) occurs in the spring, while the minimum (10.3 mph) occurs in the summer. As expected, the average wind speeds at the upper elevation are greater than the lower level wind speeds where surface effects reduce the wind speed.

The maximum 2-minute and 5-second wind speeds at Dallas Fort Worth Airport (1971 – 2000) for each month is presented in Table 2.3-202. As shown, the maximum 5-second wind speed of 78 mph occurred in February 2000.

Wind direction persistence, determined for a ten-yr period (1997 – 2006) at the Dallas Fort Worth Airport, is presented in Tables 2.3-272 through 2.3-274. Table 2.3-272 gives the persistence for a single sector (22.5°). As expected, the direction with the maximum average number of hours with wind from a single sector was south. Table 2.3-273 provides similar data for persistence from three adjacent sectors. For this case, the south-southeast direction had the maximum average number of hours (106 hrs) with wind from three adjacent sectors. Table 2.3-274 gives the persistence for five adjacent sectors. For this case, the south-southeast direction had the maximum average number of hours (167 hrs) with wind from five adjacent sectors. Persistence was assumed to be broken by calm or missing observations. Because of these criteria, persistence as given by the number of consecutive three-hour observations tends to have a bias towards shorter durations.

Wind direction persistence, determined for a six-yr period (2001 – 2006) at the Mineral Wells Airport, is presented in Tables 2.3-275 through 2.3-277. Table 2.3-275 gives the persistence for a single sector (22.5°). The direction with the maximum average number of hours (18.7 hours) with wind from a single sector was south. Table 2.3-276 provides similar data for persistence from three adjacent sectors. For this case, the south-southeast direction had the maximum average number of hours (103 hrs) with wind from three adjacent sectors. Table 2.3-277 gives the persistence for five adjacent sectors. For this case, the south direction had the maximum average number of hours (157 hrs) with wind from five adjacent sectors. As before, persistence was assumed to be broken by calm or missing observations. Because of these criteria, persistence as given by the number of consecutive three-hour observations tends to have a bias towards shorter durations.

Annual wind direction persistence from a single sector, determined from hourly on-site observations at the 10-meter level are presented in Table 2.3-278. These data, which are independent of atmospheric stability, indicate that approximately one-third of the monthly maximum number of consecutive hours of persistence at the CPNPP site are less than 12 hours in duration. During the five-yr period of record, there were only five cases of persistence greater than 24 hours, two of which occurred in the north sector and two in the south sector. The direction with the maximum average number of hours with wind from a single sector was north. Table 2.3-279 provides similar data for persistence from three adjacent sectors. For this case, the south direction had the maximum average number of hours (119 hrs) with wind from three adjacent sectors. Table 2.3-280 gives the persistence for five adjacent sectors. For this case, the south-southeast direction had the maximum average number of hours (200 hrs) with wind from five adjacent sectors.

Annual wind direction persistence from a single sector, determined from hourly on-site observations at the 60-meter level, is presented in Table 2.3-281. These data, which are independent of atmospheric stability, indicate that one-half of the monthly maximum number of consecutive hours of persistence at the CPNPP site are less than 12 hours in duration. During the five-yr period of record, there was

only three cases of persistence greater than 24 hours, two of which occurred in the north and north-northwest sectors. The directions with the maximum average number of hours with wind from a single sector were south-southeast and north-northwest. Table 2.3-282 provides similar data for persistence from three adjacent sectors. For this case, the south direction had the maximum average number of hours (147 hrs) with wind from three adjacent sectors. Table 2.3-283 gives the persistence for five adjacent sectors. For this case, the south direction had the maximum average number of hours (222 hrs) with wind from five adjacent sectors.

A comparison of the average wind persistence for Dallas Fort Worth Airport, Mineral Wells, and CPNPP is provided in Table 2.3-284. These data show that the wind persistence is generally higher at both CPNPP measurement levels than the persistence at Dallas Fort Worth Airport or Mineral Wells for single or multiple sectors. This comparison is illustrated in Figures 2.3-259 through 2.3-261 for a single sector, three adjacent sectors, and five adjacent sectors, respectively. These figures show good general agreement between the three locations, with the exception of the single sector persistence for Dallas Fort Worth Airport, which has a higher persistence in the southern direction.

The monthly and seasonal wind roses for Mineral Wells Airport are provided on Figures 2.3-262 through 2.3-277. On a monthly basis, these figures show that dominant south-southeast wind direction. The seasonal wind rose plots show an additional north-northwest component in the winter and fall. The annual wind rose plot for Mineral Wells is provided on Figure 2.3-209.

Similar monthly and seasonal wind roses for the lower level CPNPP data are provided on Figures 2.3-278 through 2.3-293. On a monthly basis, these figures show the dominant south south-southeast wind direction. The seasonal wind rose plots show a significant additional north and north-northwest component in the winter and fall. The annual wind rose plot for CPNPP is provided on Figure 2.3-210. Monthly and seasonal wind roses for the upper level CPNPP data are provided on Figures 2.3-294 through 2.3-309. On a monthly basis, these figures show the dominant south-southeast wind direction. The seasonal wind rose plots show that the only significant north and north-northwest component is in the winter. The annual wind rose plot for CPNPP is provided on Figure 2.3-310.

2.3.2.1.3 Temperatures

During the winter and early spring, outbreaks of polar continental air are the most common frontal activity. Although these fronts frequently have little weather associated with them, they often stall in Central and South Texas. Low stratus clouds often linger for a day or two before skies become clear (Reference 2.3-229). On occasion, arctic air masses push through the region and cause some of the coldest temperatures. Cold spells, however, rarely last more than a few days. Normally, temperatures drop to 32°F or below about 30 days each yr (Reference 2.3-205). Winter is the driest season, but one or two occurrences of

snow and one or two occurrences of sleet or freezing rain may be expected in both January and February, the coldest months. (Reference 2.3-205)

Pacific maritime cold fronts are more frequent in spring and fall than in winter or summer. These air masses usually bring clear skies to the region, although the weather along the leading edge of the front may be quite violent. Most of the dust storms of early spring and the violent thunderstorms during April, May, and June are associated with these frontal systems. Warm fronts are generally confined to the late fall and early spring months in this region. They are usually confined to the southern half of the region and move northward very slowly (Reference 2.3-205).

Spring is characterized by rapid changes of temperature, i.e., alternating periods of warm and cold conditions. On the average, thunderstorms are more frequent and more violent in the spring than in any other season. Spring is normally the wettest season of the year. The fall is characterized by fair weather, low wind speeds, and moderate temperatures. (Reference 2.3-205)

Typically, summer has over 90 days with temperatures of 90°F or above and temperatures often exceed 100°F (Reference 2.3-230). Table 2.3-219 provides the number of days with temperatures above 90°F, above 100°F, and below 32°F in addition to the number of days with precipitation or snow for Dublin and Weatherford based on data from 1902 through 2004. Tables 2.3-202 through 2.3-205 provide similar data for Dallas Fort Worth Airport, Dallas Love Field, Mineral Wells, and Glen Rose, respectively, over the period of 1971 to 2000. These data show that there are approximately 100 days with maximum daily temperatures above 90°F and approximately three days per yr with maximum daily temperatures below 32°F for these stations. The normal mean temperature for these stations is 64-66°F.

Normal monthly average temperatures for Benbrook Dam, Cleburne, Dallas Fort Worth Airport, Dallas Love Field, Dublin, Glen Rose, Mineral Wells, Stephenville, and Weatherford are shown on Figure 2.3-251 for the period 1971 – 2000. The monthly average temperature for these stations ranges from 45°F in winter to almost 85°F in summer. The normal monthly minimum temperature for the same stations are shown on Figure 2.3-252. The normal monthly minimum average temperature ranges from 30°F in winter to 75°F in summer. The normal monthly maximum temperature for these stations are shown on Figure 2.3-253. The normal monthly average maximum temperature ranges from 55°F in winter to 95°F in summer. The monthly averages indicate that July and August are the hottest months and January the coldest month. A longer term temperature record is provided by the U. S. Historical Climatology Network for Dublin and Weatherford. This database covers the years 1897 – 2005. The monthly minimum, mean, and maximum temperatures for Dublin for the 1897 – 2005 time period are shown on Figure 2.3-254. The annual average minimum, mean, and maximum temperatures for Dublin over the period 1902 – 2005 are shown on Figure 2.3-255. The range of the monthly mean maximum temperature over the period of record (1895 – 2005) for Dublin is shown on Figure 2.3-256 and the

monthly mean and monthly mean minimum temperatures for Dublin are shown on Figures 2.3-257 and 2.3-258, respectively. The annual mean of the monthly mean maximum temperature for Dublin over the period of record (1895 – 2005) is shown on Figure 2.3-311. This figure shows that the annual mean of the monthly mean maximum temperature varied from approximately 73°F to 78°F over the last 111 yr. The annual mean of the monthly mean for Dublin shown on Figure 2.3-312 shows that the annual mean has varied from about 62°F to 66°F over the last 45 yr. The annual mean before 1960 was slightly higher. The variation of the annual mean of the monthly mean minimum temperature at Dublin (Figure 2.3-313) over the same time period (1895 – 2005) is less consistent showing a downward trend in temperature to a range of 51°F to 54°F in the last 45 yr.

The monthly minimum, mean, and maximum temperatures for Weatherford for the 1896 – 2005 time period are shown on Figure 2.3-314. The annual average minimum, mean, and maximum temperatures for Weatherford over the period 1897 – 2005 are shown on Figure 2.3-315. The range of the monthly mean maximum temperature over the period of record (1897 – 2005) for Weatherford is shown on Figure 2.3-316 and the monthly mean and monthly mean minimum temperatures for Weatherford are shown on Figures 2.3-317 and 2.3-318, respectively. The annual mean of the monthly mean maximum temperature for Weatherford over the period of record (1897 – 2005) is shown on Figure 2.3-319. This figure shows that the annual mean of the monthly mean maximum temperature varied from approximately 74°F to 78°F over the last 70 yr. The annual mean of the monthly mean for Weatherford, Figure 2.3-320, shows that the annual mean has varied from about 62°F to 66°F over the last 45 yr. The annual mean before 1960 was slightly higher. The variation of the annual mean of the monthly minimum temperature at Weatherford (Figure 2.3-321) over the same time period (1897 – 2005) is less consistent showing a downward trend in temperature to a range of 49°F to 54°F in the last 45 yr.

The monthly minimum, mean, and maximum temperatures at the site are shown in Table 2.3-285. The annual daily mean at the site is 67°F, which is only slightly higher than the regional data. The monthly mean, minimum, and maximum temperatures at CPNPP over the time period of 2001-2004 and 2006 are shown on Figure 2.3-322. The monthly mean, minimum, and maximum temperatures at Mineral Wells over the time period of 1971 – 2000 are shown on Figure 2.3-323. Comparison of the site data from Figure 2.3-322 with the Mineral Wells data in Figure 2.3-323 shows good general agreement but with relatively higher winter temperatures reported at the CPNPP site. This is due to the shorter period of record at the CPNPP site. The daily mean, minimum, and maximum temperatures at Mineral Wells over the time period of 1971 – 2000 are shown on Figure 2.3-324.

Annual exceedance dry bulb and wet bulb temperature values for Dallas/Fort Worth International Airport (0.4 percent, 1 percent, and 2 percent) are given in Table 2.3-202 along with the 100-yr return dry bulb and wet bulb temperatures.

2.3.2.1.4 Water Vapor

Monthly and annual average relative humidity for four different times of day are given in Table 2.3-286 from 10 yr of record at the Dallas Fort Worth Airport weather station. Based on these data the annual average relative humidity is estimated to be about 65 percent. Monthly and annual average relative humidity for four different times of day are given in Table 2.3-287 from five yr of record at the Mineral Wells Airport. Based on these data the annual average relative humidity at Mineral Wells is estimated to be about 69 percent. The monthly and annual mean dewpoint temperatures and extreme maximum and minimum dewpoint temperatures are shown in Table 2.3-288, based on 1949 – 2006 data from the Mineral Wells Airport. The average daily dewpoint temperature from Mineral Wells Airport for the same time period is shown on Figure 2.3-325.

Based on 10 yr of data (1997 – 2006) from the Dallas Fort Worth Airport (Table 2.3-289), the worst one-day (May 26, 1997) average wet bulb temperature was 78.6°F and the corresponding average dry bulb temperature was 83.6°F. The worst five consecutive day period (June 29, 1997 – July 3,1997) is given in Table 2.3-290. The average wet bulb temperature for these five days was 77.4°F and the corresponding dry bulb temperature of 84.6°F. The worst 30 day consecutive period for Forth Worth is given in Table 2.3-291. The average wet bulb temperature for this period (July 4, 2001 through August 2, 2001) was 76.1°F and the dry bulb temperature was 87.4°F. Based on six yr of data (2001 – 2006) from the Mineral Wells Airport (Table 2.3-292), the worst one-day (June 24, 2003) average wet bulb temperature was 77.0°F and the corresponding average dry bulb temperature was 84.4°F. The worst five consecutive day period (June 21, 2003 – June 25, 2003) is given in Table 2.3-293. The average wet bulb temperature for these five days was 75.8°F, with a dry bulb temperature of 83.3°F. The worst 30 consecutive period for Mineral Wells is given in Table 2.3-294. The average wet bulb temperature for this period (July 14, 2001 through August 12, 2001) was 73.8°F, with a dry bulb temperature of 88.3°F.

2.3.2.1.5 Precipitation

Monthly and annual precipitation normals and the mean number of days with precipitation greater than 0.01 in for CPNPP are presented in Table 2.3-295. These data indicate that the highest monthly average rainfall occurs in March, with an annual average total rainfall of 30.3 in. The number of days with measurable precipitation (74) is also presented in Table 2.3-295 based on-site data from 2001, 2003, and 2006. The maximum 24-hour rainfall and 48-hour rainfall totals are also given in this table as 3.8 in and 4.5 in, respectively. The annual rainfall frequency distribution as a function of rainfall intensity is given in Tables 2.3-296 through 2.3-298 for Fort Worth, Mineral Wells, and CPNPP, respectively. These figures show that the winter months have the highest total hours of rainfall; however, most of this rainfall is light. The monthly and annual distribution of rainfall by direction for a 10 yr period of record at the Dallas Fort Worth Airport, six yr at the Mineral Wells Airport, and three yr at CPNPP are given in Tables 2.3-299 through 2.3-301, respectively. These tables show that rainfall

with wind from the north is the most common due to arctic air intrusions followed by rainfall with winds from the most common southerly direction. The long term (1987 – 2006) average annual rainfall at Mineral Wells is given in Figure 2.3-326. This figure shows an increasing trend in rainfall, which is biased by the drought in 1993 – 1995. The average annual rainfall for this station over the longer period of 1949 – 2006 for which there is data is 34.1 in. Figures 2.3-327 through 2.3-330 give the average monthly precipitation for Mineral Wells, Weatherford, Dublin, and CPNPP, respectively. The Mineral Wells data (1971 – 2000) show a peak in the spring, with a secondary, smaller, peak in the fall and a minimum in January. This distribution agrees with the data from the longer term records for Weatherford (1896 – 2005) and Dublin (1896 – 2005) shown in Figures 2.3-328 and 2.3-329. The data from CPNPP also show a spring peak and a smaller fall peak but the other details of the precipitation curve do not match the longer term records from other weather stations. This is due to the very short data record (three yr) used for CPNPP. The long term annual precipitation data for Dublin (1896 – 2005) and Weatherford (1889 – 2005) are given in Figures 2.3-331 and 2.3-332. The data for Dublin show a gradually increasing trend, which may be due to localized relative drought conditions in the early 1900s. The data for Weatherford in Figure 2.3-328 are considered to be more representative of the general regional conditions, with an annual average of about 30 in.

Monthly, seasonal, and annual precipitation wind roses for Mineral Wells are presented in Figures 2.3-333 through Figure 2.3-349. These data are based on six yr of data at Mineral Wells Airport. These data show that the highest incidence of precipitation occurred with winds from the north. The monthly, seasonal, and annual precipitation wind roses for CPNPP for the years 2001, 2003, and 2006 presented in Figures 2.3-350 through Figure 2.3-366 show the same pattern as the Mineral Wells data. The annual precipitation wind rose for Dallas Fort Worth Airport presented on Figure 2.3-367 also show the maximum frequency of precipitation occurred with north winds.

Snow and sleet occur from December through March, with an occasional snow flurry in late November or early April. Monthly and annual average totals of snow from 30 yr of record at the Dallas Fort Worth Airport, Dallas Love Field, Mineral Wells, and Glen Rose are provided in Tables 2.3-202 through 2.3-205, respectively. These data give an annual expectancy of 2.5 in of snow. Extremes of snowfall at these selected stations were also previously presented in Tables 2.3-202 through 2.3-205.

2.3.2.1.6 Fog

Heavy fog is that which reduces visibility to one-quarter mi or less. Average monthly and annual number of heavy fog days based on 10 yr of data at the Dallas Fort Worth Airport are presented in Table 2.3-302. These data indicate that most (63 percent) of the heavy fog days occur in winter, with a few occurrences during the remainder of the year. The annual average hours of fog was 16.2 hours. Average monthly and annual number of heavy fog days based on six yr of data at the Mineral Wells Airport presented in Table 2.3-303 also show that

winter produces the highest number of hours of fog although the annual hours of fog at Mineral Wells is higher (46.7 hours).

2.3.2.1.7 Atmospheric Stability

Based on data for the period 2001 – 2004, and 2006 at CPNPP, the monthly and annual frequency distributions of stability classes are shown in Table 2.3-304. The stability classes are based on the standard Pasquill classification using the 10m to 60 m temperature differential. These data indicate that the frequency of stable classes reaches a peak during the fall and winter. However, the stable classes (F and G) only account for less than ten percent of the total hours. Neutral (Class D) and slightly stable (Class E) account for almost 70 percent of the annual hours.

The CPNPP joint frequency distribution for each stability category is provided in Table 2.3-305. The upper bounds for each wind speed category are £0.5 m/s, £0.75 m/s, £1.0 m/s, £1.25 m/s, £1.5 m/s, £2.0 m/s, £3.0 m/s, £4.0 m/s, £5.0 m/s, £6.0 m/s, £8.0 m/s, and £16.0 m/s. For the year of data under consideration, there were no hourly recordings of wind speeds greater than 16.0 m/s. In this table, calms were classified as hourly average wind speeds below the vane or anemometer starting speed, whichever is higher. According to the meteorological tower instrumentation data given in Table 2.3-34 of the CPNPP Units 1 and 2 UFSAR (Reference 2.3-205), the starting wind speeds for the anemometer and vane are 0.45 m/s. Therefore, a starting wind speed of 0.45 m/s (1.0 mph) is used.

The CPNPP joint frequency distributions were not compared to the long-term joint frequency distributions from the National Weather Service stations because the joint frequency distributions using the National Weather Service data would be based on a different criteria for determining Pasquill stability classes.

2.3.2.1.8 Mixing Heights

The frequencies of seasonal and annual mixing heights are included and discussed in Subsection 2.3.1.2.7. Because on-site measurements of mixing depth are neither required nor made, monthly mixing depths from upper air data at Stephenville, Texas and data from the Ventilation Climate Information System are used.

Temperature inversions are also important in evaluating the potential for dispersion of pollutants. A temperature inversion generally refers to an increase in temperature with height or to the layer within which such an increase occurs. An inversion can lead to pollution such as smog being trapped close to the ground, with possible adverse effects on health. An inversion can also suppress convection by acting as a "cap". An inversion is defined as any three readings on a sounding that show temperatures increasing with elevation (below 3000 meters). The inversion layer height is the point (found by interpolation between readings) at which temperature again starts to decrease with elevation. The maximum inversion strength is the maximum temperature rise divided by

elevation difference within the inversion layer. The frequency and strength of inversion layers are evaluated using six yr of weather balloon data collected at the Fort Worth radiosonde station (Reference 2.3-222). Weather balloons are released twice daily at 0:00 GMT (6:00 a.m. CST) and 12:00 GMT (6:00 p.m. CST) to obtain vertical profiles of temperature, wind, and dewpoint temperature. The monthly data are provided in Table 2.3-306 through Table 2.3-317 in terms of number of mornings and afternoons containing inversions, average inversion layer elevation, and the average strength of the inversions. Table 2.3-318 provides annual average data for the period.

2.3.2.1.9 Representativeness of the On-site Data

The comparison of the temperature, precipitation, wind speed, and wind direction provided in the previous sections demonstrates that the CPNPP on-site data are representative of longer-term climatological conditions. The differences that do occur indicate that diffusion estimates will probably be higher than normal (conservative).

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

2.3.2.2.1 General

Potential modifications of the local meteorology at the site resulting from the construction and operation of Units 3 and 4 are believed to be small. The Units 1 and 2 Containment Buildings and associated facilities in addition to the Units 3 and 4 reactor complex are expected to have some small influence on the local air flow; specifically, mechanical turbulence is expected downwind of the plant due to building wake effects.

2.3.2.2.2 Impact of Squaw Creek Reservoir

The impact of filling SCR on the local meteorology has already taken place and no changes are anticipated during the Unit 3 and 4 construction or operations. The impact of filling SCR was addressed in Section 2.3 of the CPNPP Units 1 and 2 FSAR and will not be discussed further.

2.3.2.2.3 Topographical Description

A map of the CPNPP area for a distance of five mi from the site is shown in Figure 2.3-368. The topographic cross-sections for each compass direction out to five mi from the site are given in Figure 2.3-369. These figures indicate the maximum elevation versus distance from the plant in each sector. The site elevation is approximately 822 ft msl. The terrain varies from 600 to 1000 ft msl within five mi of the site, and is generally in this range out to 50 mi. General topographic features for a radius of 50 mi are shown in Figure 2.3-370. The topographic cross-sections out to 50 mi in each compass direction are given in Figure 2.3-371.

As seen from these figures, the elevation increases to about 600 – 700 ft above the plant elevation in the west, north-west, and west-southwest directions.

Variable terrain has a potential to influence local diffusion characteristics. Terrain variations on the order of plus or minus 200 ft are not pronounced enough to cause any significant flow blocking. Two possible influences, though, cold air drainage and channeling, have been investigated. The occurrences of cold (denser) air drainage down Squaw Creek was assessed in the CPNPP Unit 1 and 2 UFSAR by a comparison of wind direction frequencies between the 10-meter (850 ft msl) and 60-meter (1000 ft msl) levels for a 131-day period. If drainage were to occur, then marked increases of down-valley wind frequencies (ESE and SE) from the upper to the lower level would be expected. Marked changes in frequency did not appear in the data; therefore, it was concluded that cold air drainage along Squaw Creek is not significant. Because Squaw Creek is now completed, this effectively modifies the topography over a large area surrounding CPNPP to a minimum elevation of 770 ft msl, or only about 50 ft less than site elevation. Thus, cold air drainage is unlikely.

Channeling of air flow, the other potential topographical effect, was evaluated in the CPNPP Units 1 and 2 FSAR by comparing the 10-meter wind directions with wind direction data from Dallas Love Field, where surroundings are relatively flat. A significant increase in wind direction frequencies for both up and down valley sectors (WNW, NW, NNW, ESE, and SE) would occur if channeling is an important influence. Approximately eight months of concurrent wind direction data were evaluated indicating that channeling of the air along Squaw Creek is not a prominent effect.

The channeling and air-drainage study results presented in the Units 1 and 2 FSAR are indicative of a relatively flat terrain with little, if any, topographic effect on the local airflow.

2.3.2.2.4 Cooling Tower Plume

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

The operation of two Linear Mechanical Draft Cooling Towers (LMDCT) for each unit at the site results in the emission of small water droplets entrained in the tower air flow (i.e., drift). The droplets contain the dissolved solids found in the circulating water (e.g., salts) that may eventually deposit on the ground as well as on structures and vegetation. The drift droplet emissions are controlled by the use of drift eliminators that rely on inertial separation caused by exhaust flow direction changes. In addition to drift emissions, there is another potential impact of the cooling towers to the environment: the warm saturated air leaving the towers is cooled by the ambient air such that the water vapor condenses into a visible

plume that may persist for some distance downwind depending on meteorological conditions (e.g., wind speed, relative humidity). These visible plume occurrences may pose some aesthetic and ground shadowing impacts. Under relatively high wind speeds and humid conditions, the aerodynamic wake turbulence may result in the visible plume touching down causing ground level fogging and, under freezing conditions, icing.

The meteorological data used in the plume analysis is a hybrid of various data sources, but the impact of merging these sources is assumed to be insignificant compared to the inherent uncertainties of predicting future meteorological conditions. The wind speeds and direction are taken from the site meteorology tower for the years 2001-2006: the temperature, humidity, and cloud cover data are from the national weather station at Mineral Wells located 37 mi to the northwest, and the mixing height data is from the airport at Stephenville, 20 mi to the southwest. The topography within 37 mi indicates no major terrain changes that would cause any of these locations to have a different microclimate from the other two. The general site is approximately 822 ft elevation, while Mineral Wells is at 930 ft and Stephenville is 1321 ft with no intervening hills or valleys.

An analysis of the potential environmental impacts caused by the operation of LMDCTs was conducted using the Electric Power Research Institute (EPRI) sponsored Seasonal/Annual Cooling Tower Impact (SACTI) Program. This model is considered a state-of-the-art cooling tower impact model by EPRI and the nuclear industry. It was developed by Argonne National Laboratory (ANL) using the knowledge obtained from extensive research conducted on cooling tower environmental effects. The SACTI model provides salt drift deposition pattern (i.e., kg/km2/month) as a function of distance and direction from the cooling towers as well as the frequency of occurrence of visible plumes, hours of plume shadowing, and ground level fogging and icing occurrences by season resulting from the operation of the cooling towers. The circulating water total dissolved solids of 8402 mg/l (based on an average input TDS of 3525 mg/l and cooling tower operation at 2.4 cycles of concentration) is the expected long term average condition for Lake Granbury.

The SACTI results, as presented in Table 2.3-319, indicate that the longest and largest visible plumes occur in the winter, with smaller plumes occurring in the spring and fall seasons, due to the cold air in winter causing condensation of the moist plumes more readily than in the warmer seasons (i.e., cold air has a much smaller capacity of holding water vapor). The summer visible plumes are noticeably smaller because warmer ambient air results in less condensation of the moist plumes due to its ability to accommodate higher water vapor concentrations.

The largest visible plumes shown in Table 2.3-319 reach a distance of 6210 meters (3.86 mi) downwind of the towers. The frequency of seasonal plume length by compass direction are given in Tables 2.3-320 through 2.3-323. It should be noted that the longest plumes occur during conditions of high ambient relative humidity that are conducive to natural fog formation and poor visibility. Under these conditions, the atmosphere is already at, or close to, saturation. Therefore,

the largest plumes may not be discernible from the ambient fogging conditions. Figure 2.3-372 provides the seasonal variation of plume length as a function of compass direction.

Table 2.3-324 provides the hours of plume shadowing by downwind distances and direction. Consistent with the visible plume frequency results, most shadowing occurs in the winter season with lesser amounts in the spring and fall and the least amounts in the summer. The annual hours of plume shadowing are given in Figure 2.3-379. The SACTI output also shows that ground level fogging occurs mainly to the north and south directions (Table 2.3-325). Figure 2.3-377 provides the hours of fogging as a function of distance and direction. The pattern of ground level icing is similar to the pattern of fogging, as shown in Table 2.3-326 and Figure 2.3-378. Most ground icing occurs within a half mi of the site except in the south and north directions.

The salt deposition pattern shown in Table 2.3-327 indicates that there is negligible salt deposition at a distance of 1.5 miles from the site with the highest amount being 2.91 kg/km^2/month. The salt deposition rate is shown in Figure 2.3-373. The maximum salt deposition amount of 137.3 kg/km^2/month at 100 meters from the site can be compared with a value of 400 kg/km^2/month below which damage to vegetation is not expected to occur according to a study of the environmental effects of cooling towers. Salt deposition as a function of distance and direction is shown on Figure 2.3-373. SCR is adjacent to the cooling towers and is likely to receive cooling tower drift that would add to TDS of the reservoir. However, TDS measured in SCR in 2007 exceeded 2600 mg/L at all sampling locations across all seasons, which is likely due to the reservoir acting as the UHS for two once through units. Increases in SCR TDS measurements due to cooling tower drift are anticipated to be negligible. In addition, according to NUREG-1555, general guidelines for predicting effects of drift deposition on plants suggest that many species have thresholds for visible leaf damage in the range of 10 to 20 kg/ha/month of NaCl deposited on leaves during the growing season. This range of deposition corresponds to 1000 to 2000 kg/km^2/month. Therefore, no impacts on vegetation outside the site boundary are expected.

The deposition patterns for chlorides and total dissolved solids are shown in Table 2.3-328 and Table 2.3-329. These results are illustrated in Figures 2.3-374 and 2.3-375, which show that the deposition is minimal at the site boundary.

The maximum predicted water deposition rate is 4.9×10^4 kg/km^2/month at a downwind distance of 100 meters from the cooling towers (Table 2.3-330). The water deposition rate is shown in Figure 2.3-376. This deposition rate is the rainfall equivalent of 0.002 inches per month based on the density of water (i.e., 1000 kg/m^3), which is a trivial amount compared to the normal monthly precipitation of 2 to 3 in. The National Weather Service (NWS) considers precipitation of less than 0.01 in as a trace amount. A summary of the seasonal visible plume lengths is given in Table 2.3-331.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Local meteorological data have not been used for design and operating basis considerations other than those conditions referred to in Subsections 2.3.4 and 2.3.5. Design wind loadings, tornado loadings, and snow loadings are referred to under Regional Meteorology, Subsection 2.3.1. Comparison of DCD site parameters with the CPNPP site characteristics is given in Table 2.0-1R.

2.3.3 On-site Meteorological Measurements Program

CP COL 2.3(1) Replace the content of DCD Subsection 2.3.3 with the following.

The meteorological monitoring program for CPNPP Units 3 and 4 is a continuation of the on-site meteorological monitoring program in place at CPNPP Units 1 and 2. The on-site program follows the program requirements defined in the CPNPP Off-site Dose Calculation Manual (ODCM) (Reference 2.3-223). The current meteorological monitoring program is in effect throughout the CPNPP Units 3 and 4 construction, pre-operational, and operational phases of the project.

The CPNPP meteorological monitoring program completed the pre-operational phase (May 15, 1972 – May 14, 1976) and was reestablished as an operational system prior to CPNPP Unit 1 fuel load (Reference 2.3-205). The program is maintained in accordance with all applicable requirements, was improved on several occasions, and maintains a high level of reliably to perform all required functions.

The pre-operational meteorological program measured the parameters needed to evaluate the dispersive characteristics of the site for both routine operational and hypothetical accidental releases of radionuclides to the atmosphere.

2.3.3.1 Meteorological Measurement System

The CPNPP Units 1 and 2 Reactor Complex is located approximately 450m west-northwest of the meteorological tower. The top of the dome is 69 meters above the level of the base of the meteorological tower. Prior to construction of CPNPP Units 1 and 2, wind was recorded from the west-northwest sector approximately 2.1 percent of all recordings; thus, any effect of the CPNPP Units 1 and 2 Reactor Complex on the overall meteorological measurements program is minimal. Current data (2001 – 2006) show that the wind is from the northwest approximately 2.4 percent of the time at the upper instrument level (60m) and approximately 1.4 percent of the time at the lower (10m) instrument level. In addition, no other structures are in such proximity to the tower that will cause a significant alteration of the meteorological data.

The meteorological measurements system consists of a primary meteorological tower, a backup tower, and a computer system with condition and limit code

checks. The location of the meteorological towers relative to other significant site structures is shown in Figure 2.3-380.

The primary tower is located east of the CPNPP Units 1 and 2 reactor complex at an elevation of 838 ft - 9 in above sea level. The primary tower structure is a 60-m, guyed, open lattice tower with an instrument elevator and instrumentation booms at the 10-m and 60-m levels. Due to the prevailing winds, the booms are located on the west side of the tower in order to minimize tower interference. The instrument booms are approximately 8 feet in length and the tower base is approximately 44 inches on a side. This boom length exceeds the length recommended in Regulatory Guide 1.23, Revision 1 to minimize airflow modification and turbulence induced by the supporting structure itself. The aspirator motors and shields for the temperature sensors are oriented north/south. The primary meteorological tower directly monitors or provides information to determine the following meteorological parameters:

- Wind speed at 10 m and 60 m.
- Wind direction at 10 m and 60 m.
- Ambient temperature at 10 m.
- Delta-temperature between 10 m and 60 m (redundant channels).
- Sigma theta at 10 m.
- Precipitation near ground level.

An additional 10-m backup tower is located 75 ft east-northeast from the primary tower. This tower is an open lattice tower with a stationary instrumentation boom located on top of the tower. The aspirator motor and shield for the backup temperature sensor are also oriented north/south. The backup tower monitors or provides information to determine the following meteorological parameters:

- Wind speed at 10 m.
- Wind direction at 10 m.
- Ambient temperature at 10 m.
- Sigma theta at 10 m.

All the towers and instrumentation described above are located in an area surrounded by a fence and maintained free of obstructions that could interfere with data collection and accuracy. The environmentally controlled Meteorological Instrumentation Building that supports the electronic components associated with the instrumentation on the towers is located within the fenced area. (Reference 2.3-205)

Pre-operational atmospheric moisture monitoring was conducted from June 12, 2008 through September 23, 2008. The instrumentation used to collect this data was a Climatronics capacitive relative humidity sensor. This instrument had the following characteristics:

- Accuracy: <+/- 1% RH from 0 - 100%
- Repeatability: +/- 0.3% RH
- Operating Range: 0 - 100%

This instrument was located on top of the Project Records Center Building approximately 30 feet above grade (grade elevation ~830 feet). The pre-operational onsite data was used to demonstrate that the actual onsite conditions correlated well with the longer term data from local weather stations which were used for the official calculations.

The CPNPP site humidity data was compared with data from the closest first order National Weather Service stations located at the Mineral Wells Airport (MWL) and the Dallas Fort Worth (DFW) Airport. Data from MWL and DFW was obtained from the National Weather Service spanning June 12, 2008, through September 23, 2008. The CPNPP site humidity data covered an identical time span.

A comparison of the monthly humidity averages is provided in Table 2.3-351 and Figures 2.3-383 through 2.3-386. As shown in Table 2.3-351, average humidity measurements at CPNPP fall directly between humidity measurements taken at DFW and MWL. The measurements taken at DFW underestimate the CPNPP humidity and measurements taken at MWL overestimate the CPNPP humidity. Likewise, measurements taken at DFW are often substantially lower than both CPNPP and MWL during peak humidity occurrences. For example, on September 9, 2008 the daily humidity average at CPNPP and MWL was 91 percent and 90 percent, respectively, while the daily humidity average at DFW was 78 percent.

The comparison of four months of data from the CPNPP site with offsite data sources indicates that the CPNPP site relative humidity data correlates very well with data from MWL. As a result of this close correlation, recording additional humidity data at the CPNPP site was not necessary. Due to relative humidity measurements at DFW being consistently below CPNPP, both on average and during peak events, MWL is selected as a better representation of CPNPP site humidity conditions. This conclusion is reasonable due to the rural setting at CPNPP and MWL compared to the urban DFW location. In addition, the proximity of MWL to CPNPP (37 miles) compared to the distance to DFW airport (61 miles) makes the MWL data more representative. The relative humidity recorded at the MWL National Weather Service station is representative of the relative humidity at the CPNPP site for the reasons discussed above and serves as the data of record for support calculations, such as cooling tower plume analysis.

2.3.3.2 Instrumentation

An overview of the instrumentation used in the meteorological monitoring system is provided below. The CPNPP Units 1 and 2 FSAR and other plant documents contain specific data about sensors and requirements for replacement of sensors. Wind speeds at the 10-m and 60-m levels are measured with a 3-cup anemometer with a threshold of 0.45 m/s and a range of 0-100 mph. Wind directions at the 10-m and 60-m levels are detected by a wind vane with a threshold of 0.45 m/s and a range of 0 to 360 degrees. Temperatures at the 10-m and 60-m levels are measured with a platinum temperature sensor with a range of -20°F to +120°F. Delta temperature between the 10-m and 60-m levels uses the temperature sensors at each level and has a range of -5°F to +15°F. Precipitation is measured at the surface with a tipping bucket gauge with a threshold of 0.01-in and a range of 0-in to 1.0-in.

2.3.3.3 System Accuracy

System accuracies are specified in Tables 2.3-332 and 2.3-333. All system accuracies meet or exceed regulatory requirements (Reference 2.3-205). Calibration and maintenance procedures ensure the accuracy of the instrumentation. All calibrations are performed semi-annually and in accordance with the ODCM. Calibration of metrological tower instrumentation is performed in accordance with the Quality Related CPNPP common unit Instrument and Control Manual. Calibration is applied to the individual instruments and the entire channel (through the plant computer points in the control rooms). The surveillance requirements provided in the ODCM require that the wind speed, wind direction, and temperature instrumentation channels at both measurement levels be operable at all times. In addition, channel checks are performed at least once per 24 hours in accordance with the ODCM. An annual inspection of the tower structure is also performed. The guyed wires and anchors are inspected every five years in accordance with the CPNPP Units 1 and 2 inspection program. The CPNPP Units 1 and 2 meteorological program complies with the requirements of the Second Proposed Revision 1 to Regulatory Guide 1.23 (April, 1986), as discussed in Units 1 and 2 FSAR Section 2.3.3.2.

2.3.3.4 Data Recovery

Data recovery from the meteorological monitoring program for the six-yr period 2001 – 2006 is presented in Table 2.3-334. Recovery rates are provided for joint frequency distribution (wind speed, wind direction, and stability class determined by delta temperature) and for each individual channel. The average joint frequency distribution recovery rate for this five-yr period is 98.9 percent.

2.3.3.5 Meteorological Data Processing

The meteorological monitoring program provides data for many functions. Meteorological data collection is the primary focus of the program, but the data are also provided to the plant computer system for easy access by operations and

emergency planning personnel. These data are available for routine operations, accident analysis, and annual reporting requirements.

2.3.3.5.1 Data Acquisition

The meteorological monitoring system includes two separate recording systems. There is a digital system and a digital paperless recorder. The digital system records all data on the Meteorological System Computer (METSYS Computer) in the Unit 1 main control room. Four (4) separate data recording systems exist:

- Meteorological System Computer (METSYS).
- Yokogawa Digital Recorder.
- U1 Plant Computer (U1-PC).
- U2 Plant Computer (U2-PC).

The digital paperless recorder is mounted inside the CPNPP Units 1 and 2 combined main control room and the Units 3 and 4 main control rooms.

The meteorological data sensors electronic signals from both towers are transmitted via Modems to demultiplexers located in the Unit 1 plant computer room.

The signals from the Meteorological Instrumentation Building that supply the METSYS computer also supply the CPNPP Units 1 and 2 Plant Computers and the digital recorder with meteorological data. The Plant Computer system is completely diverse from the METSYS computer, uses separate software, and it displays and stores data to support the Operations and Emergency Planning Departments. The Plant Computer is used to help meet the requirements of NUREG-0696, with displays in the CPNPP Units 1 and 2 Technical Support Center (TSC), the CPNPP Unit 3 TSC, the CPNPP Unit 4 TSC, and the Emergency Operations Facility (EOF). (Reference 2.3-205)

2.3.3.5.2 Data Processing

The meteorological computer system provides a digital readout of all channels received from the instrumentation located on the primary and back-up towers. The data are sampled every five seconds for all parameters. Using the data signals received, 15-min and 1-hr averages are calculated by the software for every instrument channel except precipitation, which is the difference between the beginning value for the averaging period and the ending value for the averaging period. The 15-min and 1-hr averages are stored by the computer system and are available for review, editing, or replacement as necessary to provide good quality data with a high recovery rate. The software performs data quality and limit checks as data are recorded, and it displays the results of these checks with the data when the data are reviewed. The data averaging methodology meets the

requirements of Regulatory Guide 1.23 Second Proposed Revision 1, April 1986 (Reference 2.3-205). Fifteen minute and one hour data averages are provided for the following meteorological channels:

Primary tower 10 m. - wind speed	Back-up tower 10 m. - wind speed
Primary tower 10 m. - wind direction	Back-up tower 10 m. - wind direction
Primary tower 60 m. - wind speed	
Primary tower 60 m. - wind direction	
Primary tower 10 m. - sigma theta	Back-up tower 10 m. - sigma theta
Primary tower 10 m. - ambient temperature	Back-up tower 10 m. - ambient temperature
Primary tower upper - 10 m. - “A” delta temperature	
Primary tower upper - 10 m. - “B” delta temperature	
Primary tower base - precipitation	

The meteorological data from the METSYS computer is available from the site wide Plant Computer display terminals.

The METSYS Computer compiles and saves 15-minute and hourly averaged data with quality codes. It is capable of automatically saving up to 10 yr data on two mirrored hard disks. This computer is also capable of developing joint frequency reports and data tape(s) in NRC prescribed format. Additional historical data are saved on external media in the plant archives.

CPNPP procedures (Reference 2.3-231) require a detailed review of all data parameters on a quarterly basis. Hourly averages are reviewed, validated, replaced with backup data if necessary, documented, then archived.

2.3.4 Short-Term Atmospheric Dispersion Estimates for Accident Releases

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CP COL 2.3(2) Replace the content of DCD Subsection 2.3.4 with the following.

2.3.4.1 Objective

The on-site meteorological data record at CPNPP site for the period 2001 through 2006 has been used to calculate dilution factors which can be anticipated in the event of an accidental release of radionuclides into the atmosphere. The two-hour dilution factors are calculated at the exclusion area boundary (EAB); for longer

time periods the factors are calculated at the outer boundary of the low population zone (LPZ).

The consequence of a design basis accident in terms of personnel exposure is a function of the atmospheric dispersion conditions at the site of the potential release. Atmospheric dispersion consists of two components: 1) atmospheric transport due to organized or mean airflow within the atmosphere and 2) atmospheric diffusion due to disorganized or random air motions. Atmospheric diffusion conditions are represented by atmospheric dispersion factor (χ/Q) values. This subsection describes the development of the short-term diffusion estimates for the site boundary and the control room. A description of the atmospheric dispersion modeling used in evaluating potential the consequences of hazardous material releases is given in Subsection 2.3.4.5.

2.3.4.2 Calculations

The efficiency of diffusion is primarily dependent on winds (speed and direction) and atmospheric stability characteristics. Dispersion is rapid within Stability Classes A through D and much slower for Classes E through G. That is, atmospheric dispersion capabilities decrease with progression from Class A to Class G, with an abrupt reduction from Class D to Class E (Regulatory Guide 1.145 and NUREG/CR-2858).

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

Using joint frequency distributions of wind direction and wind speed by atmospheric stability, PAVAN provides the χ/Q values as functions of direction for various time periods at the exclusion area boundary (EAB) and the low population zone (LPZ). The meteorological data needed for this calculation included wind speed, wind direction, and atmospheric stability (NUREG/CR-2858). The meteorological data used for this analysis were collected from the on-site monitoring equipment from 2001 through 2006. Because the data recovery for 2005 was below 90 percent these data were not used. These data were averaged and the joint frequency distributions are reported in Table 2.3-305. Other plant specific data included tower height at which wind speed was measured (10.0 m) and distances to the EAB (0.5 mi) and LPZ (2 mi). The distances to the EAB, LPZ, and from the release boundary to the EAB are given in Table 2.3-335.

Within the ground release category, two sets of meteorological conditions are treated differently. During neutral (D) or stable (E, F, or G) atmospheric stability

conditions when the wind speed at the 10-meter level is less than six meters per second (m/s), horizontal plume meander is considered. χ/Q values are determined through the selective use of the following set of equations for ground-level relative concentrations at the plume centerline:

$$\chi/Q = \frac{1}{\bar{U}_{10}(\Pi\sigma_y\sigma_z + A/2)} \qquad \text{Equation 1}$$

$$\chi/Q = \frac{1}{\bar{U}_{10}(3\Pi\sigma_y\sigma_z)} \qquad \text{Equation 2}$$

$$\chi/Q = \frac{1}{\bar{U}_{10}\Pi\Sigma_y\sigma_z} \qquad \text{Equation 3}$$

where:

χ/Q is relative concentration, in sec/m^3

U_{10} is wind speed at 10 meters above plant grade, in m/sec

s_y is lateral plume spread, in meters, a function of atmospheric stability and distance

s_z is vertical plume spread, in meters, a function of atmospheric stability and distance

S_y is lateral plume spread with meander and building wake effects, in meters, a function of atmospheric stability, wind speed, and distance

A is the smallest vertical-plane cross-sectional area of the reactor building, in meters^2

PAVAN calculates χ/Q values using Equations 1, 2, and 3. The values from equations 1 and 2 are compared and the higher value is selected. This value is then compared with the value from Equation 3, and the lower value of these two is selected as the appropriate χ/Q value.

During all other meteorological conditions, unstable (A, B, or C) atmospheric stability and/or 10-meter level wind speeds of 6 m/s or more, plume meander is not considered. The higher value calculated from equation 1 or 2 is used as the appropriate χ/Q value.

From here, PAVAN constructs a cumulative probability distribution of χ/Q values for each of the 16 directional sectors. This distribution is the probability of the given χ/Q values being exceeded in that sector during the total time. The sector

χ/Q values and the maximum sector χ/Q value are determined by effectively "plotting" the χ/Q versus probability of being exceeded and selecting the χ/Q value that is exceeded 0.5 percent of the total time. This same method is used to determine the five percent overall site χ/Q value.

The χ/Q value for the EAB or LPZ boundary evaluations will be the maximum sector χ/Q or the five percent overall site χ/Q, whichever is greater (Regulatory Guide 1.145).

Regulatory Guide 1.145 divides release configurations into two modes, ground release and stack release. A ground release includes all release points that are effectively lower than two and one-half times the height of the adjacent solid structures. This is conservative, because the building wake effect will tend to reduce the calculated χ/Q. All release points will be considered as ground releases.

PAVAN requires the meteorological data in the form of joint frequency distributions of wind direction and wind speed by atmospheric stability class. The meteorological data used were obtained from the CPNPP meteorological data collected from 2001 through 2006. Data from 2005 was not used due to low data recovery.

The stability classes were based on the classification system given in Table 1 of U.S. Nuclear Regulatory Commission Regulatory Guide 1.23, as follows:

Classification of Atmospheric Stability
(Reference, Regulatory Guide 1.23, Table 1)

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

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

Also, the data in the joint frequency distribution tables were rounded for input into the PAVAN code.

Building area is defined as the smallest vertical-plane cross-sectional area of the reactor building, in sq meters. Building height is the height above plant grade of the containment structure used in the building-wake term for the annual-average calculations. For conservatism, the containment area is used in the determination of building-wake effects. A conservative building cross-sectional area of 2500 m^2 and a building height of 69.9 meters were used for building wake calculations based on parameters from Figure 2.2-11 and Table 2.2-2 of the US-APWR DCD Tier 1 material.

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

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

The cumulative frequency of χ/Q at the EAB can be found in Table 2.3-337. Table 2.3-337 also presents the cumulative frequency at the LPZ. A summary of results is provided below. Median (50 percent) values, provided in Table 2.3-337, may be used in making realistic estimates of the environmental effects of potential radiological accidents; conservative estimates are based on calculated 5 percent values. A comparison of the site specific χ/Q values with the DCD χ/Q values is provided in Table 2.3-337. The site-specific χ/Q values were arbitrarily increased by 10% to provide margin.

2.3.4.3 Relative Concentration Estimates at the Main Control Room and Technical Support Center Emergency Intake

The atmospheric dispersion estimates for the CPNPP main control room were calculated based on the guidance provided in Regulatory Guide 1.194. The main control room and Technical Support Center (TSC) χ/Qs were calculated for all probable release points to the main control room air intakes using the ARCON96 computer code (NUREG/CR-6331) based on the hourly meteorological data collected for the years 2001 through 2004 and 2006. The locations of the assumed release points and location of the main control room and TSC intakes are shown on Figure 2.3-382. In all cases, the intervening structures between the release point and the main control room and TSC intake were ignored for calculational simplicity, thereby underestimating the true distance to the main control room and TSC intakes. Atmospheric stability was determined by the vertical temperature difference (DT) measured over the difference in measurement height and the stability classes given in Regulatory Guide 1.23. All releases were assumed to be point ground-level releases, except the containment shell, which is assumed to be a diffuse area source. For each of the source-to-receptor combinations (Table 2.3-338), the χ/Q value that is not exceeded more than 5.0 percent of the total hours in the meteorological data set

(e.g., 95-percentile χ/Q) was determined. The χ/Q values for source-receptor pairs are shown in Table 2.3-339.

The χ/Q values calculated for the main control room HVAC intakes are conservatively applied for the main control room inleakage as well. The intake X/Q values are conservative because the values calculated by ARCON96 for other inleakage pathways would be reduced due to dispersion throughout the building as the plume travels to the main control room.

2.3.4.4 Hazardous Material Releases

Hazardous material releases and main control room habitability are discussed in Section 6.4. The methodology used to calculate concentrations of hazardous materials (e.g., flammable or toxic clouds) outside building structures resulting from the on-site and/or off-site airborne releases of such materials is also presented in this subsection. Conformance with the requirements of Regulatory Guide 1.78 is also given in this subsection.

2.3.4.5 Representativeness and Topographic Effects

As discussed in above, the on-site data are considered to be conservatively representative of meteorological conditions at the site. Topographic effects at the site were discussed in Subsection 2.3.2.2.3. The results were indicative of a flat terrain, with no appreciable effects on short-term diffusion estimates.

2.3.5 Long-Term Atmospheric Dispersion Estimates for Routine Releases

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CP COL 2.3(3) Replace the content of DCD Subsection 2.3.5 with the following.

2.3.5.1 Objective

The on-site meteorological record is used to provide realistic estimates of annual average atmospheric dilution factors to a distance of 50 mi from the plant for use in calculating the dispersion through air pathways of radionuclides released in routine plant operations.

2.3.5.2 Calculations

2.3.5.2.1 Plant Vent

The average annual dilution factors which are applicable to routine venting or other routine gaseous-effluent releases have been evaluated from the data record using the technique presented in Regulatory Guide 1.111.

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

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

The CPNPP normal effluent release atmospheric dispersion evaluations used the XOQDOQ program which is based on the theory that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. In predicting concentrations for longer time periods, the Gaussian distribution is assumed to be evenly distributed within each directional sector. A straight-line trajectory is assumed between the point of release and all receptors. The program implements the assumptions outlined in Section C (excluding Sections C.1.a and C.1.b) of NRC Regulatory Guide (RG) 1.111. FSAR Section 2.3 provides extensive evaluations of wind speed, wind direction, "atmospheric stability, mixing height, and precipitation for the CPNPP site and surrounding meteorological stations, which demonstrates that the CPNPP meteorological data is sufficient to represent conditions between the site and the nearest receptors and conditions out to a distance of 50 miles from the site. There is no evidence of any spatial or temporal variations in atmospheric transport and diffusion conditions that would invalidate the use of a constant mean wind direction model (such as XOQDOQ).

Meteorological data for the period from 2001 through 2004 and 2006 were used, and receptor locations were determined from the locations given in the current land-use census (Reference 2.3-223). An assumed release point located at the center point between Units 3 and 4 was used to calculate χ/Q and D/Q values beyond the EAB. For χ/Q and D/Q values calculated at the EAB, the distance is measured from an assumed release boundary, with a 670 ft radius from the containment centerline, to the EAB. Hourly meteorological data were used in the development of joint frequency distributions, in hours, of wind direction and wind speed by atmospheric stability class. The wind speed categories used were consistent with the CPNPP short-term (accident) diffusion χ/Q calculation discussed above. Calms were distributed as the first wind speed class.

Joint frequency distribution tables were developed from the hourly meteorological data with the assumption that if data required as input to the XOQDOQ program (i.e., lower level wind direction and wind speed, and temperature differential as opposed to upper level wind direction and wind speed) were missing from the

hourly data record, all data for that hour would be discarded. This assumption maximizes the data being included in the calculation of the χ/Q and D/Q values because hourly data are not discarded if only upper data is missing.

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

Building cross-sectional area and building height are used in calculation of building wake effects. Regulatory Guide 1.111 identifies the tallest adjacent building, in many cases the reactor building, as appropriate for use. A conservative building area of 2500 m^2 and a building height of 69.9 m were used in the calculation of building wake effects.

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

Terrain recirculation factor was not considered because the meteorological data does not show any conclusive or systematic up and down or cross valley flow.

Off-site receptor locations for the CPNPP site were also evaluated (Table 2.3-336). χ/Q and/or D/Q at points of potential maximum concentration outside the site boundary, at points of maximum individual exposure, and at points within a radial grid of sixteen 22½ degree sectors (centered on true north, north-northeast, northeast, etc.) and extending to a distance of 80 km (50 mi) from the station were determined. A set of data points were located within each sector at increments of 0.4 km (0.25 mi) to a distance of 1.6 km (1 mi) from the plant, at increments of 0.8 km (0.5 mi) from a distance of 1.6 km (1 mi) to 8 km (5 mi), at increments of 4 km (2.5 mi) from a distance of 8 km (5 mi) to 16 km (10 mi), and at increments of

8 km (5 mi) thereafter to a distance of 80 km (50 mi). Estimates of χ/Q (undecayed and undepleted; depleted for radioiodines) and D/Q radioiodines and particulates is provided at each of these grid points. Receptor locations representing recreational users of Squaw Creek Reservoir (SCR) were also evaluated. The limiting SCR receptor locations are given in Table 2.3-336.

The results of the analysis, based on the five years of on-site data for years 2001 through 2004 and 2006, are presented in Tables 2.3-340, 2.3-341, 2.3-342, 2.3-343, 2.3-344, 2.3-345, and 2.3-346.

Annual average undecayed and undepleted dilution factors to a distance of 50 mi from the plant are shown in Table 2.3-340. The maximum value at the actual EAB is 5.5 x 10^{-6} seconds/meter^3 and occurs north-northwest of the plant at a distance of 0.37 mi. There are no higher values beyond the site boundary because for ground level releases concentrations monontonically decrease from the release point to all locations downwind. Annual values for undecayed and depleted χ/Qs are given in Table 2.3-241. Annual average undecayed and undepleted dilution and deposition factors for special off-site receptor locations, including recreational users of SCR, are given in Table 2.3-342. Values for eight day decay depleted χ/Qs are given in Table 2.3-244 D/Q values out to a distance of 50 mi are given in Table 2.3-245.

2.3.5.2.2 Evaporation Pond

An additional CPNPP Units 3 and 4 gaseous release source is the evaporation pond (EP). The purpose of the EP is to prevent tritium concentration in the SCR from exceeding the limit described in the existing CPNPP Off-site Dose Calculation Manual (ODCM) , Revision 26, due to tritium discharge from CPNPP Units 3 and 4. The EP decreases the level of tritium discharge into the SCR by accepting liquid wastes, including tritium, from the liquid waste management system (LWMS) and evaporating the liquid wastes by natural processes. The atmospheric transport and dispersion of radioactive materials, in the form of aerosols, vapors, or gases, released from the EP are discussed below.

The χ/Q and D/Q values for the evaporation pond are determined at points of potential maximum concentration, outside the site boundary, at points of maximum individual exposure and at points within a radial grid of sixteen 22.5° sectors extending to a distance of 50 miles. Radioactive decay and dry deposition are considered. The atmospheric dispersion calculation uses meteorological data collected at CPNPP for the five-yr period beginning January 1, 2001 and ending December 31, 2006, excluding January 1 through December 31 of 2005.

The evaporation pond is located approximately 0.4 mi southwest of CPNPP Units 3 and 4 power blocks. Given the distance from the power block, the effects of building wake are conservatively neglected in the atmospheric dispersion analysis. Consistent with the guidance of Regulatory Guide 1.111, a ground level release mode is used. The release elevation of the EP is 0.0 m relative to the plant grade. The evaporation pond has a surface area of approximately one acre.

Although the evaporation pond is a diffuse area source, in the atmospheric dispersion evaluation, it is assumed to be a point source. This assumption is conservative because, for a given release rate, a ground level point source has a higher concentration than a ground level diffuse area source at the release location and locations downwind. Near ground level releases usually produce concentrations that decrease from the release point to all locations downwind. Therefore, for distant receptors, the assumption of a point source results in conservatively high relative concentrations.

Distances from the center of the evaporation pond to the closest point on the EAB in each of the 16 compass directions are given in Table 2.3-349. The nearest receptor locations include residences or locations at which plants or animals that become food for the public may be exposed to either direct radiation or contamination. No milk or meat animals (cows or goats) were identified near the CPNPP based on the land-use census presented in the CPNPP Annual Radiological Environmental Operating Report for 2006 (AREOR). For each of the 16 compass directions, the shortest distance from the center point of the evaporation pond to a receptor within a 45° angle centered on the compass direction was used. Because of this conservative methodology, the nearest garden is captured in both the ENE and E sectors instead of just the ENE sector (the direction relative to Units 1 and 2 given in the ODCM). The distances from the center point of the evaporation pond to the nearest receptor in each sector are given in Table 2.3-350. The XOQDOQ software (NUREG/CR-2919) was used to determine the EP atmospheric dispersion values.

From Table 2.3-348, the highest χ/Q and D/Q values for the EAB occur in the south sector and are 5.2×10^{-5} s/m^3 and 2.3×10^{-7} m^{-2}, respectively. Table 2.3-348 gives the annual average χ/Qand D/Q values for no decay, undepleted, as well as 2.26 day decay, undepleted and 8.00 day decay, depleted.

There are no meat animals identified in the area surrounding the CPNPP site. Therefore, it is assumed that the χ/Q and D/Q values at any location of meat animals within five miles of the plant would be bounded by values determined at other receptors, and no specific χ/Q or D/Q values are provided.

2.3.6 Combined License Information

CP COL 2.3(1) ***2.3(1)*** *Site Meteorology*

This COL item is addressed in Subsections 2.3.1 and 2.3.2 and associated tables.

CP COL 2.3(2) ***2.3(2)*** *Short term atmospheric transport and diffusion*

This COL item is addressed in Subsection 2.3.4 and associated tables.

CP COL 2.3(3) ***2.3(3)*** *Long term atmospheric transport and diffusion*

This COL item is resolved in Subsection 2.3.5 and associated tables.

2.3.7 References

CP SUP 2.3(1) Add the following references after the last reference in DCD Subsection 2.3.7.

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Table 2.3-201 (Sheet 1 of 2)
Texas Weather Records

CP COL 2.3(1)

Temperature (°F)			
Coldest	-23	Tulia (40 mi S of Amarillo)	Feb. 12, 1899
		Seminole (65 mi SW of Lubbock)	Feb. 8, 1933
Hottest	120	Seymour (180 mi NW of Dallas)	Aug. 12, 1936
		Monahans (45 mi SW of Midland)	Jun. 28, 1994
Warmest yr statewide	68.6		1921
Coldest yr statewide	63.2		1976
Highest monthly average	102.4	Presidio (210 mi SE of El Paso)	June 1962
Lowest monthly average	19.4	Dalhart (60 mi NW of Amarillo)	January 1959
Highest annual average	74.1	McAllen (60 mi NW of Brownsville)	1988
Lowest annual average	56.1	Dalhart (60 mi NW of Amarillo)	1959
Rainfall (in)			
Greatest in a 24-hour period	29.05	Albany (105 mi W of Fort Worth)	Aug. 4, 1978
Greatest in one month	35.70	Alvin (20 mi SE of Houston)	July 1979
Greatest in one yr	109.38	Clarksville (105 mi NE of Dallas)	1873
Least in one yr	1.64	Presidio (210 SE of El Paso)	1956
Snowfall (in)			
Greatest in a 24-hour period	24.0	Plainview (45 mi N of Lubbock)	Feb.3-5, 1956
Greatest maximum depth	33.0	Hale Center (35 mi N of Lubbock)	Feb. 4, 1956
		Vega (25 mi w of Amarillo)	Feb. 4, 1956
Greatest in a single storm	61.0	Vega	Feb. 1-8, 1956
Greatest in one month	61.0	Vega	Feb. 1956

Table 2.3-201 (Sheet 2 of 2)
Texas Weather Records

Greatest in one season	65.0	Romero	1923-24
Wind (mi per hour)			
Highest sustained speed	SE 145	Matagorda (70 mi SW of Houston) Hurricane Carla	Sep. 11, 1961
	NE 145	Port Lavaca (70 mi NE of Corpus Christi) Hurricane Carla	Sep. 11, 1961
Highest peak gust	SW 180	Aransas Pass (20 m E of Corpus Christi) Hurricane Celia	Aug. 3, 1970
Hazardous Weather			
Longest and worst drought			1950-1956
Worst heat wave			1980
Most damage from 1 tornado	442M	Wichita Falls (120 mi NW of Dallas)	April 10, 1979
Most tornadoes in 1 yr	232		1967
Most tornadoes in 1 month	124		Sep. 1967
Most damage from 1 hailstorm	1.2B **U.S. record	Parker, Tarrant counties	May 5, 1995
Deadliest hurricane	6000-8000	Galveston	Sep. 8, 1900
Most damaging hurricane	3.0 Billion	Hurricane Alicia	Aug. 18, 1983

Notes:

Data from: Texas Weather Records, s.v. “Texas Web Guide”, http://web2.airmail.net/danb1/records.htm, accessed January 6, 2008 8:31 PM.

Table 2.3-202 (Sheet 1 of 4)
Dallas-Fort Worth TX (DFW)
Normals, Means, and Extremes

CP COL 2.3(1)

	ELEMENT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Yr
TEMPERATURE °F	NORMAL DAILY MAXIMUM	54.1	60.1	68.3	75.9	83.2	91.1	95.4	94.8	87.7	77.9	65.1	56.5	75.8
	MEAN DAILY MAXIMUM	54.9	59.9	67.5	76.2	83.3	91.5	95.9	95.8	88.4	78.6	66.3	57.9	76.3
	HIGHEST DAILY MAXIMUM	88	95	96	95	103	113	110	109	111	102	89	88	113
	YR OF OCCURRENCE	1969	1996	1991	1990	1985	1980	1998	2003	2000	1979	1989	1955	JUN-80
	MEAN OF EXTREME MAXS.	76.1	79.8	85	89.1	94.2	98.6	102.7	103.3	98.6	92.4	83	77.1	90
	NORMAL DAILY MINIMUM	34	38.7	46.4	54	63	70.7	74.6	74	67.2	56.4	45.1	36.8	55.1
	MEAN DAILY MINIMUM	33.9	38.3	45.3	54.6	63.1	70.9	74.7	74.2	67	56.2	44.9	37	55
	LOWEST DAILY MINIMUM	4	7	15	29	41	51	59	56	43	29	20	-1	-1
	YR OF OCCURRENCE	1964	1985	2002	1989	1978	1964	1972	1967	1984	1993	1959	1989	DEC-89
	MEAN OF EXTREME MINS.	16.2	21.6	27.6	37.7	49.8	60.5	67.8	66	52.6	40.5	28.7	20.7	40.8
	NORMAL DRY BULB	44.1	49.4	57.4	65	73.1	80.9	85	84.4	77.5	67.2	55.1	46.7	65.5
	MEAN DRY BULB	44.5	49.1	56.3	65.4	73.1	81.2	85.3	85.1	77.6	67.3	55.5	47.3	65.6
	MEAN WET BULB	40.5	44.7	50.4	58	66.7	72.3	73.9	73.2	68	59.7	50.2	42.5	58.3
	MEAN DEWPOINT	34.3	37.9	43.9	51.9	62.4	68.2	68.7	67.5	62.5	54.1	44.5	36.5	52.7
	NORMAL NUMBER OF DAYS WITH:													
	MAXIMUM ≥ 90°	0	0.1	0.2	0.8	5.6	20.2	28.2	27.1	15.7	3	0	0	100.9
	MAXIMUM ≤ 32°	1.5	0.9	0.1	0	0	0	0	0	0	0	0	1.1	3.6
	MINIMUM ≤ 32°	13.5	6.9	2	0.2	0	0	0	0	0	0.1	2.9	10	35.6
	MINIMUM ≤ 0°	0	0	0	0	0	0	0	0	0	0	0	*	0
H/C	NORMAL HEATING DEG. DAYS	650	448	248	74	13	0	0	0	2	52	312	571	2370
	NORMAL COOLING DEG. DAYS	3	7	10	72	265	478	621	601	376	118	15	2	2568

CP COL 2.3(1)

Table 2.3-202 (Sheet 2 of 4)
Dallas-Fort Worth TX (DFW)
Normals, Means, and Extremes

	ELEMENT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Yr
RH	NORMAL (PERCENT)	68	66	64	64	70	67	60	59	64	66	69	70	66
	HOUR 00 LST	74	71	69	71	78	74	67	66	71	73	74	73	72
	HOUR 06 LST	80	80	79	81	87	86	81	80	83	83	82	80	82
	HOUR 12 LST	61	58	56	55	59	55	49	49	54	55	58	60	56
	HOUR 18 LST	59	55	51	51	57	52	45	45	51	54	60	60	53
S	PERCENT POSSIBLE SUNSHINE	52	54	58	61	57	67	75	73	67	63	57	52	61
W/0	MEAN NO. DAYS WITH:													
	HEAVY FOG(VISBY ≤ 1/4 MI)	2.4	1.5	1.0	0.6	0.3	0.1	0.0	0.0	0.1	0.8	1.5	2.4	10.7
	THUNDERSTORMS	1.5	1.9	4.4	6.0	7.7	6.2	4.6	4.5	3.4	3.0	2.1	1.3	46.6
CLOUDINESS	MEAN:													
	SUNRISE-SUNSET (OKTAS)			4.0		4.0	3.2						4.8	
	MIDNIGHT-MIDNIGHT (OKTAS)			4.0										
	MEAN NUMBER OF DAYS WITH:													
	CLEAR	2.0	6.0	15.0		10.0	11.0							
	PARTLY CLOUDY		2.0			4.0	8.0							
	CLOUDY	2.0		7.0		6.0	2.0							
PR	MEAN STATION PRESSURE (IN)	29.49	29.49	29.40	29.30	29.30	29.30	29.40	29.40	29.39	29.40	29.40	29.50	29.40
	MEAN SEA-LEVEL PRES. (IN)	30.14	30.09	30.01	29.93	29.90	29.91	29.96	29.96	29.98	30.04	30.08	30.13	30.01

Table 2.3-202 (Sheet 3 of 4) Dallas-Fort Worth TX (DFW) Normals, Means, and Extremes

CP COL 2.3(1)

	ELEMENT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Yr
WINDS	MEAN SPEED (MPH)	11.0	11.7	12.6	12.4	11.1	10.6	10.0	9.1	9.4	9.9	11.0	11.1	10.8
	PREVAIL.DIR(TENS OF DEGS)	18	18	18	18	18	18	18	18	16	16	18	18	18
	MAXIMUM 2-MINUTE:													
	SPEED (MPH)	41	51	48	47	43	51	41	47	33	46	40	39	51
	DIR. (TENS OF DEGS)	29	23	30	30	34	32	06	33	24	23	28	31	32
	YR OF OCCURRENCE	1996	2000	2000	2000	1998	2002	2002	1996	1996	2001	2001	2003	JUN 2002
	MAXIMUM 5-SECOND:													
	SPEED (MPH)	51	78	74	64	55	57	53	47	39	54	47	47	78
	DIR. (TENS OF DEGS)	19	23	27	26	28	34	06	34	19	23	30	28	23
	YR OF OCCURRENCE	1996	2000	2000	2000	2000	1996	2002	2002	2001	2001	1998	2003	FEB 2000
PRECIPITATION	NORMAL (IN)	1.90	2.37	3.06	3.20	5.15	3.23	2.12	2.03	2.42	4.11	2.57	2.57	34.73
	MAXIMUM MONTHLY (IN)	5.07	7.40	7.39	12.19	13.66	8.75	11.13	6.85	9.52	14.18	6.95	8.75	14.18
	YR OF OCCURRENCE	1998	1997	2002	1957	1982	1989	1973	1970	1964	1981	2000	1991	OCT 1981
	MINIMUM MONTHLY (IN)	T	0.15	0.10	0.11	0.95	0.40	0.00	0.00	0.09	T	0.20	0.17	0.00
	YR OF OCCURRENCE	1986	1963	1972	1987	1996	1964	1993	2000	1984	1975	1970	1981	AUG 2000
	MAXIMUM IN 24 HOURS (IN)	3.46	4.06	4.39	4.55	5.34	3.15	3.83	4.05	4.76	5.91	2.83	4.22	5.91
	YR OF OCCURRENCE	2002	1965	1977	1957	1989	1989	2001	1976	1965	1959	1964	1991	OCT 1959
	NORMAL NUMBER OF DAYS WITH:													
	PRECIPITATION $\geq$ 0.01	7.2	6.3	7.8	7.1	9.3	7.2	4.3	4.5	5.9	6.7	6.4	6.5	79.2
	PRECIPITATION $\geq$ 1.00	0.3	0.7	0.8	1.1	1.8	0.8	0.6	0.6	0.8	1.4	0.7	0.6	10.2

CP COL 2.3(1)

Table 2.3-202 (Sheet 4 of 4) Dallas-Fort Worth TX (DFW) Normals, Means, and Extremes

	ELEMENT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Yr
SNOWFALL	NORMAL (IN)	0.8	1.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.*	0.2	0.2	2.5
	MAXIMUM MONTHLY (IN)	12.1	13.5	2.5	T	T	0.0	0.0	0.0	0.0	T	5.0	2.6	13.5
	YR OF OCCURRENCE	1964	1978	1962	1995	1995					1993	1976	1963	FEB 1978
	MAXIMUM IN 24 HOURS (IN)	12.1	7.5	2.5	T	T	0.0	0.0	0.0	0.0	T	4.8	2.5	12.1
	YR OF OCCURRENCE	1964	1978	1962	1995	1995					1993	1976	1963	JAN 1964
	MAXIMUM SNOW DEPTH (IN)	6	8	2	0	0	0	0	0	0	0	3	2	8
	YR OF OCCURRENCE	1964	1978	1971								1976	1983	FEB 1978
	NORMAL NO. DAYS WITH:													
	SNOWFALL ≥ 1.0	0.4	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	1.0

	Annual Exceedance[3]		
	0.4%	1%	2%
Dry Bulb	100 F	98 F	96 F
Coincident Wet Bulb	74 F	74 F	74 F
Non-coincident Wet Bulb	78 F	77 F	76 F

100-yr Return Period	
Extreme Maximum Dry Bulb	115 F
Coincident Wet Bulb	78 F
Extreme Minimum Dry Bulb	-5 F

NOTES:
1. Dallas-Fort Worth, Texas (WBAN Station No. 3927), Monthly Climate Summary, Period of Record: 1971 to 2000.
2. Reference: Southern Regional Climate Center, http://www.srcc.lsu.edu/southernClimate/atlas/
3. ASHRAE Fundamentals, Chapter 14, 2009 (Reference 2.3-235).

Table 2.3-203 (Sheet 1 of 4) Dallas Love Field, TX Normals, Means, and Extremes

CP COL 2.3(1)

	Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Temperature (°F)	Mean Daily Max[a]	55.4	61	69.1	76.5	83.8	91.6	96.1	95.8	88.5	78.6	66	57.4	76.7
	Mean Daily Min[a]	36.4	41	48.5	56.1	64.9	72.7	76.8	76.4	69.2	58.2	46.8	38.6	57.1
	Daily Mean	45.9	51	58.8	66.3	74.4	82.2	86.5	86.1	78.9	68.4	56.4	48	66.9
	Highest Daily Extreme[b]	95	95+	98	99	103+	112+	111	115	110	100+	92	89	115
	Yr	1911	1996	1911	1963	1985	1980	1954	1909	2000	1979	1910	1955	9-Aug
	Day	31	22	10	10	31	27	25	18	4	1	24	24	18
	Highest Monthly Mean[a]	53.6	59.7	64.5	71.1	80.8	87.2	92.1	90.5	84.8	71.9	62.6	54.7	92.1
	Yr	1990	1976	1974	1981	1996	1998	1998	2000	1998	1998	1999	1984	Jul-98
	Lowest Daily Extreme[b]	2	2+	12	29+	36+	48	57	55+	40+	26	15+	1	1
	Yr	1949	1910	1948	1914	1908	1903	1905	1906	1908	1910	1911	1989	Dec-89
	Day	31	19	11	10	1	1	10	29	29	30	30	23	23
	Lowest Monthly Mean[a]	35.1	38.3	54.9	60.9	69.1	78.8	82.4	81	69.1	61.2	50	35.8	35.1
	Yr	1978	1978	1996	1983	1976	1989	1976	1992	1974	1976	1976	1983	Jan-78
	Mean Number of Days with[c]													
	Max > = 100	0	0	0	0	0.1	1.6	8.3	8.6	1.7	@	0	0	20.3
	Max > = 90	0	0.1	0.3	1.1	6.9	20.5	27.8	26.9	15.9	3.3	0	0	102.8
	Max > = 50	20.5	22.1	29.4	30	31	30	31	31	30	30.9	27.7	23.4	337
	Max < = 32	1.4	0.8	0.1	0	0	0	0	0	0	0	0	0.9	3.2
	Min < = 32	11.6	6.3	1.8	@	0	0	0	0	0	0.1	2.2	8.2	30.2
	Min <= 0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-203 (Sheet 2 of 4)
Dallas Love Field, TX
Normals, Means, and Extremes

CP COL 2.3(1)

		Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Precipitation (in)	Precipitation Totals	Mean[a]	1.89	2.31	3.13	3.46	5.30	3.92	2.43	2.17	2.65	4.65	2.61	2.53	37.05
		Median[a]	1.93	2.09	2.65	3.40	5.91	2.97	2.06	1.79	2.30	3.43	2.14	2.02	36.98
		Highest Daily Extreme[b]	5.14	3.35	6.02	5.10	5.14	3.64	4.62	4.42	4.32	6.01	3.40	3.98	6.02
		Yr	1949	1997	1977	1957	1949	1989	1962	1915	1965	1959	1902	1991	Mar-77
		Day	24	12	27	26	17	13	27	18	21	1	4	20	27
		Highest Monthly[a]	5.49	7.91	9.09	8.05	10.56	10.87	6.14	5.98	7.16	16.05	7.01	9.25	16.05
		Yr	1998	1997	1977	1997	1989	1989	1988	1974	1974	1981	2000	1991	Oct-81
		Lowest Monthly Extreme[a]	.00+	0.17	0.26	0.04	0.54	1.26	.00+	.00+	0.03	.00	0.17	0.05	.00+
		Yr	1988	1996	1972	1983	1977	1983	2000	2000	2000	1975	1979	1981	Aug-00
	Mean Number of Days with Daily Precipitation[c]														
		>= 0.01	7.2	6.1	7.4	7.2	9.3	7.2	4.7	4.6	5.8	7.1	6.6	6.4	79.6
		>= 0.10	3.7	3.8	4.7	4.7	6.3	4.8	3.4	3.1	3.9	5.0	4.2	3.9	51.5
		>= 0.50	1.1	1.5	2.2	2.5	3.6	2.8	1.5	1.6	1.8	2.8	2.0	1.9	25.3
		>= 1.00	0.3	0.8	0.9	1.1	2.0	1.5	0.7	0.6	0.8	1.6	0.7	0.8	11.8

Table 2.3-203 (Sheet 3 of 4) Dallas Love Field, TX Normals, Means, and Extremes

CP COL 2.3(1)

		Element		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Snow Totals (in)	Means/Medians(d)	Snow Fall Mean		0.7	0.6	.0	.0	.0	.0	.0	.0	.0	.0	0.2	0.2	1.7
		Snow Fall Median		.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	#	#
		Snow Depth Mean		#	#	#	0	#	0	0	0	0	0	#	#	N/A
		Snow Dept Median		0	0	0	0	0	0	0	0	0	0	0	0	N/A
	Extremes(b)	Highest Daily Snow Fall		4.5	6.0	0.8	.0	.0	.0	.0	.0	.0	.0	3.1	4.0	6.0
			Yr	1977	1978	1971	0	0	0	0	0	0	0	1976	1983	Feb-78
			Day	30	17	2	0	0	0	0	0	0	0	13	16	17
		Highest Monthly Snow Fall		5.5	10.1	0.8	.0	.0	.0	.0	.0	.0	.0	3.1	4.0	10.1
			Yr	1977	1978	1971	0	0	0	0	0	0	0	1976	1983	Feb-78
		Highest Daily Snow Depth		4	4	1+	0	0	0	0	0	0	0	3	2	4+
			Yr	1977	1978	1989	0	0	0	0	0	0	0	1976	1983	Feb-78
			Day	31	18	6	0	0	0	0	0	0	0	14	16	18
		Highest Monthly Mean Snow Depth		#	1	#	0	#	0	0	0	0	0	#	#	1
			Yr	1988	1978	1989	0	1997	0	0	0	0	0	1993	1983	Feb-78

Table 2.3-203 (Sheet 4 of 4) Dallas Love Field, TX Normals, Means, and Extremes

CP COL 2.3(1)

		Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Mean Number of Days[d]	Snow Fall >= Thresholds	0.1	0.6	0.5	0.1	.0	.0	.0	.0	.0	.0	.0	0.1	0.1	1.4
		1.0	0.4	0.3	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	0.7
		3.0	0.1	@	.0	.0	.0	.0	.0	.0	.0	.0	@	@	0.1
		5.0	.0	@	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	@
		10.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
	Snow Depth >= Thresholds	1	0.5	0.5	0.1	.0	.0	.0	.0	.0	.0	.0	0.2	@	1.3
		3	0.2	0.2	.0	.0	.0	.0	.0	.0	.0	.0	@	.0	0.4
		5	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
		10	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0

a) From the 1971 – 2000 Monthly Normals

b) Derived from station's available digital record: 1897 – 2001

c) Derived from 1971 – 2000 serially complete daily data

d) Derived from Snow Climatology and 1971 – 2000 daily data

+ Also occurred on an earlier date(s)
@ Denotes mean number of days greater than 0 but less than .05
#Denotes trace amounts
** Statistics not computed because less than six yr out of thirty had measurable precipitation

Notes:

1. Data From: Climatography of the United States, No. 20, 1971 – 2000, Station: DALLAS LOVE AP, TX, COOP ID: 412244 Complete documentation available from: www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

Table 2.3-204 (Sheet 1 of 4) Mineral Wells, TX Normals, Means, and Extremes

CP COL 2.3(1)

	Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Temperature (°F)	Mean Daily Max[a]	58.2	63.7	72.5	79.6	86.1	92.8	97.3	96.6	89.7	80.5	67.8	59.6	78.7
	Mean Daily Min[a]	33.4	38.0	46.1	53.1	62.0	68.7	72.2	71.7	65.3	55.3	44.3	35.6	53.8
	Daily Mean	45.8	50.9	59.3	66.4	74.1	80.8	84.8	84.2	77.5	67.9	56.1	47.6	66.3
	Highest Daily Extreme[b]	91	97	96+	100+	106+	114+	112	110	111	104	93	90	114+
	Yr	1969	1996	1995	1972	2000	1980	1954	1964	2000	1951	1980	1955	Jun-80
	Day	8	21	22	12	24	28	25	6	4	3	9	24	28
	Highest Monthly Mean[a]	53.2	60.1	64.7	71.5	80.7	85.7	89.7	89.8	84.1	71.1	63.6	53.0	89.8
	Yr	1990	2000	2000	1981	1996	1980	1998	1999	1998	1979	1999	1984	Aug-99
	Lowest Daily Extreme[b]	4+	3	12	28	39	51	58	47	40+	23	12	-8	-8
	Yr	1966	1951	1980	2000	1954	1983	1971	1967	1989	1993	1950	1989	Dec-89
	Day	23	2	2	4	3	1	31	12	25	31	11	23	23
	Lowest Monthly Mean[a]	35.7	38.4	54.4	59.3	68.4	75.1	80.0	80.0	69.1	59.3	48.5	35.9	35.7
	Yr	1978	1978	1983	1983	1983	1983	1976	1971	1974	1976	1976	1983	Jan-78
	Mean Number of Days with[c]													
	Max > = 100	.0	.0	.0	@	0.3	2.4	9.0	9.9	2.0	0.1	.0	.0	23.7
	Max > = 90	.0	0.2	0.6	2.3	8.4	20.9	28.3	27.4	17.7	3.7	0.1	.0	109.6
	Max > = 50	22.0	23.0	29.7	30.0	31.0	30.0	31.0	31.0	30.0	30.8	28.0	24.4	340.9
	Max < = 32	1.2	0.6	@	.0	.0	.0	.0	.0	.0	.0	.0	0.9	2.7
	Min < = 32	15.6	9.0	3.5	0.5	.0	.0	.0	.0	.0	0.2	4.6	13.1	46.5
	Min <= 0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	0.1	0.1

Table 2.3-204 (Sheet 2 of 4)
Mineral Wells, TX
Normals, Means, and Extremes

CP COL 2.3(1)

		Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Precipitation (in)	Precipitation Totals	Mean[1]	1.42	1.99	2.69	2.75	4.59	3.25	2.25	2.34	2.80	3.81	2.16	1.74	31.79
		Median[1]	1.29	1.53	2.35	2.51	3.97	2.08	1.95	2.33	2.49	3.04	1.60	1.47	30.72
		Highest Daily Extreme[2]	2.46	2.48	4.18	5.15	4.09+	5.23	6.24	3.42	3.48	6.65	2.84+	3.25	6.65
		Yr	1968	1997	1977	1978	1983	2000	1962	1991	1993	1981	1996	1991	Oct-81
		Day	18	20	27	10	23	4	27	12	14	12	24	20	12
		Highest Monthly[1]	3.88	8.53	7.87	12.30	12.68	10.30	8.07	6.95	8.57	20.00	5.92	8.50	20.00
		Yr	1973	1997	1977	1990	1982	2000	1973	1996	1980	1981	1994	1991	Oct-81
		Lowest Monthly Extreme[1]	.03	0.25	0.43	0.23	1.03	0.10	.01	0.00	0.03	0.16	.00	0.04	.00+
		Yr	1976	1996	1971	1983	1988	1980	1993	2000	1983	1975	1999	1973	Aug-00
	Mean Number of Days with Daily Precipitation[c]														
		>= 0.01	6.8	6.4	7.5	7.0	8.7	6.9	5.3	5.7	5.1	7.3	6.4	7.2	80.3
		>= 0.10	3.2	3.7	4.4	4.1	6.2	4.6	2.9	3.5	3.2	4.5	3.7	3.6	47.6
		>= 0.50	0.8	1.3	1.6	1.4	3.3	2.2	1.3	1.3	1.4	2.4	1.4	1.2	19.6
		>= 1.00	0.2	0.3	0.6	0.8	1.4	1.1	0.7	0.5	0.8	1.1	0.5	0.4	8.4

CP COL 2.3(1)

Table 2.3-204 (Sheet 3 of 4)
Mineral Wells, TX
Normals, Means, and Extremes

		Element		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Snow Totals (in)	Means/Medians(d)	Snow Fall Mean		0.6	0.8	.1	#	.0	.0	.0	.0	.0	#	0.1	0.2	1.8
		Snow Fall Median		.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	#	#
		Snow Depth Mean		#	#	#	0	#	0	0	0	0	0	#	#	N/A
		Snow Dept Median		0	0	0	0	0	0	0	0	0	0	0	0	N/A
	Extremes(b)	Highest Daily Snow Fall		4.0	4.0	2.0	#	.0	.0	.0	.0	.0	#	1.6	4.0	4.0+
			Yr	1977	1978	1971	1996	0	0	0	0	0	1993	1995	1975	Feb-78
			Day	30	15	2	12	0	0	0	0	0	30	28	25	15
		Highest Monthly Snow Fall		5.0	8.5	2.0	#+	.0	.0	.0	.0	.0	#	1.6	4.0	8.5
			Yr	1977	1978	1971	1996	0	0	0	0	0	1993	1995	1975	Feb-78
		Highest Daily Snow Depth		2+	3+	2	0	0	0	0	0	0	#	1+	1+	3+
			Yr	1997	1980	1971	0	0	0	0	0	0	1991	1996	1982	Feb-80
			Day	8	10	3	0	0	0	0	0	0	26	25	31	10
		Highest Monthly Mean Snow Depth		#	#	#	0	#	0	0	0	0	0	#	#	#+
			Yr	1997	1996	1971	0	2000	0	0	0	0	0	1996	1982	May-00

Table 2.3-204 (Sheet 4 of 4)
Mineral Wells, TX
Normals, Means, and Extremes

CP COL 2.3(1)

Element			Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Mean Number of Days(d)	Snow Fall >= Thresholds	0.1	0.5	0.5	0.0	.0	.0	.0	.0	.0	.0	.0	0.2	0.1	1.3
		1.0	0.4	0.3	0.0	.0	.0	.0	.0	.0	.0	.0	0.0	0.1	0.8
		3.0	@	0.1	0	.0	.0	.0	.0	.0	.0	.0	0	0.1	0.2
		5.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
		10.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
	Snow Depth >= Thresholds	1	0.3	0.4	@	.0	.0	.0	.0	.0	.0	.0	0.1	0.1	0.9
		3	.0	0.1	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	0.1
		5	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
		10	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0

a) From the 1971 – 2000 Monthly Normals

b) Derived from station's available digital record: 1897 – 2001

c) Derived from 1971 – 2000 serially complete daily data

d) Derived from Snow Climatology and 1971 – 2000 daily data

+ Also occurred on an earlier date(s)
@ Denotes mean number of days greater than 0 but less than .05
#Denotes trace amounts
** Statistics not computed because less than six yr out of thirty had measurable precipitation

Notes:

1. Data From: Climatography of the United States, No. 20, 1971 – 2000, Station: MINERAL WELLS AP, TX, COOP ID: 415958. Complete documentation available from: www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

Table 2.3-205 (Sheet 1 of 4) Glen Rose, TX Normals, Means, and Extremes

CP COL 2.3(1)

	Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Temperature (°F)	Mean Daily Max[a]	58.2	64.0	72.3	79.6	86.0	92.5	97.3	97.0	89.5	80.5	68.5	59.9	78.8
	Mean Daily Min[a]	28.9	33.7	41.9	49.7	58.7	66.6	69.5	67.7	61.7	50.9	39.9	31.1	50.0
	Daily Mean	43.6	48.9	57.1	64.7	72.4	79.6	83.4	82.4	75.6	65.7	54.2	45.5	64.4
	Highest Daily Extreme[b]	89	96+	101	100	105	110+	110+	115	110	99+	95	86	115
	Yr	1969	1996	1974	1990	1967	1994	1978	1984	1985	1983	1980	1973	Aug-84
	Day	8	22	31	29	11	26	15	19	1	4	8	12	19
	Highest Monthly Mean[a]	48.5	57.8	64.2	70.5	78.1	84.6	89.6	88.0	83.2	69.2	61.5	53.1	89.6
	Yr	1990	1976	1974	1972	1996	1980	1978	1999	1977	1979	1973	1984	Jul-78
	Lowest Daily Extreme[b]	-1+	-8	7	16	29	47	45	41	30	9	5	-15	-15
	YR	1982	1996	1996	1994	1999	1993	1994	1992	1989	1993	1993	1989	Dec-89
	Day	14	4	9	7	7	1	28	28	25	31	27	23	23
	Lowest Monthly Mean[a]	35.6	39.0	51.3	59.2	66.5	76.2	79.4	75.6	70.0	59.0	46.1	35.2	35.2
	Yr	1978	1978	1996	1993	1999	1983	1976	1992	1989	1993	1993	1983	Dec-83
	Mean Number of Days with[c]													
	Max > = 100	.0	.0	@	@	0.4	2.5	10.9	12.6	2.4	.0	.0	.0	28.8
	Max > = 90	.0	0.2	0.6	2.7	10.1	22.3	29.1	28.1	18.7	5.3	0.2	.0	117.3
	Max > = 50	23.2	23.7	30.1	30.0	31.0	30.0	31.0	31.0	30.0	30.9	28.0	25.1	344.0
	Max < = 32	0.7	0.6	@	.0	.0	.0	.0	.0	.0	.0	.0	0.6	1.9
	Min < = 32	21.1	13.3	6.9	2.2	0.2	.0	.0	.0	0.2	1.6	9.2	17.7	72.4
	Min <= 0	0.1	0.1	.0	.0	.0	.0	.0	.0	.0	.0	.0	0.1	0.3

Table 2.3-205 (Sheet 2 of 4)
Glen Rose, TX
Normals, Means, and Extremes

CP COL 2.3(1)

		Element	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Precipitation (in)	Precipitation Totals	Mean[a]	1.64	2.28	2.80	2.91	5.20	4.02	2.19	2.18	3.15	3.83	2.24	2.38	34.82
		Median[a]	1.51	1.70	2.28	2.80	4.95	3.68	1.52	1.51	2.54	3.35	1.88	1.99	33.45
		Highest Daily Extreme[b]	1.91	2.56	6.25	3.83	5.92	4.27	8.48	3.29	6.90	4.84	2.87	7.14	8.48
		Yr	1979	1998	1989	1964	1989	1988	1995	1990	1986	1991	1998	1991	Jul-95
		Day	19	26	28	21	17	1	31	4	2	27	13	20	31
		Highest Monthly[a]	3.82	9.13	7.48	8.07	11.40	10.24	9.73	9.55	12.04	10.60	5.74	11.41	12.04
		Yr	1973	1997	1989	1973	1989	1989	1995	1996	1986	1991	1998	1991	Sep-86
		Lowest Monthly Extreme[a]	.00	0.03	0.03	0.33	0.97	0.42	.00	0.12	0.05	0.28	.40	0.25	.00+
		Yr	1986	1999	1971	1987	1996	1978	1993	1973	1982	1992	1979	1973	Jul-93
	Mean Number of Days with Daily Precipitation[c]														
		>= 0.01	7.4	7.1	7.5	7.0	9.2	7.4	4.8	5.8	6.5	7.6	7.0	7.5	84.8
		>= 0.10	3.8	3.9	4.4	4.4	6.3	5.2	3.1	3.4	4.3	5.2	3.9	3.8	51.7
		>= 0.50	0.9	1.6	1.7	2.0	3.0	2.5	1.3	1.4	1.8	2.2	1.7	1.7	21.8
		>= 1.00	0.3	0.6	0.6	0.8	1.7	1.4	0.6	0.6	0.9	1.3	0.7	0.6	10.1

Table 2.3-205 (Sheet 3 of 4)
Glen Rose, TX
Normals, Means, and Extremes

CP COL 2.3(1)

		Element		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Snow Totals (in)	Means/Medians(d)	Snow Fall Mean		0.7	0.5	.2	.0	.0	.0	.0	.0	.0	.0	0.2	0.2	1.8
		Snow Fall Median		.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	0	0
		Snow Depth Mean		#	#	#	0	0	0	0	0	0	#	#	#	N/A
		Snow Dept Median		0	0	0	0	0	0	0	0	0	0	0	0	N/A
	Extremes(b)	Highest Daily Snow Fall		4.5	3.5	3.0	.0	.0	.0	.0	.0	.0	.0	3.5	3.5	4.5
			Yr	1973	1975	1978	0	0	0	0	0	0	0	1976	1983	Jan-73
			Day	11	23	3	0	0	0	0	0	0	0	13	16	11
		Highest Monthly Snow Fall		6.3	5.0	3.0	.0	.0	.0	.0	.0	.0	.0	4.0	3.5	6.3
			Yr	1978	1978	1978	0	0	0	0	0	0	0	1976	1986	Jan-78
		Highest Daily Snow Depth		5	2	1+	0	0	0	0	0	0	#	3	2	5
			Yr	1973	1979	1989	0	0	0	0	0	0	1988	1976	1978	Jan-73
			Day	11	17	6	0	0	0	0	0	0	1	13	31	11
		Highest Monthly Mean Snow Depth		#+	#+	#+	0	0	0	0	0	0	#	#+	#+	#+
			Yr	1988	1996	1989	0	0	0	0	0	0	1988	1997	1997	Dec-97

Table 2.3-205 (Sheet 4 of 4)
Glen Rose, TX
Normals, Means, and Extremes

CP COL 2.3(1)

Element			Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Mean Number of Days$^{(d)}$	Snow Fall >= Thresholds	0.1	0.5	0.4	0.1	.0	.0	.0	.0	.0	.0	.0	0.2	0.2	1.4
		1.0	0.2	0.2	0.1	.0	.0	.0	.0	.0	.0	.0	0.1	0.2	0.8
		3.0	0.2	0.1	@	.0	.0	.0	.0	.0	.0	.0	@	0.1	0.4
		5.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
		10.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
	Snow Depth >= Thresholds	1	0.4	0.2	0.1	.0	.0	.0	.0	.0	.0	.0	0.1	0.1	0.9
		3	0.1	.0	.0	.0	.0	.0	.0	.0	.0	.0	@	.0	0.1
		5	@	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	@
		10	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0

a) From the 1971 – 2000 Monthly Normals

b) Derived from station's available digital record: 1897 – 2001

c) Derived from 1971 – 2000 serially complete daily data

d) Derived from Snow Climatology and 1971 – 2000 daily data

+ Also occurred on an earlier date(s)
@ Denotes mean number of days greater than 0 but less than .05
#Denotes trace amounts
** Statistics not computed because less than six yr out of thirty had measurable precipitation

Notes:

1. Data from: Climatography of the United States, No. 20, 1971 – 2000, Station: GLEN ROSE 2 W, TX, COOP ID: 413591. Complete documentation available from: www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

CP COL 2.3(1)

Table 2.3-206 (Sheet 1 of 2)
Hurricane Landfalls in Texas

1899 – 2006

Yr	Month	Name	Category	State and Category
1900	SEP	-	4	TX 4
1909	JUL	-	3	TX 3
1909	AUG	-	2	TX 2
1910	SEP	-	2	TX 2
1912	OCT	-	1	TX 1
1913	JUN	-	1	TX 1
1915	AUG	-	4	TX 4
1916	AUG	-	3	TX 3
1919	SEP	-	4	FL 4, TX 4
1921	JUN	-	2	TX 2
1929	JUN	-	1	TX 1
1932	AUG	-	4	TX 4
1933	JUL/AUG	-	2	FL 1, TX 2
1933	SEP	-	3	TX 3
1934	JUL	-	2	TX 2
1936	JUN	-	1	TX 1
1940	AUG	-	2	TX 2, LA 2
1941	SEP	-	3	TX 3
1942	AUG	-	1	TX 1
1942	AUG	-	3	TX 3
1943	JUL	-	2	TX 2
1945	AUG	-	2	TX 2
1947	AUG	-	1	TX 1
1949	OCT	-	2	TX 2
1957	JUN	Audrey	4	TX 4, LA 4
1959	JUL	Debra	1	TX 1
1961	SEP	Carla	4	TX 4
1963	SEP	Cindy	1	TX 1
1967	SEP	Beulah	3	TX 3
1970	AUG	Celia	3	TX 3
1971	SEP	Fern	1	TX 1

CP COL 2.3(1)

Table 2.3-206 (Sheet 2 of 2)
Hurricane Landfalls in Texas

1899 – 2006				
1980	AUG	Allen	3	TX 3
1983	AUG	Alicia	3	TX 3
1986	JUN	Bonnie	1	TX 1
1989	AUG	Chantal	1	TX 1
1989	OCT	Jerry	1	TX 1
1999	AUG	Bret	3	TX 3
2003	AUG	Claudette	1	TX 1
2005	SEP	Rita	5	TX 3

Notes:

1. Data are from "Atlantic Tropical Storms And Hurricanes Affecting The United States:1899 – 2002," NOAA Technical Memorandum NWS SR-206 (Updated through 2002).
2. No tropical storms struck Texas in 2006 (see http://www.nhc.noaa.gov/2006atlan.shtml).
3. Data for 2004 and 2005 from the National Hurricane Center http://www.nhc.noaa.gov/2003claudette.shtml? and http://www.nhc.noaa.gov/pdf/TCR-AL182005_Rita.pdf

Table 2.3-207
Frequency of Tropical Cyclones (By Month)

CP COL 2.3(1)

	Category of Storm							
	1 (No.)	2 (No.)	3 (No.)	4 (No.)	5 (No.)	Monthly Total (No.)	Annual Frequency (yr-1)	% of Total
Jun	4	1	0	1	0	6	0.06	15%
Jul	1	3	1	0	0	5	0.05	13%
Aug	4	3	6	2	0	15	0.14	38%
Sep	2	1	4	3	0	10	0.09	26%
Oct	2	1	0	0	0	3	0.03	8%
Total	13	9	11	6	0	39	0.36	100%

	Number of Hurricanes: Saffir/Simpson Category Number						Landfall Frequency (storms per yr)	Return Period (yr)
Area	1	2	3	4	5	Total		
Texas	13	9	11	6	0	39	0.38	2.8

Where the definition of Storm Category is as follows:

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

NOTES:

1. Data are from "Atlantic Tropical Storms And Hurricanes Affecting The United States:1899 – 2002," NOAA Technical Memorandum NWS SR-206 (Updated through 2002).
2. No tropical storms struck Texas in 2006 (see http://www.nhc.noaa.gov/2006atlan.shtml).
3. Data for 2004 and 2005 from the National Hurricane Center http://www.nhc.noaa.gov/2003claudette.shtml and http://www.nhc.noaa.gov/pdf/TCR-AL182005_Rita.pdf

Table 2.3-208 (Sheet 1 of 2)
Tropical Storms Within 50 Mi of CPNPP

CP COL 2.3(1)

Number	Yr	Month	Day	Storm Name	Wind Speed (KTS)	Wind Speed (mph)	Pressure(MB)	Category
1	1874	9	7	NOT NAMED	30	34.5	0	TD
2	1900	9	9	NOT NAMED	65	74.8	0	H1
3	1900	9	9	NOT NAMED	50	57.5	0	TS
4	1932	8	14	NOT NAMED	35	40.3	0	TS
5	1932	8	15	NOT NAMED	25	28.8	1002	TD
6	1943	7	29	NOT NAMED	25	28.8	0	TD
7	1943	7	29	NOT NAMED	25	28.8	0	TD
8	1945	8	29	NOT NAMED	30	34.5	1002	TD
9	1945	8	29	NOT NAMED	25	28.8	1006	TD
10	1947	8	26	NOT NAMED	20	23.0	0	TD
11	1947	8	26	NOT NAMED	20	23.0	0	TD
12	1947	8	27	NOT NAMED	15	17.3	0	TD
13	1954	7	30	BARBARA	25	28.8	0	TD
14	1961	9	12	CARLA	60	69.0	975	TS
15	1961	9	12	CARLA	45	51.8	979	TS
16	1968	6	24	CANDY	30	34.5	0	TD
17	1968	6	24	CANDY	25	28.8	0	TD
18	1970	9	16	FELICE	30	34.5	1006	TD
19	1970	9	17	FELICE	25	28.8	0	TD
20	1970	9	17	FELICE	25	28.8	0	TD

Table 2.3-208 (Sheet 2 of 2)
Tropical Storms Within 50 Mi of CPNPP

CP COL 2.3(1)

Number	Yr	Month	Day	Storm Name	Wind Speed (KTS)	Wind Speed (mph)	Pressure(MB)	Category
21	1983	8	19	ALICIA	35	40.3	998	TS
22	1983	8	19	ALICIA	30	34.5	1003	TD
23	1989	8	2	CHANTAL	25	28.8	1004	TD
24	1989	8	2	CHANTAL	20	23.0	1007	TD
25	1995	8	1	DEAN	20	23.0	1004	TD
26	1995	8	1	DEAN	20	23.0	1004	TD

NOTES:

4. NOAA Coastal Services Center, Historical Hurricane Tracks, 1851 – 2006.http://maps.csc.noaa.gov/hurricanes/, accessed December 4, 2007.http://maps.csc.noaa.gov/hurricanes/viewer.html

Table 2.3-209 (Sheet 1 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
Bosque County, TX						
1 BOSQUE	4/28/1954	1700	F1	100	2	0.114
2 BOSQUE	5/18/1954	1500	F1	0	0	
3 BOSQUE	8/15/1958	1650	F1	50	1	0.028
4 BOSQUE	4/11/1961	1700	F1	100	14	0.795
5 BOSQUE	4/19/1966	1600	F2	880	2	1.000
6 BOSQUE	4/19/1966	1630	F1	0	0	
7 BOSQUE	10/26/1970	1730	F0	33	1	0.019
8 BOSQUE	10/26/1970	1730	F0	50	0	
9 BOSQUE	5/9/1971	1755	F2	67	0	
10 BOSQUE	5/25/1976	1315	F2	33	4	0.075
11 BOSQUE	6/18/1976	2034	F1	0	0	
12 BOSQUE	9/13/1977	1410	F1	0	0	
13 BOSQUE	4/30/1978	1920	F2	0	0	
14 BOSQUE	6/20/1980	2010	F2	100	4	0.227
15 BOSQUE	6/2/1987	1320	F0	10	0	
16 BOSQUE	9/17/1988	1415	F0	10	0	
17 BOSQUE	4/14/1990	350	F1	440	1	0.250
18 BOSQUE	4/27/1990	1527	F2	10	0	
19 BOSQUE	5/2/1990	2230	F2	10	0	
20 Iredell	4/26/1994	1720	F2	100	1	0.057
21 Meridian	4/26/1994	1806	F0	10	0	
22 Meridian	4/26/1994	1930	F0	10	0	
23 Morgan	4/26/1994	2043	F1	10	0	
24 Kopperl	10/21/1996	2:30 PM	F0	30	0	
25 Valley Mills	3/16/1998	4:53 PM	F0	0	0	

Table 2.3-209 (Sheet 2 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
26 Meridian	3/8/1999	9:37 AM	F0	0	0	
27 Laguna Park	5/12/2000	4:10 PM	F3	400	7	1.591
28 Kopperl	5/5/2001	3:05 PM	F0	75	2	0.085
29 Valley Mills	10/12/2001	7:54 PM	F1	75	2	0.085
30 Valley Mills	10/12/2001	8:00 PM	F0	50	2	0.057
Erath County, TX						
1 ERATH	2/12/1950	115	F1	233	2	0.265
2 ERATH	5/4/1960	1810	F2	33	0	
3 ERATH	5/4/1960	1820	F1	33	0	
4 ERATH	3/20/1962	930	F1	67	1	0.038
5 ERATH	5/30/1967	2020	F0	17	1	0.010
6 ERATH	4/28/1971	1650	F2	0	0	
7 ERATH	4/19/1976	1930	F2	300	11	1.875
8 ERATH	5/31/1976	1555	F2	33	4	0.075
9 ERATH	5/31/1976	1655	F1	33	0	
10 ERATH	4/24/1980	1730	F1	0	0	
11 ERATH	4/24/1980	1745	F0	0	0	
12 ERATH	6/20/1980	1920	F1	0	0	
13 ERATH	5/12/1982	1224	F0	7	0	
14 ERATH	5/12/1982	1410	F3	100	5	0.284
15 ERATH	5/12/1982	2007	F1	50	6	0.170
16 ERATH	6/15/1982	2030	F1	50	3	0.085
17 ERATH	6/20/1982	1525	F1	50	2	0.057
18 ERATH	2/26/1984	400	F1	30	0	
19 ERATH	4/18/1986	1900	F0	10	0	
20 ERATH	9/29/1988	50	F1	50	1	0.028

Table 2.3-209 (Sheet 3 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
21 ERATH	5/2/1989	1915	F1	10	0	
22 ERATH	4/25/1990	1727	F2	50	3	0.085
23 ERATH	4/25/1990	1857	F1	50	1	0.028
24 ERATH	4/25/1990	2114	F1	50	3	0.085
25 Hico	4/26/1994	1705	F0	10	0	
26 Alexander	4/26/1994	1950	F0	10	0	
27 Stephenville	5/7/1995	2020	F0	10	0	
28 Thurber	10/21/1996	10:10 AM	F0	30	0	
29 Morgan Mill	6/1/1999	6:00 PM	F0	20	0	
30 Dublin	6/1/1999	8:22 PM	F0	10	0	
31 Chalk Mtn	3/30/2002	4:20 PM	F0	30	0	
Hood County, TX						
1 HOOD	5/25/1957	1400	F0	33	1	0.019
2 HOOD	11/15/1960	1610	F0	167	0	
3 HOOD	9/14/1966	1800	F1	33	2	0.038
4 HOOD	4/28/1971	1730	F2	33	12	0.225
5 HOOD	8/8/1972	1750	F1	10	1	0.006
6 HOOD	4/19/1976	2028	F2	33	0	
7 HOOD	4/19/1976	2055	F2	33	0	
8 HOOD	5/9/1977	1400	F1	0	0	
9 HOOD	7/27/1977	1930	F2	250	1	0.142
10 HOOD	10/30/1979	835	F0	0	0	
11 HOOD	5/12/1982	1740	F1	73	7	0.290
12 HOOD	4/29/1985	1533	F0	10	0	
13 HOOD	5/4/1989	2110	F1	10	0	
14 HOOD	5/4/1989	2120	F2	500	4	1.136

Table 2.3-209 (Sheet 4 of 7) Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
15 HOOD	5/16/1989	1800	F1	73	1	0.041
16 HOOD	5/16/1989	1815	F1	73	1	0.041
17 HOOD	6/7/1989	614	F1	10	0	
18 HOOD	6/2/1991	928	F1	10	0	
19 Granbury	10/21/1996	7:00 AM	F1	10	0	
Johnson County, TX						
1 JOHNSON	6/16/1951	30	F2	20	15	0.170
2 JOHNSON	8/31/1956	1400	F1	100	1	0.057
3 JOHNSON	4/19/1957	400	F0	17	0	
4 JOHNSON	8/3/1958	425	F0	100	6	0.341
5 JOHNSON	10/3/1959	230	F1	100	1	0.057
6 JOHNSON	10/4/1959	230	F1	50	13	0.369
7 JOHNSON	5/20/1960	345	F1	300	1	0.170
8 JOHNSON	3/16/1961	1700	F1	200	3	0.341
9 JOHNSON	3/16/1961	1755	F2	33	1	0.019
10 JOHNSON	3/26/1961	1600	F3	50	1	0.028
11 JOHNSON	3/26/1961	1600	F2	17	0	
12 JOHNSON	6/8/1962	1500	F2	133	2	0.151
13 JOHNSON	6/28/1962	1800	F2	27	1	0.015
14 JOHNSON	9/7/1962	2040	F2	167	2	0.190
15 JOHNSON	8/8/1963	1500	F1	67	2	0.076
16 JOHNSON	9/6/1963	1430	F2	50	2	0.057
17 JOHNSON	11/19/1964	1	F0	50	1	0.028
18 JOHNSON	6/23/1965	2015	F2	17	0	
19 JOHNSON	5/13/1968	1217	F2	33	0	
20 JOHNSON	12/18/1968	1115	F1	13	0	

Table 2.3-209 (Sheet 5 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
21 JOHNSON	12/18/1968	1137	F0	100	0	
22 JOHNSON	10/26/1970	2100	F1	23	1	0.013
23 JOHNSON	2/18/1971	1645	F1	50	0	
24 JOHNSON	4/28/1971	1830	F1	20	1	0.011
25 JOHNSON	4/28/1971	1830	F2	20	1	0.011
26 JOHNSON	4/28/1971	1830	F1	20	1	0.011
27 JOHNSON	10/19/1971	1800	F2	50	8	0.227
28 JOHNSON	12/14/1971	1710	F2	17	0	
29 JOHNSON	12/14/1971	1715	F1	17	0	
30 JOHNSON	12/14/1971	1720	F1	17	0	
31 JOHNSON	4/23/1973	1700	F3	333	3	0.568
32 JOHNSON	5/6/1973	1915	F2	100	12	0.682
33 JOHNSON	11/24/1973	1315	F3	33	0	
34 JOHNSON	4/11/1974	415	F1	100	3	0.170
35 JOHNSON	4/7/1975	2230	F2	27	0	
36 JOHNSON	4/20/1976	20	F1	33	0	
37 JOHNSON	5/26/1976	1430	F1	50	0	
38 JOHNSON	5/26/1976	1445	F1	50	1	0.028
39 JOHNSON	5/26/1976	1512	F3	100	4	0.227
40 JOHNSON	5/26/1976	1525	F1	50	0	
41 JOHNSON	5/26/1976	1540	F2	100	0	
42 JOHNSON	5/26/1976	1617	F4	300	2	0.341
43 JOHNSON	9/2/1976	1735	F0	0	0	
44 JOHNSON	6/12/1977	1645	F0	0	0	
45 JOHNSON	9/13/1977	1200	F1	0	0	
46 JOHNSON	4/30/1978	1820	F2	33	2	0.038

Table 2.3-209 (Sheet 6 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
47 JOHNSON	4/30/1978	1850	F1	33	1	0.019
48 JOHNSON	5/3/1979	1000	F1	33	0	
49 JOHNSON	10/30/1979	915	F1	50	1	0.028
50 JOHNSON	10/13/1981	1145	F2	50	3	0.085
51 JOHNSON	10/13/1981	1150	F1	0	0	
52 JOHNSON	10/13/1981	1200	F1	50	6	0.170
53 JOHNSON	11/8/1981	1722	F1	73	2	0.083
54 JOHNSON	11/8/1981	1730	F0	0	0	
55 JOHNSON	4/27/1985	1712	F0	30	1	0.017
56 JOHNSON	4/27/1985	1814	F0	30	1	0.017
57 JOHNSON	5/16/1989	1838	F1	73	3	0.124
58 JOHNSON	4/12/1991	1745	F2	200	2	0.227
59 JOHNSON	4/12/1991	1805	F2	300	2	0.341
60 JOHNSON	4/12/1991	1851	F2	10	0	
61 JOHNSON	4/12/1991	1940	F0	10	0	
62 JOHNSON	4/12/1991	1955	F1	10	0	
63 Grandview	5/9/1993	1408	F0	10	0	
64 Lake Pat Cleburne	9/13/1993	610	F1	150	13	1.108
65 Keene	10/17/1993	2300	F0	10	0	
66 Godley	4/28/1994	2357	F0	10	0	
67 Alvarado	4/29/1994	23	F0	10	0	
68 Mansfield	4/29/1994	40	F0	10	0	
69 Cleburne	5/4/2001	7:42 PM	F1	50	0	
70 Alvarado	5/4/2001	8:45 PM	F1	50	3	0.085
71 Grandview	4/16/2002	5:56 PM	F0	70	1	0.040
72 Grandview	4/16/2002	6:07 PM	F0	40	1	0.023

Table 2.3-209 (Sheet 7 of 7)
Tornadoes In Surrounding Area

CP COL 2.3(1)

Location or County	Date	Time	Magnitude	Width (yards)	Length (mi)	Area (mi^2)
73 Grandview	4/16/2002	6:11 PM	F0	100	1	0.057
74 Grandview	4/16/2002	6:14 PM	F0	40	1	0.023
75 Alvarado	4/25/2005	3:25 PM	F0	15	0	
Somervell County, TX						
1 SOMERVELL	10/1/1988	1617	F0	10	0	
2 SOMERVELL	4/12/1991	1640	F2	100	2	0.114
3 SOMERVELL	4/12/1991	1655	F2	150	3	0.256

TORNADO MAGNITUDE
Bosque, Erath, Somervell, Hood, and Johnson Counties

Month	F0	F1	F2	F3	F4	Grand Total	%
Feb		3				3	1.9%
Mar	3	2	2	1		8	5.1%
Apr	19	14	18	1		52	32.9%
May	6	18	9	3	1	37	23.4%
Jun	4	6	5			15	9.5%
Jul			1			1	0.6%
Aug	1	4				5	3.2%
Sep	2	5	2			9	5.7%
Oct	8	8	2			18	11.4%
Nov	3	1		1		5	3.2%
Dec	1	3	1			5	3.2%
Total	47	64	40	6	1	158	100.0%
Percent	29.7%	40.5%	25.3%	3.8%	0.6%	100.0%	

NOTES:

1. Tornado data from all yr were used to calculate the annual frequencies.
2. Data recorded in the NOAA's National Environmental Satellite, Data, and Information Service (NEDSIS) - NCDC Storm Event database, January 1, 1950 through July 31, 2006, http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms

Table 2.3-210
Tornadoes in Surrounding Counties by Month

CP COL 2.3(1)

Month	Bosque (#)	Erath (#)	Hood (#)	Johnson (#)	Somervell (#)	All Five Areas (#)	Average per yr (#/yr)
Jan			1			1	0.02
Feb		2		1		3	0.05
Mar	2	2		4		8	0.14
Apr	11	10	4	25	2	52	0.92
May	6	10	7	14		37	0.65
Jun	3	5	2	5		15	0.27
Jul			1			1	0.02
Aug	1		1	3		5	0.09
Sep	2	1	1	5		9	0.16
Oct	5	1	2	9	1	18	0.32
Nov			1	4		5	0.09
Dec				5		5	0.09
Total	30	31	20	75	3	159	2.81
Percent	18.9%	19.5%	12.6%	47.2%	1.9%	100.0%	

NOTES:

1. Tornado data from all yr were used to calculate the annual frequencies.
2. Data recorded in the NOAA's National Environmental Satellite, Data, and Information Service (NEDSIS) - NCDC Storm Event database, January 1, 1950 through July 31, 2006, http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms

Table 2.3-211
Thunderstorms and High Wind Events

CP COL 2.3(1)

Month	Bosque (#)	Erath (#)	Hood (#)	Johnson (#)	Somervell (#)	All Five Areas (#)	Average per Yr (#/yr)
Jan	1	2	1	1		5	0.19
Feb		2	2	6		10	0.39
Mar	7	6	5	2	2	22	0.86
Apr	10	15	6	19	7	57	2.22
May	15	24	19	26	11	95	3.70
Jun	14	22	21	23	13	93	3.62
Jul	4	2	2	8	1	17	0.66
Aug	3	2	8	15	5	33	1.29
Sep	3	5	8	5	3	24	0.94
Oct	6	5	6	13	2	32	1.25
Nov	3		1	4	1	9	0.35
Dec	1	2	2	6	1	12	0.47
Total	67	87	81	128	46	409	15.73
Percent	16.4%	21.3%	19.8%	31.3%	11.2%	100%	

NOTES:

1. Storms listed at different sites in the same county on the same day were counted as separate events.
2. Data obtained for the period January 1, 1950 through July 31, 2006. Prior to 1981, the yearly storm averages were markedly less frequent, suggesting less thorough storm data collection. Consequently, the average/yr was based on 1981 through 7/31/2006 data
3. CPNPP site is in Somervell County. The other counties listed surround Somervell County.
4. Data recorded in the NOAA Storm Events Database, 1950 – 2005 http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms.

Table 2.3-212
Hail Storm Events

CP COL 2.3(1)

COUNTIES SURROUNDING SITE		
County	Number of Events	Percentage
Bosque	159	22.5%
Erath	198	28.0%
Somervell	54	7.6%
Hood	107	15.1%
Johnson	189	26.7%
Total	707	100%

Average number per yr = 12.3

NOTES:

1. Data from NOAA's Satellite & Information System - NCDC Storm Events Database, January 1, 1950 through March 31, 2007, http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms

2. For this table, each occurrence of hail was counted as an individual event, even if two counties recorded hail simultaneously.

Table 2.3-213
Mixing Height CPNPP Vicinity

CP COL 2.3(1)

	Morning		Afternoon	
	Mixing Height (m)	Wind Speed Averaged Through the Mixing Layer (m/sec)	Mixing Height (m)	Wind Speed Averaged Through the Mixing Layer (m/sec)
Winter	400	7	1050	7.8
Spring	500	8	1600	9
Summer	550	7	2000	6.8
Fall	450	6.5	1600	7
Annual	480	7	1600	7.5

Reference: Holzworth, G. C., "Mixing Heights, Wind Speeds, and Potential for Urban Air Pollution Throughout the Contiguous United States"

Table 2.3-214
Mixing Heights at Stephenville Texas

CP COL 2.3(1)

Season	Morning (m)	Afternoon (m)
Winter	509	1187
Spring	616	2076
Summer	366	1778
Fall	445	1383
Annual	484	1612

	Mixing Height Data	
Month	Avg. Morning (m)	Avg. Afternoon (m)
Jan	576	1195
Feb	491	1207
Mar	709	2154
Apr	422	2158
May	694	1830
Jun	425	1454
July	365	1825
Aug	306	2046
Sep	463	1583
Oct	367	1249
Nov	482	1191
Dec	437	1030

NOTES:

1. Season is selected per designated 3 month period, and as such seasons are not necessarily the same number of days. Furthermore, minor discrepancies between the annual value and the average per season may be present, due to the inconsistent length of period used.

2. Data are from the NCDC SCRAM Mixing Height Data collection for the period of 1984 – 1985 and 1987 – 1990 http://www.epa.gov/scram001/mixingheightdata.htm

Table 2.3-215
Mean Ventilation Rate by Month
Stephenville Texas

CP COL 2.3(1)

	Morning Ventilation Rate (m^2/s)	Afternoon Ventilation Rate (m^2/s)	Mean Ventilation Rate (m^2/s)
Jan	3364	6565	4965
Feb	3377	7219	5298
Mar	4332	10940	7636
Apr	2994	12391	7692
May	4771	9343	7057
Jun	2992	8611	5801
July	2210	9307	5759
Aug	1643	9496	5569
Sep	2775	8933	5854
Oct	2713	6856	4784
Nov	3475	6553	5014
Dec	2422	5794	4108

NOTES:

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

2. Data are from the NCDC SCRAM Mixing Height Data collection for the period of 1984 – 1985 and 1987 – 1990 http://www.epa.gov/scram001/mixingheightdata.htm

Table 2.3-216 (Sheet 1 of 5) Mixing Height CPNPP Area

CP COL 2.3(1)

	Wind Speed 1959 – 1998 (m/s)	Mixing Height 1961 – 1990 (meters agl)	Ventilation Index 1961 – 1990 (m^2/s)
January AM			
Min	3.2	340	1402 (Marginal)
Max	4.4	615	2374 (Fair)
Mean	3.9	485	1920 (Marginal)
January PM			
Min	3.1	680	2455 (Fair)
Max	4.1	1148	4330 (Good)
Mean	3.5	926	3307 (Fair)
February AM			
Min	2.9	416	1595 (Marginal)
Max	4.5	662	2832 (Fair)
Mean	3.8	530	2105 (Marginal)
February PM			
Min	2.6	828	2279 (Marginal)
Max	4.0	1552	6075 (Good)
Mean	3.4	1131	3943 (Good)
March AM			
Min	3.4	423	1710 (Marginal)
Max	4.5	719	2903 (Fair)
Mean	4.0	589	2475 (Fair)

Table 2.3-216 (Sheet 2 of 5) Mixing Height CPNPP Area

CP COL 2.3(1)

	Wind Speed 1959 – 1998 (m/s)	Mixing Height 1961 – 1990 (meters agl)	Ventilation Index 1961 – 1990 (m^2/s)
March PM			
Min	3.1	1168	4244 (Good)
Max	4.0	1774	6276 (Good)
Mean	3.6	1399	5158 (Good)
April AM			
Min	3.6	425	1717 (Marginal)
Max	4.5	818	3640 (Good)
Mean	4.0	612	2544 (Fair)
April PM			
Min	3.1	1107	3708 (Good)
Max	4.1	2011	7721 (Good)
Mean	3.6	1527	5702 (Good)
May AM			
Min	3.2	440	1406 (Marginal)
Max	4.7	856	3478 (Fair)
Mean	3.9	647	2592 (Fair)
May PM			
Min	2.7	1193	4265 (Good)
Max	4.5	1936	7718 (Good)
Mean	3.5	1579	5646 (Good)

Table 2.3-216 (Sheet 3 of 5)
Mixing Height
CPNPP Area

CP COL 2.3(1)

	Wind Speed 1959 – 1998 (m/s)	Mixing Height 1961 – 1990 (meters agl)	Ventilation Index 1961 – 1990 (m^2/s)
June AM			
Min	3.1	558	1704 (Marginal)
Max	4.7	828	3243 (Fair)
Mean	3.7	688	2604 (Fair)
June PM			
Min	2.7	1373	4721 (Good)
Max	4.1	2199	8678 (Good)
Mean	3.4	1746	6084 (Good)
July AM			
Min	3.0	496	1756 (Marginal)
Max	4.6	751	2717 (Fair)
Mean	3.5	658	2292 (Marginal)
July PM			
Min	2.9	1621	5126 (Good)
Max	4.3	2605	9684 (Good)
Mean	3.6	2009	7230 (Good)
August AM			
Min	2.6	438	1318 (Marginal)
Max	3.9	788	2956 (Fair)
Mean	3.3	619	2085 (Marginal)

Table 2.3-216 (Sheet 4 of 5) Mixing Height CPNPP Area

CP COL 2.3(1)

	Wind Speed 1959 – 1998 (m/s)	Mixing Height 1961 – 1990 (meters agl)	Ventilation Index 1961 – 1990 (m^2/s)
August PM			
Min	2.6	1636	4677 (Good)
Max	3.9	2486	9009 (Good)
Mean	3.4	2030	7086 (Good)
September AM			
Min	2.7	446	1485 (Marginal)
Max	4.2	788	3241 (Fair)
Mean	3.3	611	2090 (Marginal)
September PM			
Min	2.6	1189	3318 (Fair)
Max	4.1	2006	6778 (Good)
Mean	3.2	1644	5278 (Good)
October AM			
Min	3.2	373	1346 (Marginal)
Max	4.0	620	2557 (Fair)
Mean	3.6	493	1867 (Marginal)
October PM			
Min	2.9	1118	3528 (Good)
Max	3.7	2182	7091 (Good)
Mean	3.3	1414	4665 (Good)

Table 2.3-216 (Sheet 5 of 5) Mixing Height CPNPP Area

CP COL 2.3(1)

	Wind Speed 1959 – 1998 (m/s)	Mixing Height 1961 – 1990 (meters agl)	Ventilation Index 1961 – 1990 (m^2/s)
November AM			
Min	3.2	426	1515 (Marginal)
Max	4.4	648	2666 (Fair)
Mean	3.9	519	2086 (Marginal)
November PM			
Min	2.9	830	2621 (Fair)
Max	4.2	1435	5883 (Good)
Mean	3.5	1111	3934 (Good)
December AM			
Min	3.2	390	1548 (Marginal)
Max	4.5	600	2555 (Fair)
Mean	3.9	483	1941 (Marginal)
December PM			
Min	2.9	795	2463 (Marginal)
Max	4.0	1134	4277 (Good)
Mean	3.5	929	3314 (Fair)

Notes:

1. agl is aboveground level.
2. Ventilation Index is the product of the mixing height and the wind speed averaged through the mixing depth.
3. Data from the Ventilation Climate Information System, USDOI-USDA Joint Fire Science Program. http://web.airfire.org/vcis/, accessed September 10, 2007.

Table 2.3-217
Point Precipitation Recurrence Intervals for Region

CP COL 2.3(1)

	Recurrence Intervals (Yr)						
Duration	1	2	5	10	25	50	100
5 minutes	-	0.5	0.6	0.6	0.7	0.9	0.9
10 minutes	-	0.8	1.0	1.1	1.2	1.5	1.5
15 minutes	-	1.1	1.3	1.4	1.6	1.9	1.9
30 minutes	1.2	1.5	1.8	2.0	2.4	2.9	2.9
1 hour	1.5	1.9	2.4	2.7	3.2	4.0	4.0
2 hours	1.8	2.2	3.0	3.5	4.3	4.6	5.1
3 hours	2.0	2.5	3.3	3.9	4.5	5.2	5.7
6 hours	2.4	2.9	3.9	4.7	5.3	6.2	6.9
12 hours	2.8	3.4	4.7	5.5	6.4	7.4	8.3
24 hours	3.2	3.9	5.4	6.2	7.5	8.5	9.5
2 days	-	4.5	6.0	7.2	8.5	9.6	11.0
4 days	-	5.3	7.0	8.0	9.8	11.0	12.5
7 days	-	6.0	8.0	9.5	11.2	12.8	14.0
10 days	-	6.7	8.9	10.3	12.3	14.0	15.8

Notes:

1. 5 minute to 60 minute data based on spatial interpolation of isopluvials given in NOAA Technical Memorandum NWS Hydro-35, “Five- to 60-minute Precipitation Frequency for the Eastern and Central United States”, June 1977.

2. 2 hour through 24 hour data based on spatial interpolation of isopluvials given in National Weather Service Technical Paper No. 40, “Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 Hours and Return Periods from 1 to 100 yr”, U.S. Department of Commerce, May 1961.

3. 2 day through 10 day data based on interpolation of isopluvials given in National Weather Service Technical Paper No. 49, “Two- to Ten-day Precipitation for Return Periods of 2 to 100 yr in the Contiguous Unites States, U.S. Department of Commerce, 1964.

Table 2.3-218 (Sheet 1 of 3)
Ice Storms

CP COL 2.3(1)

Bosque, Erath, Somervell, Hill, Hood, Johnson, and Dallas Counties

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
Bosque County, Texas						
2/9/1994	0	Ice Storm	0	0	50.0M	0
11/24/1996	2:00 PM	Winter Storm	0	0	0	0
1/6/1997	12:00 PM	Heavy Snow	0	0	0	0
12/22/1998	12:00 AM	Ice Storm	6	0	0	0
1/25/2000	12:00 AM	Winter Storm	4	0	0	0
12/12/2000	6:00 PM	Winter Storm	0	0	0	0
12/25/2000	12:00 AM	Winter Storm	0	0	0	0
12/31/2000	12:00 AM	Winter Storm	0	0	0	0
1/1/2001	12:00 AM	Heavy Snow	0	0	0	0
11/27/2001	12:30 PM	Ice Storm	0	0	0	0
2/24/2003	11:20 AM	Winter Storm	0	0	15.0M	0
12/22/2004	12:01 AM	Winter Weather/mix	0	0	0	0
2/18/2006	3:30 AM	Winter Weather/mix	0	0	0	0
1/17/2007	3:00 AM	Winter Weather	0	0	105K	0
Erath County, Texas						
2/9/1994	0	Ice Storm	0	0	50.0M	0
11/24/1996	2:00 PM	Winter Storm	0	0	0	0
1/6/1997	12:00 PM	Heavy Snow	0	0	0	0
12/22/1998	12:00 AM	Ice Storm	6	0	0	0
1/25/2000	12:00 AM	Winter Storm	4	0	0	0
12/12/2000	6:00 PM	Winter Storm	0	0	0	0
12/25/2000	12:00 AM	Winter Storm	0	0	0	0
12/31/2000	12:00 AM	Winter Storm	0	0	0	0
1/1/2001	12:00 AM	Heavy Snow	0	0	0	0
1/18/2001	12:00 AM	Winter Storm	0	0	0	0
11/27/2001	12:30 PM	Ice Storm	0	0	0	0
2/5/2002	5:00 AM	Winter Storm	0	0	0	0
3/2/2002	2:15 AM	Winter Storm	0	0	0	0
2/24/2003	11:20 AM	Winter Storm	0	0	15.0M	0
12/22/2004	12:01 AM	Winter Weather/mix	0	0	0	0
12/7/2005	7:00 AM	Winter Storm	0	0	0	0
1/13/2007	5:00 AM	Ice Storm	0	5	715K	0
1/17/2007	3:00 AM	Winter Weather	0	0	105K	0

Table 2.3-218 (Sheet 2 of 3)
Ice Storms

CP COL 2.3(1)

Bosque, Erath, Somervell, Hill, Hood, Johnson, and Dallas Counties

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
Hood County, Texas						
2/9/1994	0	Ice Storm	0	0	50.0M	0
11/24/1996	2:00 PM	Winter Storm	0	0	0	0
1/6/1997	9:50 AM	Winter Storm	0	0	0	0
12/22/1998	12:00 AM	Ice Storm	6	0	0	0
1/25/2000	12:00 AM	Winter Storm	4	0	0	0
12/12/2000	6:00 PM	Winter Storm	0	0	0	0
12/25/2000	12:00 AM	Winter Storm	0	0	0	0
12/31/2000	12:00 AM	Winter Storm	0	0	0	0
1/1/2001	12:00 AM	Heavy Snow	0	0	0	0
1/18/2001	12:00 AM	Winter Storm	0	0	0	0
11/27/2001	12:30 PM	Ice Storm	0	0	0	0
3/2/2002	2:15 AM	Winter Storm	0	0	0	0
2/24/2003	11:20 AM	Winter Storm	0	0	15.0M	0
12/22/2004	12:01 AM	Winter Weather/mix	0	0	0	0
12/7/2005	7:00 AM	Winter Storm	0	0	0	0
2/18/2006	3:30 AM	Winter Weather/mix	0	0	0	0
1/13/2007	5:00 AM	Ice Storm	0	5	715K	0
1/17/2007	3:00 AM	Winter Weather	0	0	105K	0
Johnson County, Texas						
2/9/1994	0	Ice Storm	0	0	50.0M	0
11/24/1996	2:00 PM	Winter Storm	0	0	0	0
1/6/1997	12:00 PM	Heavy Snow	0	0	0	0
12/22/1998	12:00 AM	Ice Storm	6	0	0	0
1/25/2000	12:00 AM	Winter Storm	4	0	0	0
12/12/2000	6:00 PM	Winter Storm	0	0	0	0
12/25/2000	12:00 AM	Winter Storm	0	0	0	0
12/31/2000	12:00 AM	Winter Storm	0	0	0	0
1/1/2001	12:00 AM	Heavy Snow	0	0	0	0
11/27/2001	12:30 PM	Ice Storm	0	0	0	0
2/5/2002	5:00 AM	Winter Storm	0	0	0	0
2/24/2003	11:20 AM	Winter Storm	0	0	15.0M	0
12/22/2004	12:01 AM	Winter Weather/mix	0	0	0	0
12/7/2005	7:00 AM	Winter Storm	0	0	0	0
1/13/2007	5:00 AM	Ice Storm	0	5	715K	0

Table 2.3-218 (Sheet 3 of 3)
Ice Storms

CP COL 2.3(1)

Bosque, Erath, Somervell, Hill, Hood, Johnson, and Dallas Counties

Date	Time	Type	Deaths	Injuries	Property Damage	Crop Damage
Somervell County, Texas						
2/9/1994	0	Ice Storm	0	0	50.0M	0
11/24/1996	2:00 PM	Winter Storm	0	0	0	0
1/6/1997	9:50 AM	Winter Storm	0	0	0	0
12/22/1998	12:00 AM	Ice Storm	6	0	0	0
1/25/2000	12:00 AM	Winter Storm	4	0	0	0
12/12/2000	6:00 PM	Winter Storm	0	0	0	0
12/25/2000	12:00 AM	Winter Storm	0	0	0	0
12/31/2000	12:00 AM	Winter Storm	0	0	0	0
1/1/2001	12:00 AM	Heavy Snow	0	0	0	0
11/27/2001	12:30 PM	Ice Storm	0	0	0	0
3/2/2002	2:15 AM	Winter Storm	0	0	0	0
2/24/2003	11:20 AM	Winter Storm	0	0	15.0M	0
12/22/2004	12:01 AM	Winter Weather/mix	0	0	0	0
12/7/2005	7:00 AM	Winter Storm	0	0	0	0
2/18/2006	3:30 AM	Winter Weather/mix	0	0	0	0
1/13/2007	5:00 AM	Ice Storm	0	5	715K	0

County Affected	Number of Ice Storms 01/01/1994 to 03/31/2007	#/yr	Return Period (yr)
Bosque	14	1.06	0.9
Erath	18	1.36	0.7
Hood	18	1.36	0.7
Johnson	15	1.13	0.9
Somervell	16	1.21	0.8

NOTES:

1. CPNPP site is in Somervell County. The other counties surround Somervell County.
2. A single storm which affects more than one county is counted as an individual storm for each county.
3. Number of Ice storms, #/yr and Return Period evaluated for the period of January 1, 1994 to March 31, 2007 due to the lack of data before 1994.
4. Data recorded in the NOAA Storm Events Database, January 1, 1950 and March 31, 2007. http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwevent~storms.

Table 2.3-219
Local Climate Averages

CP COL 2.3(1)

Average number of days with	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Yr
Dublin, Texas													
Temperature >= 90	0	0	0	2	5	17	24	24	14	3	0	0	89
Temperature >= 100	0	0	0	0	0	1	4	6	1	0	0	0	13
Temperature <= 32	14	9	5	0	0	0	0	0	0	0	4	12	45
Precipitation	5	6	6	7	8	5	4	4	5	6	5	5	66
Snow	1	0	0	0	0	0	0	0	0	0	0	0	2
Weatherford, Texas													
Temperature >= 90	0	0	0	1	5	19	27	26	15	3	0	0	97
Temperature >= 100	0	0	0	0	0	1	6	7	1	0	0	0	15
Temperature <= 32	18	12	6	1	0	0	0	0	0	0	6	14	57
Precipitation	6	6	6	7	8	6	5	5	6	6	6	6	72
Snow	0	0	0	0	0	0	0	0	0	0	0	0	1

Notes:

1. United States Historical Climatology Network, Dublin, Texas, Site 412598 and Weatherford, Texas, Site 419532, Data 1902 through 2004, http://cdiac.ornl.gov/cgi-bin/broker?_PROGRAM=prog.climsite.sas&_SERVICE=default&id=412598 http://cdiac.ornl.gov/cgi-bin/broker?_PROGRAM=prog.climsite.sas&_SERVICE=default&id=419532#gplot_clim_year

Table 2.3-220
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
January 1997 – 2006

CP COL 2.3(1)

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.62%	3.44%	5.32%	2.91%	0.95%	0.27%	0.01%	13.51%	10.97
N-NE	0.31%	1.45%	2.46%	1.34%	0.39%	0.01%	0.00%	5.97%	10.35
NE	0.35%	0.77%	0.72%	0.16%	0.05%	0.00%	0.00%	2.06%	7.69
E-NE	0.34%	0.61%	0.69%	0.15%	0.01%	0.00%	0.00%	1.80%	7.85
E	0.58%	1.35%	0.77%	0.20%	0.00%	0.00%	0.00%	2.91%	7.27
E-SE	0.35%	1.26%	1.20%	0.09%	0.00%	0.00%	0.00%	2.91%	7.46
SE	0.37%	1.79%	1.91%	0.42%	0.04%	0.00%	0.00%	4.52%	8.33
S-SE	0.39%	1.83%	3.34%	1.45%	0.22%	0.01%	0.00%	7.24%	10.22
S	0.50%	2.64%	7.79%	6.71%	2.96%	0.88%	0.16%	21.64%	13.26
S-SW	0.18%	1.23%	2.42%	1.42%	0.70%	0.18%	0.03%	6.15%	11.87
SW	0.14%	0.69%	1.16%	0.39%	0.19%	0.01%	0.00%	2.58%	10.29
W-SW	0.11%	0.57%	0.68%	0.43%	0.15%	0.09%	0.00%	2.03%	11.06
W	0.09%	0.30%	0.87%	0.61%	0.19%	0.05%	0.00%	2.11%	11.76
W-NW	0.14%	0.50%	1.27%	0.87%	0.31%	0.23%	0.04%	3.35%	12.92
NW	0.16%	0.89%	2.11%	1.57%	0.87%	0.37%	0.07%	6.03%	13.07
N-NW	0.15%	1.57%	3.60%	2.14%	1.28%	0.54%	0.19%	9.47%	13.18
CALM	5.06%	0.66%	0.00%	0.00%	0.00%	0.00%	0.00%	5.72%	1.00
Total	9.83%	21.53%	36.30%	20.86%	8.32%	2.65%	0.50%	100.00%	10.74

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-221
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
February 1997 – 2006

CP COL 2.3(1)

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.79%	2.99%	5.69%	3.05%	0.95%	0.12%	0.06%	13.65%	10.76
N-NE	0.30%	1.11%	2.62%	0.96%	0.12%	0.04%	0.01%	5.17%	9.96
NE	0.31%	1.21%	1.01%	0.09%	0.01%	0.00%	0.00%	2.64%	7.68
E-NE	0.37%	0.99%	0.77%	0.10%	0.00%	0.00%	0.00%	2.24%	7.57
E	0.70%	1.94%	1.60%	0.04%	0.03%	0.00%	0.00%	4.31%	7.08
E-SE	0.58%	1.53%	1.59%	0.36%	0.04%	0.00%	0.00%	4.09%	8.37
SE	0.37%	1.99%	2.44%	0.71%	0.19%	0.10%	0.00%	5.81%	9.55
S-SE	0.36%	1.59%	3.41%	1.70%	0.58%	0.09%	0.01%	7.73%	11.07
S	0.47%	1.99%	6.05%	5.54%	3.05%	1.47%	0.31%	18.88%	14.07
S-SW	0.18%	0.90%	1.38%	0.77%	0.41%	0.10%	0.00%	3.75%	11.27
SW	0.12%	0.73%	0.89%	0.31%	0.10%	0.03%	0.00%	2.18%	9.72
W-SW	0.15%	0.40%	0.46%	0.19%	0.09%	0.10%	0.00%	1.39%	10.57
W	0.16%	0.50%	0.95%	0.43%	0.22%	0.13%	0.04%	2.44%	12.12
W-NW	0.16%	0.53%	1.01%	0.71%	0.34%	0.22%	0.04%	3.02%	13.18
NW	0.12%	0.79%	2.64%	1.59%	0.96%	0.46%	0.07%	6.62%	13.60
N-NW	0.37%	1.82%	3.69%	1.88%	1.14%	0.50%	0.10%	9.51%	12.28
CALM	5.66%	0.90%	0.00%	0.00%	0.00%	0.00%	0.00%	6.56%	1.16
Total	11.16	21.91%	36.18%	18.45%	8.25%	3.38%	0.67%	100.00%	10.72

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-222
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
March 1997 – 2006

CP COL 2.3(1)

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.30%	2.65%	4.59%	2.49%	1.07%	0.36%	0.04%	11.50%	11.62
N-NE	0.26%	1.42%	3.04%	1.38%	0.39%	0.11%	0.01%	6.61%	11.00
NE	0.20%	1.03%	1.36%	0.24%	0.09%	0.00%	0.00%	2.93%	8.97
E-NE	0.34%	0.99%	1.20%	0.32%	0.04%	0.00%	0.00%	2.89%	8.71
E	0.45%	1.55%	2.39%	0.50%	0.04%	0.00%	0.00%	4.93%	8.40
E-SE	0.39%	1.49%	1.84%	0.53%	0.04%	0.01%	0.00%	4.30%	8.53
SE	0.39%	1.70%	2.22%	1.09%	0.23%	0.01%	0.01%	5.66%	9.73
S-SE	0.30%	1.78%	3.97%	2.30%	0.96%	0.27%	0.04%	9.62%	11.69
S	0.42%	1.74%	6.01%	6.00%	4.11%	2.28%	0.43%	21.00%	15.18
S-SW	0.15%	0.93%	1.53%	0.72%	0.45%	0.20%	0.01%	3.99%	11.88
SW	0.03%	0.46%	0.74%	0.16%	0.20%	0.04%	0.00%	1.63%	11.18
W-SW	0.07%	0.34%	0.65%	0.27%	0.04%	0.05%	0.00%	1.42%	10.46
W	0.12%	0.26%	0.74%	0.41%	0.26%	0.11%	0.03%	1.92%	12.56
W-NW	0.04%	0.31%	0.78%	0.68%	0.42%	0.24%	0.07%	2.54%	14.12
NW	0.09%	0.78%	1.66%	1.00%	1.08%	0.49%	0.19%	5.30%	14.38
N-NW	0.14%	1.15%	3.08%	1.65%	1.46%	0.57%	0.03%	8.07%	13.58
CALM	4.59%	1.11%	0.00%	0.00%	0.00%	0.00%	0.00%	5.70%	1.43
Total	8.27%	19.69%	35.82%	19.73%	10.88	4.76%	0.86%	100.00%	11.60

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-223
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
April 1997 – 2006

CP COL 2.3(1)

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.29%	1.96%	3.44%	1.91%	0.81%	0.21%	0.01%	8.63%	17.82
N-NE	0.19%	0.79%	1.57%	0.70%	0.31%	0.11%	0.03%	3.70%	11.33
NE	0.25%	1.04%	0.67%	0.24%	0.06%	0.00%	0.00%	2.25%	8.19
E-NE	0.11%	0.78%	0.81%	0.36%	0.11%	0.01%	0.00%	2.18%	9.87
E	0.45%	1.24%	1.53%	0.38%	0.04%	0.01%	0.00%	3.65%	8.58
E-SE	0.32%	1.14%	1.98%	0.50%	0.04%	0.00%	0.00%	3.98%	9.13
SE	0.42%	1.73%	3.49%	1.10%	0.36%	0.00%	0.00%	7.10%	10.18
S-SE	0.18%	1.41%	4.97%	3.83%	1.29%	0.29%	0.03%	12.00%	12.98
S	0.29%	1.64%	7.36%	9.17%	6.47%	2.84%	0.42%	28.19%	16.00
S-SW	0.13%	0.90%	1.73%	0.74%	0.47%	0.15%	0.00%	4.12%	11.95
SW	0.06%	0.67%	0.90%	0.33%	0.18%	0.06%	0.00%	2.20%	10.72
W-SW	0.03%	0.26%	0.64%	0.28%	0.11%	0.07%	0.00%	1.39%	11.67
W	0.10%	0.42%	0.75%	0.38%	0.25%	0.15%	0.01%	2.06%	12.28
W-NW	0.04%	0.31%	0.99%	0.43%	0.36%	0.19%	0.03%	2.35%	13.92
NW	0.13%	0.50%	2.10%	1.28%	0.64%	0.29%	0.07%	5.01%	13.35
N-NW	0.25%	1.21%	2.17%	1.35%	0.71%	0.53%	0.06%	6.28%	12.67
CALM	4.19%	0.72%	0.00%	0.00%	0.00%	0.00%	0.00%	4.91%	1.13
Total	7.42%	16.73%	35.10%	22.96%	12.22%	4.93%	0.65%	100.00%	12.74

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-224
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
May 1997 – 2006

CP COL 2.3(1)

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.46%	1.95%	3.33%	1.26%	0.34%	0.05%	0.00%	7.39%	9.95
N-NE	0.17%	1.59%	1.81%	0.91%	0.13%	0.04%	0.00%	4.66%	10.07
NE	0.35%	1.53%	1.14%	0.15%	0.03%	0.00%	0.00%	3.20%	7.69
E-NE	0.43%	1.13%	0.93%	0.13%	0.01%	0.01%	0.00%	2.65%	7.40
E	0.59%	1.92%	1.69%	0.35%	0.01%	0.00%	0.00%	4.57%	7.64
E-SE	0.27%	1.51%	1.86%	0.35%	0.05%	0.01%	0.00%	4.05%	8.68
SE	0.36%	1.69%	3.68%	1.47%	0.17%	0.04%	0.00%	7.42%	10.25
S-SE	0.24%	1.71%	6.34%	5.07%	1.73%	0.17%	0.01%	15.28%	12.81
S	0.35%	2.30%	9.52%	11.29%	5.82%	1.37%	0.16%	30.81%	14.59
S-SW	0.01%	0.70%	1.87%	1.55%	0.39%	0.12%	0.01%	4.65%	12.68
SW	0.04%	0.51%	0.74%	0.20%	0.15%	0.00%	0.00%	1.64%	10.43
W-SW	0.05%	0.31%	0.54%	0.16%	0.07%	0.01%	0.00%	1.14%	9.78
W	0.07%	0.30%	0.44%	0.19%	0.09%	0.00%	0.01%	1.10%	10.50
W-NW	0.00%	0.12%	0.35%	0.15%	0.03%	0.03%	0.01%	0.69%	12.40
NW	0.05%	0.47%	0.78%	0.52%	0.16%	0.07%	0.00%	2.06%	11.77
N-NW	0.17%	0.85%	1.26%	0.67%	0.31%	0.11%	0.03%	3.40%	10.93
CALM	4.15%	1.13%	0.00%	0.00%	0.00%	0.00%	0.00%	5.28%	1.64
Total	7.78%	19.71%	36.30%	24.43%	9.50%	2.04%	0.24%	100.00%	11.24

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-225
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
June 1997 – 2006

CP COL 2.3(1)

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.38%	1.95%	2.33%	0.91%	0.10%	0.04%	0.01%	5.71%	9.35
N-NE	0.24%	1.30%	1.99%	0.64%	0.18%	0.01%	0.00%	4.36%	9.31
NE	0.46%	0.92%	0.92%	0.22%	0.06%	0.00%	0.00%	2.58%	7.81
E-NE	0.38%	0.99%	1.03%	0.20%	0.01%	0.00%	0.00%	2.61%	7.80
E	0.50%	2.58%	1.48%	0.22%	0.03%	0.00%	0.00%	4.81%	7.23
E-SE	0.60%	2.91%	2.38%	0.61%	0.01%	0.01%	0.00%	6.54%	8.06
SE	0.78%	3.15%	5.56%	1.73%	0.21%	0.04%	0.01%	11.48%	9.50
S-SE	0.47%	2.91%	7.26%	4.15%	0.91%	0.04%	0.01%	15.76%	11.21
S	0.40%	2.63%	10.10%	9.41%	3.64%	1.02%	0.06%	27.26%	13.49
S-SW	0.18%	0.78%	1.92%	1.74%	0.67%	0.11%	0.00%	5.41%	12.24
SW	0.07%	0.45%	0.43%	0.22%	0.08%	0.01%	0.00%	1.27%	9.80
W-SW	0.04%	0.20%	0.17%	0.06%	0.01%	0.00%	0.00%	0.47%	8.71
W	0.04%	0.24%	0.26%	0.08%	0.01%	0.01%	0.01%	0.67%	10.31
W-NW	0.03%	0.10%	0.17%	0.04%	0.01%	0.01%	0.03%	0.39%	11.61
NW	0.07%	0.26%	0.47%	0.15%	0.03%	0.03%	0.00%	1.02%	10.34
N-NW	0.14%	0.96%	0.74%	0.15%	0.07%	0.03%	0.01%	2.10%	9.03
CALM	5.62%	1.94%	0.00%	0.00%	0.00%	0.00%	0.00%	7.55%	2.03
Total	10.40%	24.26%	37.23%	20.54%	6.03%	1.38%	0.15%	100.00%	10.23

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-226
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
July 1997 – 2006

CP COL 2.3(1)

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.27%	1.05%	1.41%	0.36%	0.05%	0.03%	0.00%	3.17%	8.96
N-NE	0.20%	0.62%	1.08%	0.27%	0.05%	0.01%	0.00%	2.23%	9.35
NE	0.22%	0.60%	0.59%	0.13%	0.01%	0.00%	0.01%	1.57%	8.56
E-NE	0.23%	0.69%	0.67%	0.17%	0.01%	0.00%	0.00%	1.77%	8.06
E	0.51%	1.65%	1.38%	0.13%	0.01%	0.00%	0.00%	3.70%	7.35
E-SE	0.47%	1.67%	2.47%	0.30%	0.05%	0.00%	0.01%	4.97%	8.54
SE	0.63%	2.10%	4.85%	0.95%	0.03%	0.00%	0.01%	8.58%	9.14
S-SE	0.43%	2.92%	8.44%	2.86%	0.36%	0.05%	0.01%	15.08%	10.31
S	0.46%	4.78%	19.18%	7.06%	1.32%	0.16%	0.00%	32.96%	10.98
S-SW	0.19%	1.87%	6.57%	2.50%	0.27%	0.03%	0.00%	11.42%	10.85
SW	0.05%	0.59%	1.73%	0.40%	0.01%	0.00%	0.00%	2.80%	10.09
W-SW	0.09%	0.40%	0.67%	0.09%	0.00%	0.01%	0.00%	1.28%	8.87
W	0.08%	0.31%	0.35%	0.05%	0.01%	0.00%	0.00%	0.81%	8.21
W-NW	0.05%	0.05%	0.07%	0.03%	0.00%	0.03%	0.00%	0.23%	9.95
NW	0.01%	0.19%	0.16%	0.05%	0.00%	0.00%	0.00%	0.42%	8.66
N-NW	0.11%	0.48%	0.70%	0.13%	0.04%	0.00%	0.00%	1.47%	8.55
CALM	5.04%	2.51%	0.00%	0.00%	0.00%	0.00%	0.00%	7.55%	2.52
Total	9.05%	22.49%	50.34%	15.51%	2.24%	0.32%	0.05%	100.00%	9.45

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-227
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
August 1997 – 2006

CP COL 2.3(1)

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.51%	1.73%	1.41%	0.27%	0.07%	0.00%	0.00%	3.99%	7.69
N-NE	0.30%	0.95%	1.16%	0.16%	0.03%	0.01%	0.00%	2.61%	8.08
NE	0.35%	0.77%	0.69%	0.13%	0.03%	0.00%	0.00%	1.96%	7.64
E-NE	0.35%	1.02%	1.24%	0.24%	0.01%	0.00%	0.00%	2.86%	8.28
E	0.63%	2.82%	2.58%	0.16%	0.03%	0.00%	0.00%	6.22%	7.40
E-SE	0.75%	3.14%	3.33%	0.15%	0.04%	0.00%	0.01%	7.43%	7.53
SE	0.69%	3.66%	4.07%	0.60%	0.01%	0.01%	0.00%	9.04%	8.05
S-SE	0.55%	3.31%	6.69%	1.40%	0.13%	0.00%	0.00%	12.08%	9.36
S	0.71%	4.89%	13.34%	5.83%	1.25%	0.09%	0.01%	26.14%	10.87
S-SW	0.16%	1.88%	4.73%	1.32%	0.30%	0.00%	0.00%	8.39%	10.40
SW	0.15%	0.91%	1.55%	0.17%	0.04%	0.00%	0.00%	2.82%	8.93
W-SW	0.12%	0.50%	0.50%	0.05%	0.01%	0.01%	0.00%	1.20%	8.04
W	0.13%	0.43%	0.54%	0.07%	0.03%	0.01%	0.00%	1.21%	8.56
W-NW	0.08%	0.20%	0.24%	0.01%	0.00%	0.00%	0.00%	0.54%	7.73
NW	0.12%	0.38%	0.27%	0.03%	0.03%	0.01%	0.00%	0.83%	8.33
N-NW	0.30%	0.97%	0.58%	0.05%	0.08%	0.03%	0.01%	2.02%	8.24
CALM	7.39%	3.27%	0.00%	0.00%	0.00%	0.00%	0.00%	10.66%	2.17
Total	13.29%	30.83%	42.91%	10.66%	2.08%	0.19%	0.04%	100.00%	8.39

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Greenville/Spartanburg International Airport, Station No. 03927.
3. Period of Record – 10 yr (199 7– 2006).

Table 2.3-228
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
September 1997 – 2006

CP COL 2.3(1)

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.61%	3.19%	3.52%	1.76%	0.53%	0.07%	0.00%	9.68%	10.02
N-NE	0.27%	1.63%	2.25%	0.92%	0.38%	0.04%	0.01%	5.50%	10.27
NE	0.43%	1.48%	1.51%	0.17%	0.08%	0.03%	0.00%	3.70%	8.03
E-NE	0.52%	1.40%	1.46%	0.31%	0.10%	0.06%	0.00%	3.84%	8.44
E	0.93%	3.56%	2.96%	0.40%	0.07%	0.00%	0.00%	7.92%	7.70
E-SE	0.86%	3.25%	2.99%	0.33%	0.03%	0.00%	0.00%	7.46%	7.71
SE	1.07%	3.98%	3.24%	0.74%	0.06%	0.01%	0.00%	9.10%	7.88
S-SE	0.60%	2.39%	5.04%	2.08%	0.29%	0.00%	0.00%	10.39%	9.89
S	0.71%	3.32%	6.92%	3.26%	0.98%	0.33%	0.03%	15.56%	10.65
S-SW	0.25%	1.41%	1.70%	0.56%	0.11%	0.01%	0.00%	4.05%	8.79
SW	0.08%	0.53%	0.39%	0.10%	0.00%	0.00%	0.00%	1.10%	7.56
W-SW	0.06%	0.40%	0.29%	0.01%	0.00%	0.00%	0.00%	0.77%	7.23
W	0.04%	0.32%	0.28%	0.10%	0.00%	0.01%	0.00%	0.75%	9.11
W-NW	0.07%	0.36%	0.38%	0.10%	0.07%	0.00%	0.00%	0.98%	9.39
NW	0.18%	0.75%	0.54%	0.28%	0.07%	0.00%	0.00%	1.83%	9.06
N-NW	0.18%	1.84%	1.37%	0.80%	0.32%	0.10%	0.03%	4.63%	10.57
CALM	9.46%	3.29%	0.00%	0.00%	0.00%	0.00%	0.00%	12.75%	1.89
Total	16.34%	33.11%	34.82%	11.91%	3.08%	0.67%	0.07%	100.00%	8.24

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-229
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
October 1997 – 2006

CP COL 2.3(1)

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.46%	2.85%	4.22%	2.14%	0.67%	0.17%	0.00%	10.51%	10.67
N-NE	0.34%	1.48%	2.04%	0.91%	0.20%	0.01%	0.00%	4.99%	9.87
NE	0.27%	1.21%	0.85%	0.15%	0.00%	0.00%	0.00%	2.47%	7.18
E-NE	0.35%	1.16%	0.71%	0.01%	0.00%	0.00%	0.00%	2.23%	6.74
E	0.62%	2.37%	1.02%	0.08%	0.00%	0.00%	0.00%	4.09%	6.52
E-SE	0.73%	2.59%	2.11%	0.31%	0.00%	0.00%	0.00%	5.74%	7.65
SE	0.75%	3.23%	4.66%	0.91%	0.12%	0.01%	0.00%	9.69%	8.62
S-SE	0.40%	3.06%	7.08%	2.43%	0.56%	0.07%	0.00%	13.62%	10.11
S	0.54%	3.45%	8.80%	5.50%	2.03%	0.46%	0.01%	20.80%	12.02
S-SW	0.20%	0.95%	1.72%	0.79%	0.31%	0.11%	0.00%	4.09%	11.05
SW	0.12%	0.48%	0.78%	0.22%	0.03%	0.03%	0.00%	1.65%	9.88
W-SW	0.12%	0.38%	0.42%	0.07%	0.01%	0.00%	0.00%	0.99%	8.07
W	0.09%	0.30%	0.55%	0.28%	0.11%	0.04%	0.01%	1.38%	10.92
W-NW	0.07%	0.30%	0.81%	0.23%	0.08%	0.05%	0.00%	1.53%	11.31
NW	0.12%	0.67%	1.09%	0.54%	0.27%	0.09%	0.05%	2.84%	11.13
N-NW	0.27%	1.64%	1.64%	0.83%	0.47%	0.07%	0.01%	4.93%	9.98
CALM	6.84%	1.60%	0.00%	0.00%	0.00%	0.00%	0.00%	8.44%	1.27
Total	12.29%	27.72%	38.51%	15.41%	4.87%	1.12%	0.09%	100.00%	9.31

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-230
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
November 1997 – 2006

CP COL 2.3(1)

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.54%	3.00%	4.62%	2.17%	0.78%	0.01%	0.00%	11.12%	10.48
N-NE	0.24%	1.07%	1.67%	0.57%	0.12%	0.03%	0.00%	3.69%	9.46
NE	0.40%	1.10%	0.83%	0.18%	0.00%	0.00%	0.00%	2.51%	7.47
E-NE	0.32%	1.04%	0.51%	0.01%	0.00%	0.00%	0.00%	1.89%	6.49
E	0.53%	1.47%	1.26%	0.03%	0.00%	0.00%	0.00%	3.29%	6.96
E-SE	0.62%	1.54%	1.42%	0.14%	0.00%	0.00%	0.00%	3.72%	7.47
SE	0.54%	2.12%	2.83%	0.44%	0.01%	0.00%	0.00%	5.96%	8.63
S-SE	0.32%	1.97%	5.41%	1.43%	0.21%	0.01%	0.00%	9.36%	10.14
S	0.53%	2.83%	8.23%	6.26%	2.57%	0.92%	0.10%	21.43%	13.14
S-SW	0.35%	1.33%	2.14%	0.75%	0.37%	0.18%	0.00%	5.12%	10.37
SW	0.18%	0.78%	0.76%	0.15%	0.12%	0.00%	0.00%	2.00%	8.67
W-SW	0.08%	0.58%	0.33%	0.14%	0.07%	0.03%	0.00%	1.24%	9.21
W	0.10%	0.54%	0.97%	0.65%	0.24%	0.12%	0.06%	2.68%	12.65
W-NW	0.06%	0.44%	1.75%	0.79%	0.39%	0.15%	0.08%	3.66%	12.43
NW	0.10%	1.01%	2.54%	1.19%	0.81%	0.35%	0.15%	6.15%	12.53
N-NW	0.25%	1.57%	3.80%	2.03%	0.87%	0.31%	0.08%	8.91%	11.97
CALM	6.66%	0.61%	0.00%	0.00%	0.00%	0.00%	0.00%	7.27%	0.76
Total	11.81%	23.01%	39.09%	16.94%	6.57%	2.11%	0.47%	100.00%	10.10

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-231
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
December 1997 – 2006

CP COL 2.3(1)

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.51%	1.96%	4.91%	2.29%	0.80%	0.14%	0.00%	10.61%	13.68
N-NE	0.16%	1.14%	2.12%	0.97%	0.20%	0.01%	0.00%	4.61%	9.60
NE	0.28%	0.80%	0.87%	0.07%	0.00%	0.01%	0.00%	2.03%	7.55
E-NE	0.31%	0.55%	0.64%	0.19%	0.00%	0.01%	0.00%	1.70%	8.10
E	0.37%	1.10%	0.73%	0.14%	0.01%	0.00%	0.00%	2.34%	6.76
E-SE	0.46%	1.23%	0.95%	0.35%	0.08%	0.00%	0.00%	3.07%	7.35
SE	0.41%	1.68%	1.53%	0.31%	0.16%	0.04%	0.00%	4.13%	8.21
S-SE	0.39%	2.26%	4.64%	1.75%	0.37%	0.00%	0.00%	9.40%	10.32
S	0.43%	3.02%	7.77%	5.03%	2.48%	0.83%	0.07%	19.62%	12.87
S-SW	0.34%	1.53%	2.00%	0.65%	0.28%	0.15%	0.05%	5.01%	10.29
SW	0.07%	0.88%	1.07%	0.20%	0.08%	0.04%	0.00%	2.34%	9.58
W-SW	0.09%	0.62%	0.85%	0.28%	0.15%	0.08%	0.04%	2.12%	10.80
W	0.14%	0.89%	1.58%	0.78%	0.24%	0.14%	0.08%	3.86%	11.70
W-NW	0.09%	0.87%	2.00%	1.15%	0.46%	0.26%	0.09%	4.92%	12.97
NW	0.16%	1.53%	3.79%	1.53%	1.08%	0.35%	0.14%	8.58%	12.57
N-NW	0.15%	1.57%	3.80%	2.58%	1.19%	0.46%	0.09%	9.85%	13.13
CALM	5.34%	0.47%	0.00%	0.00%	0.00%	0.00%	0.00%	5.82%	0.67
Total	9.71%	22.09%	39.25%	18.28%	7.59%	2.52%	0.57%	100.00%	10.89

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-232
Percentage Frequency of Wind Direction and Speed (MPH)
Dallas-Fort Worth Airport
Annual 1997 – 2006

CP COL 2.3(1)

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.48%	2.39%	3.72%	1.78%	0.59%	0.12%	0.01%	9.09%	11.17
N-NE	0.25%	1.21%	1.98%	0.81%	0.21%	0.04%	0.01%	4.50%	10.06
NE	0.32%	1.04%	0.93%	0.16%	0.04%	0.00%	0.00%	2.49%	7.86
E-NE	0.34%	0.94%	0.89%	0.18%	0.03%	0.01%	0.00%	2.39%	8.00
E	0.57%	1.96%	1.62%	0.22%	0.02%	0.00%	0.00%	4.39%	7.49
E-SE	0.53%	1.94%	2.01%	0.33%	0.03%	0.00%	0.00%	4.86%	8.01
SE	0.57%	2.40%	3.38%	0.87%	0.13%	0.02%	0.00%	7.38%	9.02
S-SE	0.39%	2.27%	5.57%	2.54%	0.63%	0.08%	0.01%	11.49%	10.94
S	0.48%	2.95%	9.30%	6.76%	3.05%	1.05%	0.15%	23.74%	13.17
S-SW	0.19%	1.21%	2.49%	1.13%	0.39%	0.11%	0.01%	5.54%	11.14
SW	0.09%	0.64%	0.93%	0.24%	0.10%	0.02%	0.00%	2.02%	9.83
W-SW	0.08%	0.41%	0.52%	0.17%	0.06%	0.04%	0.00%	1.29%	9.86
W	0.10%	0.40%	0.69%	0.34%	0.14%	0.07%	0.02%	1.75%	11.45
W-NW	0.07%	0.34%	0.82%	0.43%	0.20%	0.12%	0.03%	2.01%	12.74
NW	0.11%	0.69%	1.50%	0.81%	0.50%	0.21%	0.06%	3.87%	12.67
N-NW	0.20%	1.30%	2.19%	1.18%	0.66%	0.27%	0.05%	5.86%	12.04
CALM	5.83%	1.52%	0.00%	0.00%	0.00%	0.00%	0.00%	7.35%	1.55
Total	10.60%	23.60%	38.53%	17.96%	6.79%	2.16%	0.36%	100.00%	10.30

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-233
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
January 2001 – 2006

CP COL 2.3(1)

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.61%	2.17%	5.84%	2.22%	0.70%	0.29%	0.02%	11.86%	11.19
N-NE	0.27%	0.93%	1.18%	0.20%	0.02%	0.00%	0.00%	2.60%	8.18
NE	0.34%	0.93%	0.84%	0.00%	0.00%	0.00%	0.00%	2.11%	7.23
E-NE	0.36%	0.95%	0.63%	0.11%	0.00%	0.00%	0.00%	2.06%	7.07
E	0.23%	1.13%	1.04%	0.11%	0.00%	0.00%	0.00%	2.51%	7.72
E-SE	0.18%	0.91%	0.88%	0.07%	0.00%	0.00%	0.00%	2.04%	7.42
SE	0.41%	3.08%	5.50%	0.91%	0.11%	0.00%	0.00%	10.01%	8.68
S-SE	0.45%	3.26%	10.37%	1.83%	0.41%	0.07%	0.00%	16.39%	9.80
S	0.34%	1.68%	4.84%	1.97%	0.75%	0.07%	0.00%	9.64%	10.77
S-SW	0.09%	0.59%	1.09%	0.75%	0.43%	0.02%	0.00%	2.97%	11.03
SW	0.14%	0.36%	1.18%	0.38%	0.20%	0.05%	0.00%	2.31%	10.33
W-SW	0.34%	0.45%	1.04%	0.36%	0.20%	0.07%	0.02%	2.49%	9.42
W	0.68%	2.33%	1.79%	0.25%	0.38%	0.25%	0.02%	5.71%	8.60
W-NW	0.45%	1.20%	1.18%	0.34%	0.25%	0.07%	0.00%	3.49%	8.72
NW	0.27%	0.95%	1.63%	1.02%	0.57%	0.16%	0.07%	4.66%	11.63
N-NW	0.52%	1.40%	2.74%	2.17%	1.02%	0.27%	0.05%	8.17%	11.93
CALM	10.73%	0.00%	0.00%	0.00%	0.00%	0.00%	0.25%	10.98%	0.00
Total	16.41%	22.32%	41.77%	12.70%	5.05%	1.31%	0.43%	100.00%	8.75

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-234
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
February 2001 – 2006

CP COL 2.3(1)

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.78%	2.64%	5.38%	3.57%	1.83%	0.35%	0.00%	14.55%	11.56
N-NE	0.38%	1.26%	1.63%	0.75%	0.20%	0.03%	0.00%	4.25%	9.31
NE	0.38%	1.31%	1.38%	0.25%	0.03%	0.00%	0.00%	3.34%	8.00
E-NE	0.48%	1.16%	0.63%	0.05%	0.00%	0.00%	0.00%	2.31%	6.26
E	0.33%	1.13%	0.90%	0.03%	0.00%	0.00%	0.00%	2.39%	7.25
E-SE	0.35%	1.03%	1.28%	0.15%	0.00%	0.00%	0.00%	2.81%	7.85
SE	0.53%	2.19%	6.08%	1.23%	0.28%	0.00%	0.03%	10.33%	9.61
S-SE	0.35%	2.36%	9.22%	2.61%	0.53%	0.03%	0.00%	15.10%	10.58
S	0.45%	1.46%	4.02%	3.12%	0.95%	0.15%	0.00%	10.15%	11.80
S-SW	0.08%	0.25%	0.93%	0.50%	0.28%	0.08%	0.00%	2.11%	11.47
SW	0.15%	0.43%	0.63%	0.38%	0.18%	0.00%	0.03%	1.78%	11.41
W-SW	0.23%	0.40%	0.40%	0.30%	0.10%	0.03%	0.03%	1.48%	11.11
W	0.75%	1.63%	1.33%	0.33%	0.30%	0.00%	0.03%	4.37%	8.88
W-NW	0.55%	1.13%	1.06%	0.23%	0.13%	0.03%	0.05%	3.17%	9.14
NW	0.28%	0.95%	1.71%	0.83%	0.48%	0.15%	0.05%	4.45%	11.88
N-NW	0.38%	1.28%	2.66%	1.81%	1.21%	0.50%	0.13%	7.96%	12.63
CALM	9.45%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	9.45%	0.00
Total	15.88%	20.60%	39.25%	16.13%	6.48%	1.33%	0.33%	100.00%	9.45

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-235
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
March 2001 – 2006

CP COL 2.3(1)

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.64%	2.70%	4.07%	2.08%	1.08%	0.30%	0.00%	10.87%	11.17
N-NE	0.41%	1.33%	1.53%	0.50%	0.14%	0.00%	0.00%	3.91%	8.73
NE	0.30%	1.46%	1.92%	0.62%	0.09%	0.02%	0.00%	4.42%	8.61
E-NE	0.41%	1.14%	1.49%	0.30%	0.11%	0.07%	0.00%	3.52%	8.16
E	0.46%	1.26%	2.04%	0.43%	0.07%	0.00%	0.00%	4.26%	8.34
E-SE	0.25%	1.33%	1.99%	0.27%	0.21%	0.00%	0.00%	4.05%	8.16
SE	0.37%	2.61%	5.61%	1.10%	0.27%	0.05%	0.00%	10.00%	9.18
S-SE	0.30%	2.65%	9.06%	2.93%	0.69%	0.18%	0.00%	15.81%	10.49
S	0.18%	1.10%	5.49%	4.32%	2.31%	0.37%	0.02%	13.80%	12.89
S-SW	0.09%	0.39%	1.05%	0.87%	0.41%	0.07%	0.02%	2.91%	12.91
SW	0.23%	0.41%	0.78%	0.23%	0.18%	0.09%	0.02%	1.94%	10.72
W-SW	0.05%	0.55%	0.64%	0.34%	0.25%	0.07%	0.02%	1.92%	11.35
W	0.66%	1.37%	0.98%	0.53%	0.14%	0.21%	0.09%	3.98%	8.67
W-NW	0.32%	0.80%	0.53%	0.34%	0.34%	0.11%	0.07%	2.52%	10.55
NW	0.23%	0.78%	0.80%	0.73%	0.64%	0.21%	0.05%	3.43%	12.50
N-NW	0.18%	1.19%	2.10%	0.87%	1.08%	0.43%	0.09%	5.95%	12.83
CALM	6.70%	0.00%	0.00%	0.00%	0.00%	0.00%	0.02%	6.73%	0.00
Total	11.78%	21.07%	40.08%	16.47%	8.01%	2.17%	0.41%	100.00%	9.84

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-236
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
April 2001 – 2006

CP COL 2.3(1)

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.61%	1.60%	2.89%	1.85%	0.89%	0.19%	0.02%	8.05%	11.80
N-NE	0.16%	0.82%	1.31%	0.49%	0.07%	0.00%	0.00%	2.86%	9.37
NE	0.16%	0.63%	1.10%	0.31%	0.09%	0.00%	0.00%	2.30%	9.34
E-NE	0.16%	0.63%	0.96%	0.12%	0.05%	0.00%	0.00%	1.93%	8.64
E	0.31%	0.68%	1.34%	0.00%	0.05%	0.00%	0.00%	2.37%	8.09
E-SE	0.28%	1.06%	1.74%	0.31%	0.02%	0.00%	0.02%	3.43%	9.11
SE	0.40%	2.84%	7.04%	1.41%	0.12%	0.00%	0.00%	11.81%	9.42
S-SE	0.19%	3.24%	13.90%	4.30%	1.20%	0.26%	0.02%	23.10%	11.04
S	0.14%	1.13%	8.03%	7.49%	3.38%	0.31%	0.09%	20.57%	13.78
S-SW	0.00%	0.28%	0.73%	0.94%	0.40%	0.02%	0.00%	2.37%	13.28
SW	0.14%	0.28%	0.59%	0.31%	0.19%	0.00%	0.00%	1.50%	11.77
W-SW	0.12%	0.42%	0.47%	0.28%	0.07%	0.09%	0.02%	1.48%	11.98
W	0.38%	0.75%	0.82%	0.35%	0.38%	0.14%	0.05%	2.86%	10.54
W-NW	0.45%	0.40%	0.42%	0.42%	0.40%	0.09%	0.02%	2.21%	11.32
NW	0.26%	0.33%	1.01%	0.52%	0.33%	0.07%	0.12%	2.63%	12.63
N-NW	0.19%	0.42%	1.60%	0.92%	0.73%	0.19%	0.07%	4.11%	12.53
CALM	5.80%	0.00%	0.00%	0.00%	0.00%	0.00%	0.61%	6.41%	0.00
Total	9.74%	15.52%	43.95%	20.00%	8.36%	1.36%	1.06%	100.00%	10.73

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-237
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
May 2001 – 2006

CP COL 2.3(1)

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.60%	1.03%	1.88%	1.03%	0.64%	0.02%	0.00%	5.20%	10.60
N-NE	0.39%	0.96%	2.01%	0.57%	0.07%	0.00%	0.00%	4.01%	8.94
NE	0.32%	1.21%	1.85%	0.30%	0.16%	0.00%	0.00%	3.85%	9.04
E-NE	0.18%	1.24%	1.95%	0.27%	0.07%	0.02%	0.02%	3.75%	8.67
E	0.43%	1.51%	1.42%	0.18%	0.05%	0.00%	0.00%	3.59%	7.37
E-SE	0.32%	1.46%	1.85%	0.30%	0.07%	0.02%	0.00%	4.03%	8.23
SE	0.34%	2.95%	6.78%	1.65%	0.14%	0.02%	0.00%	11.88%	9.39
S-SE	0.41%	2.91%	15.86%	5.10%	1.37%	0.23%	0.05%	25.93%	11.13
S	0.27%	1.92%	9.00%	6.45%	1.92%	0.21%	0.05%	19.82%	12.44
S-SW	0.05%	0.57%	1.44%	1.28%	0.16%	0.00%	0.00%	3.50%	11.63
SW	0.14%	0.23%	0.73%	0.34%	0.02%	0.02%	0.00%	1.49%	9.99
W-SW	0.09%	0.25%	0.57%	0.16%	0.00%	0.00%	0.02%	1.10%	9.26
W	0.18%	0.41%	0.39%	0.16%	0.05%	0.00%	0.00%	1.19%	8.95
W-NW	0.16%	0.37%	0.32%	0.07%	0.02%	0.00%	0.02%	0.96%	8.38
NW	0.14%	0.34%	0.48%	0.14%	0.11%	0.00%	0.00%	1.21%	8.31
N-NW	0.05%	0.34%	0.69%	0.62%	0.23%	0.05%	0.00%	1.97%	12.17
CALM	6.50%	0.00%	0.00%	0.00%	0.00%	0.00%	0.02%	6.52%	0.00
Total	10.57%	17.72%	47.22%	18.63%	5.08%	0.60%	0.18%	100.00%	9.83

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-238
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
June 2001 – 2006

CP COL 2.3(1)

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.76%	2.19%	1.76%	0.43%	0.10%	0.00%	0.02%	5.26%	8.16
N-NE	0.36%	1.00%	1.81%	0.29%	0.12%	0.00%	0.00%	3.57%	8.62
NE	0.48%	1.62%	1.31%	0.36%	0.05%	0.02%	0.00%	3.83%	8.32
E-NE	0.36%	1.52%	1.19%	0.33%	0.02%	0.02%	0.00%	3.45%	7.56
E	0.43%	1.26%	1.93%	0.43%	0.05%	0.00%	0.00%	4.09%	8.32
E-SE	0.40%	2.19%	2.88%	0.43%	0.07%	0.00%	0.00%	5.97%	8.50
SE	0.69%	6.18%	8.59%	2.07%	0.33%	0.02%	0.02%	17.91%	8.81
S-SE	0.43%	5.92%	14.41%	3.57%	1.05%	0.10%	0.00%	25.48%	10.04
S	0.19%	1.81%	7.59%	4.59%	0.98%	0.14%	0.00%	15.29%	12.17
S-SW	0.07%	0.33%	1.36%	0.71%	0.05%	0.02%	0.00%	2.55%	11.74
SW	0.05%	0.29%	0.38%	0.12%	0.00%	0.00%	0.02%	0.86%	9.21
W-SW	0.02%	0.10%	0.21%	0.02%	0.00%	0.00%	0.00%	0.36%	8.16
W	0.12%	0.29%	0.26%	0.07%	0.02%	0.00%	0.00%	0.76%	7.04
W-NW	0.21%	0.21%	0.24%	0.05%	0.00%	0.00%	0.00%	0.71%	7.44
NW	0.21%	0.19%	0.24%	0.02%	0.00%	0.00%	0.00%	0.67%	6.73
N-NW	0.12%	0.43%	0.38%	0.17%	0.02%	0.00%	0.02%	1.14%	9.07
CALM	8.11%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	8.11%	0.00
Total	13.01%	25.52%	44.53%	13.65%	2.85%	0.33%	0.10%	100.00%	8.88

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-239
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
July 2001 – 2006

CP COL 2.3(1)

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.79%	1.59%	0.99%	0.25%	0.05%	0.00%	0.00%	3.67%	7.10
N-NE	0.09%	0.58%	0.69%	0.21%	0.07%	0.00%	0.00%	1.64%	8.97
NE	0.30%	0.69%	0.49%	0.14%	0.05%	0.02%	0.00%	1.69%	7.83
E-NE	0.16%	0.74%	0.97%	0.21%	0.05%	0.02%	0.00%	2.15%	8.82
E	0.37%	1.16%	1.59%	0.16%	0.07%	0.02%	0.00%	3.37%	7.85
E-SE	0.44%	1.71%	2.10%	0.25%	0.12%	0.02%	0.00%	4.65%	8.31
SE	0.76%	6.17%	7.65%	1.02%	0.14%	0.00%	0.00%	15.74%	8.27
S-SE	1.18%	8.99%	16.04%	2.70%	0.46%	0.00%	0.00%	29.37%	8.62
S	0.30%	4.16%	10.75%	3.30%	0.76%	0.02%	0.00%	19.30%	9.92
S-SW	0.16%	0.67%	2.54%	0.60%	0.07%	0.00%	0.00%	4.04%	9.73
SW	0.09%	0.65%	1.41%	0.12%	0.00%	0.00%	0.00%	2.26%	8.46
W-SW	0.09%	0.55%	0.83%	0.12%	0.00%	0.00%	0.00%	1.59%	8.41
W	0.14%	0.55%	0.44%	0.07%	0.02%	0.00%	0.00%	1.22%	7.82
W-NW	0.09%	0.18%	0.23%	0.00%	0.00%	0.00%	0.00%	0.51%	6.64
NW	0.05%	0.21%	0.23%	0.02%	0.02%	0.00%	0.02%	0.55%	9.51
N-NW	0.16%	0.23%	0.32%	0.05%	0.00%	0.00%	0.00%	0.76%	7.01
CALM	7.46%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	7.46%	0.00
Total	12.64%	28.84%	47.28%	9.22%	1.87%	0.12%	0.02%	100.00%	8.02

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-240
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
August 2001 – 2006

CP COL 2.3(1)

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.24%	1.93%	0.83%	0.17%	0.02%	0.00%	0.00%	4.19%	6.08
N-NE	0.21%	0.59%	0.74%	0.14%	0.02%	0.00%	0.00%	1.71%	8.45
NE	0.24%	0.83%	0.88%	0.14%	0.02%	0.02%	0.00%	2.14%	8.28
E-NE	0.29%	1.05%	1.31%	0.14%	0.02%	0.00%	0.00%	2.81%	8.01
E	0.45%	2.02%	2.74%	0.36%	0.02%	0.02%	0.00%	5.62%	8.24
E-SE	0.71%	2.78%	2.76%	0.52%	0.02%	0.02%	0.00%	6.83%	7.77
SE	1.05%	6.97%	6.42%	1.07%	0.17%	0.00%	0.00%	15.68%	7.82
S-SE	1.12%	8.07%	12.54%	1.40%	0.21%	0.00%	0.00%	23.34%	8.28
S	0.50%	3.09%	8.61%	2.62%	0.21%	0.02%	0.00%	15.06%	9.89
S-SW	0.05%	0.74%	2.17%	0.76%	0.07%	0.00%	0.00%	3.78%	10.46
SW	0.05%	0.36%	0.86%	0.21%	0.00%	0.00%	0.00%	1.48%	9.45
W-SW	0.21%	0.26%	0.74%	0.10%	0.02%	0.02%	0.02%	1.38%	8.75
W	0.31%	0.64%	0.62%	0.00%	0.00%	0.00%	0.00%	1.57%	6.26
W-NW	0.10%	0.29%	0.36%	0.05%	0.02%	0.00%	0.00%	0.81%	7.57
NW	0.26%	0.17%	0.21%	0.07%	0.00%	0.00%	0.00%	0.71%	7.04
N-NW	0.24%	0.45%	0.40%	0.07%	0.00%	0.00%	0.00%	1.17%	7.12
CALM	11.68%	0.00%	0.00%	0.00%	0.00%	0.00%	0.05%	11.73%	0.00
Total	18.70%	30.24%	42.18%	7.83%	0.86%	0.12%	0.07%	100.00%	7.32

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-241
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
September 2001 – 2006

CP COL 2.3(1)

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	1.03%	2.27%	3.00%	1.00%	0.24%	0.02%	0.00%	7.56%	8.52
N-NE	0.74%	1.79%	2.12%	0.76%	0.14%	0.00%	0.02%	5.58%	8.68
NE	0.50%	1.45%	2.05%	0.45%	0.07%	0.02%	0.00%	4.55%	8.21
E-NE	0.57%	1.65%	1.31%	0.21%	0.05%	0.00%	0.00%	3.79%	7.64
E	0.41%	2.36%	2.58%	0.43%	0.02%	0.02%	0.00%	5.82%	8.29
E-SE	0.55%	2.46%	2.31%	0.21%	0.07%	0.00%	0.00%	5.60%	7.72
SE	0.57%	6.37%	6.65%	0.91%	0.07%	0.00%	0.00%	14.57%	8.01
S-SE	0.60%	6.70%	8.56%	1.10%	0.14%	0.00%	0.00%	17.10%	8.36
S	0.17%	1.72%	6.08%	2.12%	0.64%	0.02%	0.00%	10.75%	10.84
S-SW	0.05%	0.62%	1.31%	0.55%	0.26%	0.00%	0.00%	2.79%	10.65
SW	0.26%	0.24%	0.45%	0.14%	0.05%	0.02%	0.00%	1.17%	7.50
W-SW	0.07%	0.17%	0.21%	0.02%	0.02%	0.00%	0.00%	0.50%	7.21
W	0.48%	0.83%	0.43%	0.07%	0.02%	0.00%	0.00%	1.84%	6.73
W-NW	0.52%	0.36%	0.14%	0.02%	0.00%	0.00%	0.00%	1.05%	5.44
NW	0.21%	0.38%	0.50%	0.10%	0.12%	0.00%	0.00%	1.31%	8.70
N-NW	0.21%	0.64%	0.88%	0.41%	0.05%	0.00%	0.00%	2.19%	8.76
CALM	13.78%	0.00%	0.00%	0.00%	0.00%	0.00%	0.05%	13.83%	0.00
Total	20.72%	30.00%	38.60%	8.51%	1.98%	0.12%	0.07%	100.00%	7.32

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-242
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
October 2001 – 2006

CP COL 2.3(1)

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.48%	2.03%	3.67%	1.48%	0.75%	0.18%	0.00%	8.60%	10.71
N-NE	0.21%	1.14%	1.55%	0.23%	0.05%	0.00%	0.00%	3.17%	8.65
NE	0.30%	1.03%	0.71%	0.21%	0.05%	0.00%	0.00%	2.28%	7.30
E-NE	0.14%	1.32%	0.57%	0.05%	0.05%	0.02%	0.00%	2.14%	6.54
E	0.25%	1.25%	1.07%	0.00%	0.00%	0.00%	0.00%	2.58%	6.64
E-SE	0.21%	1.30%	1.57%	0.02%	0.00%	0.00%	0.00%	3.10%	7.29
SE	0.66%	3.97%	6.57%	0.39%	0.07%	0.00%	0.00%	11.66%	8.15
S-SE	0.59%	4.81%	11.13%	1.41%	0.11%	0.00%	0.00%	18.07%	9.14
S	0.30%	1.76%	6.09%	2.26%	0.66%	0.00%	0.00%	11.06%	11.15
S-SW	0.11%	0.34%	0.98%	0.78%	0.11%	0.00%	0.00%	2.33%	11.02
SW	0.14%	0.50%	0.64%	0.23%	0.02%	0.00%	0.00%	1.53%	8.63
W-SW	0.18%	0.23%	0.46%	0.18%	0.02%	0.00%	0.00%	1.07%	8.90
W	0.43%	0.78%	0.68%	0.23%	0.00%	0.00%	0.00%	2.12%	7.33
W-NW	0.23%	0.43%	0.66%	0.18%	0.07%	0.07%	0.00%	1.64%	8.10
NW	0.32%	0.62%	1.16%	0.32%	0.14%	0.07%	0.02%	2.65%	9.80
N-NW	0.36%	0.55%	1.69%	0.84%	0.41%	0.16%	0.05%	4.06%	10.61
CALM	11.43%	0.00%	0.00%	0.00%	0.00%	0.00%	10.52%	21.94%	0.00
Total	16.33%	22.06%	39.21%	8.80%	2.51%	0.50%	10.58%	100.00%	8.01

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-243
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
November 2001 – 2006

CP COL 2.3(1)

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.70%	2.90%	4.07%	1.40%	0.61%	0.19%	0.02%	9.89%	10.07
N-NE	0.33%	0.73%	1.29%	0.37%	0.00%	0.00%	0.00%	2.71%	8.89
NE	0.23%	0.94%	0.75%	0.00%	0.00%	0.00%	0.00%	1.92%	6.80
E-NE	0.30%	0.63%	0.65%	0.02%	0.00%	0.00%	0.00%	1.61%	6.61
E	0.40%	1.03%	1.10%	0.02%	0.00%	0.00%	0.00%	2.55%	7.06
E-SE	0.30%	1.10%	1.15%	0.14%	0.00%	0.00%	0.00%	2.69%	7.80
SE	0.54%	3.27%	5.85%	0.65%	0.05%	0.00%	0.00%	10.36%	8.31
S-SE	0.73%	4.07%	13.54%	1.85%	0.37%	0.00%	0.00%	20.56%	9.35
S	0.40%	1.50%	5.87%	2.27%	0.56%	0.05%	0.00%	10.64%	10.79
S-SW	0.07%	0.54%	0.91%	0.91%	0.42%	0.00%	0.00%	2.85%	12.28
SW	0.05%	0.35%	0.65%	0.63%	0.14%	0.02%	0.00%	1.85%	11.69
W-SW	0.12%	0.42%	0.54%	0.16%	0.14%	0.02%	0.02%	1.43%	11.08
W	0.75%	1.78%	1.29%	0.33%	0.12%	0.05%	0.09%	4.40%	8.58
W-NW	0.42%	1.19%	1.05%	0.49%	0.21%	0.12%	0.09%	3.58%	10.17
NW	0.28%	1.03%	1.73%	1.15%	0.61%	0.19%	0.28%	5.26%	11.93
N-NW	0.30%	1.47%	2.60%	1.31%	0.63%	0.37%	0.26%	6.95%	11.74
CALM	10.34%	0.00%	0.00%	0.00%	0.00%	0.00%	0.40%	10.74%	0.00
Total	16.26%	22.95%	43.04%	11.72%	3.86%	1.01%	1.17%	100.00%	8.68

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-244
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
December 2001 – 2006

CP COL 2.3(1)

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.63%	2.22%	3.67%	1.99%	0.70%	0.05%	0.02%	9.29%	10.77
N-NE	0.23%	0.86%	1.11%	0.23%	0.07%	0.00%	0.00%	2.49%	8.69
NE	0.27%	0.52%	0.70%	0.02%	0.05%	0.02%	0.00%	1.59%	7.05
E-NE	0.34%	0.23%	0.34%	0.00%	0.02%	0.00%	0.00%	0.93%	6.33
E	0.14%	0.50%	0.54%	0.02%	0.00%	0.00%	0.00%	1.20%	7.19
E-SE	0.29%	0.72%	0.77%	0.05%	0.05%	0.00%	0.00%	1.88%	6.62
SE	0.48%	1.90%	6.25%	0.63%	0.20%	0.07%	0.00%	9.54%	8.91
S-SE	0.45%	3.35%	11.46%	2.06%	0.29%	0.02%	0.00%	17.65%	9.70
S	0.27%	1.88%	5.14%	2.70%	1.09%	0.11%	0.00%	11.19%	11.44
S-SW	0.16%	0.54%	1.22%	0.88%	0.25%	0.05%	0.00%	3.10%	11.78
SW	0.23%	0.45%	0.68%	0.20%	0.20%	0.02%	0.00%	1.79%	10.32
W-SW	0.18%	0.82%	0.68%	0.20%	0.07%	0.02%	0.00%	1.97%	8.17
W	1.00%	2.83%	1.95%	0.45%	0.16%	0.14%	0.05%	6.57%	8.12
W-NW	0.54%	1.31%	1.56%	0.48%	0.34%	0.11%	0.00%	4.35%	9.25
NW	0.59%	0.91%	2.67%	1.40%	0.72%	0.27%	0.18%	6.75%	12.58
N-NW	0.20%	1.25%	2.72%	1.74%	0.86%	0.18%	0.07%	7.02%	12.21
CALM	12.69%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	12.69%	0.00
Total	18.69%	20.30%	41.48%	13.07%	5.07%	1.06%	0.32%	100.00%	8.76

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-245
Percentage Frequency of Wind Direction and Speed (MPH)
Mineral Wells Airport
Annual 2001 – 2006

CP COL 2.3(1)

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.74%	2.10%	3.17%	1.45%	0.63%	0.13%	0.01%	8.23%	10.45
N-NE	0.31%	1.00%	1.41%	0.39%	0.08%	0.00%	0.00%	3.20%	8.80
NE	0.32%	1.05%	1.16%	0.23%	0.05%	0.01%	0.00%	2.83%	8.14
E-NE	0.31%	1.02%	1.00%	0.15%	0.04%	0.01%	0.00%	2.53%	7.70
E	0.35%	1.27%	1.52%	0.18%	0.03%	0.01%	0.00%	3.35%	7.86
E-SE	0.36%	1.50%	1.77%	0.23%	0.05%	0.01%	0.00%	3.91%	8.00
SE	0.56%	4.03%	6.58%	1.08%	0.16%	0.01%	0.00%	12.43%	8.66
S-SE	0.57%	4.69%	12.19%	2.57%	0.57%	0.07%	0.01%	20.67%	9.70
S	0.29%	1.93%	6.80%	3.60%	1.19%	0.12%	0.01%	13.95%	11.62
S-SW	0.08%	0.49%	1.31%	0.80%	0.24%	0.02%	0.00%	2.95%	11.42
SW	0.14%	0.38%	0.75%	0.27%	0.10%	0.02%	0.01%	1.67%	10.07
W-SW	0.14%	0.39%	0.57%	0.19%	0.08%	0.03%	0.01%	1.40%	9.70
W	0.49%	1.19%	0.92%	0.24%	0.13%	0.07%	0.03%	3.06%	8.36
W-NW	0.34%	0.66%	0.65%	0.22%	0.15%	0.05%	0.02%	2.09%	9.22
NW	0.26%	0.57%	1.04%	0.53%	0.31%	0.09%	0.07%	2.87%	11.47
N-NW	0.24%	0.81%	1.57%	0.92%	0.52%	0.18%	0.06%	4.29%	11.75
CALM	9.55%	0.00%	0.00%	0.00%	0.00%	0.00%	1.01%	10.57%	0.00
Total	15.05%	23.07%	42.40%	13.06%	4.33%	0.84%	1.25%	100.00%	8.81

NOTES:

1. Calm is classified as a wind speed less than 2.3 mph or a variable wind direction.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-246
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
January 2001 – 2006

CP COL 2.3(1)

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.24%	0.89%	3.20%	3.63%	1.32%	0.16%	0.00%	9.44%	12.48
N-NE	0.22%	0.91%	1.75%	1.18%	0.05%	0.00%	0.00%	4.12%	9.71
NE	0.16%	0.46%	0.70%	0.51%	0.03%	0.00%	0.00%	1.86%	9.03
E-NE	0.30%	0.30%	0.86%	0.19%	0.00%	0.00%	0.00%	1.64%	7.78
E	0.27%	1.43%	0.70%	0.16%	0.00%	0.00%	0.00%	2.56%	6.47
E-SE	0.46%	1.91%	1.45%	0.08%	0.00%	0.00%	0.00%	3.90%	6.34
SE	0.43%	2.07%	4.98%	0.70%	0.03%	0.00%	0.00%	8.20%	8.27
S-SE	0.24%	2.31%	5.46%	2.61%	0.43%	0.00%	0.00%	11.05%	9.96
S	0.43%	1.69%	3.85%	3.85%	1.29%	0.05%	0.00%	11.16%	11.47
S-SW	0.51%	1.88%	2.26%	2.18%	0.91%	0.03%	0.00%	7.77%	10.45
SW	0.67%	1.94%	1.34%	1.86%	0.51%	0.00%	0.00%	6.32%	9.60
W-SW	0.54%	1.88%	1.59%	1.21%	0.27%	0.00%	0.00%	5.49%	8.92
W	0.48%	0.62%	0.46%	0.32%	0.11%	0.00%	0.00%	1.99%	7.67
W-NW	0.51%	1.21%	1.37%	0.59%	0.16%	0.08%	0.00%	3.93%	8.32
NW	0.35%	2.18%	2.82%	1.56%	1.00%	0.24%	0.11%	8.26%	10.77
N-NW	0.30%	1.32%	2.85%	3.42%	2.80%	0.78%	0.11%	11.57%	13.99
CALM	0.75%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.75%	0.00
Total	6.11%	23.00%	35.64%	24.05%	8.90%	1.34%	0.22%	100.00%	10.25

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-247
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
February 2001 – 2006

CP COL 2.3(1)

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.27%	1.45%	5.44%	3.96%	2.54%	0.80%	0.03%	14.48%	12.88
N-NE	0.12%	0.95%	1.60%	1.71%	0.35%	0.09%	0.03%	4.85%	11.30
NE	0.27%	0.86%	1.48%	0.35%	0.03%	0.00%	0.00%	2.98%	8.14
E-NE	0.21%	0.83%	1.12%	0.15%	0.00%	0.00%	0.00%	2.30%	7.39
E	0.50%	1.74%	0.59%	0.00%	0.00%	0.00%	0.00%	2.84%	5.46
E-SE	0.33%	3.34%	1.39%	0.24%	0.09%	0.00%	0.00%	5.38%	6.49
SE	0.33%	2.98%	5.17%	1.45%	0.30%	0.00%	0.00%	10.22%	8.88
S-SE	0.47%	1.60%	5.02%	3.25%	1.21%	0.12%	0.00%	11.67%	11.18
S	0.30%	1.39%	3.63%	3.90%	1.95%	0.41%	0.24%	11.82%	13.08
S-SW	0.38%	0.80%	1.33%	1.27%	0.41%	0.03%	0.00%	4.23%	10.53
SW	0.30%	0.68%	1.03%	0.38%	0.12%	0.00%	0.00%	2.51%	8.45
W-SW	0.44%	1.12%	0.56%	0.27%	0.12%	0.09%	0.00%	2.60%	7.65
W	0.47%	0.50%	0.15%	0.09%	0.09%	0.00%	0.00%	1.30%	6.09
W-NW	0.59%	1.06%	1.09%	0.71%	0.12%	0.03%	0.00%	3.61%	8.27
NW	0.33%	1.95%	1.98%	1.24%	1.03%	0.27%	0.00%	6.80%	11.05
N-NW	0.15%	0.98%	2.54%	3.52%	2.51%	1.95%	0.24%	11.88%	15.53
CALM	0.53%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.53%	0.00
Total	5.44%	22.22%	34.13%	22.49%	10.87	3.78%	0.53%	100.00%	10.92

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-248
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
March 2001 – 2006

CP COL 2.3(1)

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.19%	1.48%	3.69%	1.96%	1.40%	0.32%	0.05%	9.10%	11.87
N-NE	0.27%	1.40%	3.77%	1.80%	0.62%	0.16%	0.03%	8.05%	10.74
NE	0.22%	0.86%	2.56%	1.05%	0.24%	0.00%	0.00%	4.92%	9.97
E-NE	0.19%	1.48%	1.70%	0.27%	0.00%	0.00%	0.00%	3.63%	7.76
E	0.22%	1.32%	1.21%	0.16%	0.03%	0.00%	0.00%	2.93%	7.28
E-SE	0.24%	2.61%	1.43%	0.19%	0.00%	0.00%	0.00%	4.47%	6.57
SE	0.27%	2.99%	5.71%	1.13%	0.03%	0.05%	0.00%	10.17%	8.64
S-SE	0.32%	1.99%	5.73%	4.74%	2.18%	0.48%	0.00%	15.45%	12.18
S	0.38%	1.56%	3.61%	4.71%	4.17%	0.65%	0.03%	15.10%	13.93
S-SW	0.40%	0.89%	1.61%	0.91%	0.35%	0.00%	0.00%	4.17%	9.80
SW	0.32%	0.73%	1.40%	0.54%	0.16%	0.03%	0.00%	3.18%	9.21
W-SW	0.16%	0.97%	0.81%	0.35%	0.22%	0.11%	0.00%	2.61%	9.32
W	0.13%	0.32%	0.48%	0.16%	0.03%	0.05%	0.00%	1.18%	8.91
W-NW	0.11%	0.54%	0.73%	0.38%	0.13%	0.11%	0.00%	1.99%	10.29
NW	0.19%	1.56%	1.29%	0.97%	0.70%	0.08%	0.08%	4.87%	10.79
N-NW	0.16%	1.10%	1.67%	2.42%	1.72%	0.59%	0.27%	7.94%	14.26
CALM	0.24%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.24%	0.00
Total	3.77%	21.80%	37.38%	21.74%	11.98%	2.64%	0.46%	100.00%	11.00

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-249
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
April 2001 – 2006

CP COL 2.3(1)

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.00%	0.75%	2.04%	1.95%	0.75%	0.34%	0.00%	5.82%	12.73
N-NE	0.14%	0.89%	1.83%	1.12%	0.40%	0.09%	0.03%	4.50%	10.77
NE	0.20%	0.77%	1.46%	0.37%	0.14%	0.00%	0.00%	2.95%	8.65
E-NE	0.14%	0.89%	1.00%	0.17%	0.00%	0.00%	0.00%	2.21%	7.28
E	0.14%	1.63%	0.80%	0.06%	0.00%	0.00%	0.00%	2.64%	6.25
E-SE	0.40%	2.38%	2.18%	0.17%	0.00%	0.00%	0.00%	5.13%	6.89
SE	0.11%	2.38%	6.08%	1.55%	0.03%	0.00%	0.00%	10.15%	9.08
S-SE	0.40%	2.29%	9.12%	7.08%	3.47%	0.37%	0.06%	22.79%	12.29
S	0.34%	1.75%	4.87%	9.26%	5.22%	1.12%	0.32%	22.88%	14.37
S-SW	0.32%	0.97%	1.69%	1.38%	0.72%	0.03%	0.00%	5.10%	10.91
SW	0.52%	0.72%	0.97%	0.29%	0.00%	0.00%	0.00%	2.49%	7.12
W-SW	0.23%	0.52%	0.77%	0.49%	0.17%	0.00%	0.00%	2.18%	9.43
W	0.23%	0.69%	0.37%	0.09%	0.09%	0.00%	0.00%	1.46%	6.99
W-NW	0.23%	0.52%	0.26%	0.60%	0.06%	0.03%	0.00%	1.69%	9.43
NW	0.29%	0.80%	0.83%	0.54%	0.23%	0.00%	0.00%	2.69%	9.21
N-NW	0.06%	0.75%	1.58%	1.03%	1.12%	0.49%	0.09%	5.10%	13.70
CALM	0.20%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.20%	0.00
Total	3.76%	18.69%	35.87%	26.15%	12.39%	2.47%	0.49%	100.00%	11.32

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-250
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
May 2001 – 2006

CP COL 2.3(1)

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.27%	1.08%	1.80%	1.10%	0.83%	0.19%	0.03%	5.30%	11.40
N-NE	0.22%	0.75%	2.29%	0.91%	0.16%	0.03%	0.00%	4.36%	9.69
NE	0.27%	1.08%	2.42%	0.59%	0.13%	0.03%	0.00%	4.52%	8.83
E-NE	0.11%	1.29%	1.10%	0.46%	0.05%	0.00%	0.00%	3.01%	8.09
E	0.11%	1.61%	1.43%	0.05%	0.00%	0.00%	0.00%	3.20%	6.93
E-SE	0.32%	2.37%	1.83%	0.35%	0.00%	0.00%	0.00%	4.87%	7.15
SE	0.32%	2.23%	6.27%	2.04%	0.11%	0.00%	0.00%	10.97%	9.35
S-SE	0.16%	2.10%	8.85%	10.16%	4.19%	0.19%	0.00%	25.65%	12.85
S	0.27%	1.40%	5.38%	8.85%	3.85%	0.27%	0.00%	20.01%	13.45
S-SW	0.19%	0.97%	1.83%	1.59%	0.46%	0.00%	0.00%	5.03%	10.81
SW	0.46%	0.81%	0.91%	0.67%	0.05%	0.03%	0.00%	2.93%	8.55
W-SW	0.19%	0.97%	0.13%	0.30%	0.05%	0.00%	0.00%	1.64%	7.35
W	0.24%	0.27%	0.08%	0.05%	0.00%	0.00%	0.00%	0.65%	5.02
W-NW	0.30%	0.67%	0.22%	0.03%	0.00%	0.00%	0.00%	1.21%	5.20
NW	0.16%	0.81%	0.56%	0.24%	0.05%	0.00%	0.00%	1.83%	7.88
N-NW	0.13%	0.81%	1.21%	1.21%	0.86%	0.22%	0.05%	4.49%	12.73
CALM	0.35%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.35%	0.00
Total	3.71%	19.20%	36.30%	28.61%	10.81%	0.94%	0.08%	100.00%	10.98

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-251
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
June 2001 – 2006

CP COL 2.3(1)

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.31%	1.37%	2.26%	0.92%	0.11%	0.03%	0.00%	5.00%	8.89
N-NE	0.14%	1.84%	2.74%	1.09%	0.28%	0.03%	0.03%	6.14%	9.21
NE	0.31%	1.70%	1.87%	0.53%	0.34%	0.00%	0.03%	4.78%	8.72
E-NE	0.45%	1.51%	1.56%	0.34%	0.00%	0.00%	0.00%	3.85%	7.06
E	0.25%	2.93%	1.12%	0.11%	0.00%	0.00%	0.00%	4.41%	6.22
E-SE	0.25%	4.38%	3.13%	0.45%	0.00%	0.00%	0.00%	8.21%	6.95
SE	0.59%	6.09%	10.67%	2.96%	0.22%	0.00%	0.00%	20.52%	8.73
S-SE	0.36%	3.32%	10.16%	5.78%	1.34%	0.11%	0.00%	21.08%	10.62
S	0.28%	1.79%	4.30%	4.36%	2.15%	0.34%	0.00%	13.21%	12.44
S-SW	0.47%	1.12%	1.59%	0.53%	0.20%	0.00%	0.00%	3.91%	8.38
SW	0.47%	0.47%	0.45%	0.08%	0.06%	0.00%	0.00%	1.54%	6.35
W-SW	0.34%	0.81%	0.25%	0.06%	0.00%	0.00%	0.00%	1.45%	5.29
W	0.20%	0.39%	0.20%	0.00%	0.00%	0.00%	0.00%	0.78%	5.70
W-NW	0.31%	0.47%	0.34%	0.06%	0.00%	0.00%	0.00%	1.17%	5.66
NW	0.31%	0.64%	0.17%	0.03%	0.03%	0.00%	0.00%	1.17%	5.57
N-NW	0.31%	0.67%	0.73%	0.50%	0.11%	0.00%	0.00%	2.32%	8.55
CALM	0.45%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.45%	0.00
Total	5.33%	29.52%	41.52%	17.79%	4.83%	0.50%	0.06%	100.00%	9.10

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-252
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
July 2001 – 2006

CP COL 2.3(1)

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.16%	0.38%	0.97%	0.40%	0.05%	0.00%	0.00%	1.97%	9.39
N-NE	0.11%	0.54%	0.97%	0.40%	0.00%	0.00%	0.00%	2.02%	8.75
NE	0.13%	1.10%	0.62%	0.27%	0.00%	0.00%	0.00%	2.13%	7.21
E-NE	0.24%	0.81%	0.54%	0.22%	0.00%	0.00%	0.00%	1.80%	7.01
E	0.38%	1.56%	0.54%	0.05%	0.00%	0.00%	0.00%	2.53%	5.72
E-SE	0.19%	2.96%	1.75%	0.05%	0.00%	0.00%	0.00%	4.96%	6.57
SE	0.22%	4.44%	8.62%	1.78%	0.05%	0.03%	0.00%	15.14%	8.68
S-SE	0.43%	4.66%	13.74%	5.20%	0.32%	0.00%	0.00%	24.35%	9.73
S	0.35%	3.72%	13.06%	5.44%	0.57%	0.00%	0.00%	23.13%	10.18
S-SW	0.43%	2.94%	6.06%	1.32%	0.03%	0.00%	0.00%	10.77%	8.60
SW	0.54%	2.10%	2.80%	0.13%	0.00%	0.00%	0.00%	5.58%	7.09
W-SW	0.40%	0.86%	1.05%	0.05%	0.00%	0.00%	0.00%	2.37%	6.33
W	0.16%	0.40%	0.22%	0.00%	0.00%	0.00%	0.00%	0.78%	5.40
W-NW	0.16%	0.22%	0.11%	0.03%	0.00%	0.00%	0.00%	0.51%	5.33
NW	0.08%	0.22%	0.22%	0.03%	0.00%	0.00%	0.00%	0.54%	6.54
N-NW	0.11%	0.40%	0.46%	0.13%	0.00%	0.00%	0.00%	1.10%	7.71
CALM	0.32%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.32%	0.00
Total	4.09%	27.31%	51.71%	15.51%	1.02%	0.03%	0.00%	100.00%	8.82

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-253
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
August 2001 – 2006

CP COL 2.3(1)

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.16%	0.97%	0.97%	0.03%	0.05%	0.03%	0.00%	2.20%	7.49
N-NE	0.22%	1.67%	1.21%	0.27%	0.16%	0.03%	0.00%	3.55%	7.77
NE	0.38%	2.12%	1.32%	0.27%	0.13%	0.00%	0.00%	4.22%	7.10
E-NE	0.24%	2.26%	2.39%	0.40%	0.00%	0.00%	0.00%	5.30%	7.46
E	0.35%	3.12%	1.42%	0.16%	0.00%	0.00%	0.00%	5.05%	6.22
E-SE	0.51%	4.70%	2.55%	0.22%	0.03%	0.00%	0.00%	8.01%	6.55
SE	0.46%	5.27%	7.39%	1.34%	0.19%	0.00%	0.00%	14.65%	8.12
S-SE	0.38%	4.06%	10.46%	3.49%	0.16%	0.00%	0.00%	18.55%	9.24
S	0.38%	2.53%	9.62%	5.62%	0.30%	0.00%	0.00%	18.44%	10.49
S-SW	0.65%	2.31%	3.33%	0.86%	0.00%	0.00%	0.00%	7.15%	7.76
SW	0.75%	1.83%	1.45%	0.22%	0.00%	0.00%	0.00%	4.25%	6.21
W-SW	0.59%	1.26%	0.32%	0.00%	0.00%	0.00%	0.00%	2.18%	4.58
W	0.70%	0.56%	0.08%	0.03%	0.00%	0.00%	0.00%	1.37%	3.81
W-NW	0.30%	0.75%	0.27%	0.00%	0.00%	0.00%	0.00%	1.32%	4.99
NW	0.30%	0.94%	0.32%	0.05%	0.00%	0.00%	0.00%	1.61%	5.43
N-NW	0.13%	0.89%	0.46%	0.05%	0.03%	0.00%	0.00%	1.56%	6.39
CALM	0.59%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.59%	0.00
Total	6.48%	35.24%	43.58%	13.01%	1.05%	0.05%	0.00%	100.00%	8.04

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-254
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
September 2001 – 2006

CP COL 2.3(1)

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.14%	2.00%	3.64%	2.17%	0.47%	0.00%	0.00%	8.43%	10.05
N-NE	0.64%	1.86%	3.95%	1.67%	0.61%	0.00%	0.00%	8.74%	9.55
NE	0.53%	1.73%	2.73%	0.86%	0.03%	0.00%	0.00%	5.87%	8.04
E-NE	0.83%	2.17%	2.17%	0.50%	0.11%	0.00%	0.00%	5.79%	7.22
E	0.31%	2.92%	2.06%	0.11%	0.00%	0.00%	0.00%	5.40%	6.60
E-SE	0.47%	5.65%	1.78%	0.17%	0.00%	0.00%	0.00%	8.07%	5.88
SE	0.45%	6.46%	7.85%	1.00%	0.06%	0.00%	0.00%	15.80%	7.61
S-SE	0.86%	3.76%	6.76%	2.17%	0.17%	0.06%	0.00%	13.77%	8.65
S	0.78%	1.25%	3.26%	2.62%	1.00%	0.06%	0.00%	8.96%	10.88
S-SW	0.89%	1.84%	1.14%	0.61%	0.17%	0.00%	0.00%	4.65%	7.33
SW	1.11%	0.89%	0.36%	0.22%	0.00%	0.00%	0.00%	2.59%	5.05
W-SW	0.58%	0.78%	0.33%	0.00%	0.00%	0.00%	0.00%	1.70%	4.82
W	0.33%	0.42%	0.08%	0.00%	0.00%	0.00%	0.00%	0.83%	3.89
W-NW	0.42%	0.70%	0.56%	0.08%	0.08%	0.00%	0.00%	1.84%	6.46
NW	0.47%	1.81%	0.42%	0.78%	0.03%	0.00%	0.00%	3.51%	7.31
N-NW	0.19%	1.14%	0.89%	0.86%	0.06%	0.00%	0.00%	3.14%	8.81
CALM	0.92%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.92%	0.00
Total	9.02%	35.36%	37.98%	13.83%	2.78%	0.11%	0.00%	100.00%	8.02

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-255
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
October 2001 – 2006

CP COL 2.3(1)

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.27%	1.02%	3.44%	2.66%	1.29%	0.40%	0.03%	9.12%	12.43
N-NE	0.11%	1.43%	2.72%	1.02%	0.08%	0.05%	0.00%	5.40%	9.32
NE	0.22%	1.32%	1.13%	0.35%	0.03%	0.00%	0.00%	3.04%	7.83
E-NE	0.27%	1.21%	1.16%	0.08%	0.00%	0.00%	0.00%	2.72%	6.77
E	0.30%	2.23%	1.34%	0.00%	0.00%	0.00%	0.00%	3.87%	6.08
E-SE	0.48%	3.76%	0.99%	0.00%	0.00%	0.00%	0.00%	5.24%	5.47
SE	0.38%	5.03%	6.10%	0.62%	0.00%	0.00%	0.00%	12.13%	7.49
S-SE	0.59%	2.93%	8.39%	3.47%	0.16%	0.00%	0.00%	15.54%	9.47
S	0.70%	1.94%	4.84%	4.38%	1.29%	0.00%	0.00%	13.15%	11.01
S-SW	0.83%	2.02%	2.02%	1.21%	0.22%	0.00%	0.00%	6.29%	8.30
SW	0.91%	1.99%	1.05%	0.59%	0.13%	0.00%	0.00%	4.68%	6.98
W-SW	0.40%	1.64%	1.05%	0.24%	0.00%	0.00%	0.00%	3.33%	6.44
W	0.35%	0.73%	0.54%	0.00%	0.00%	0.00%	0.00%	1.61%	5.55
W-NW	0.56%	0.91%	0.83%	0.27%	0.08%	0.03%	0.00%	2.69%	7.16
NW	0.56%	1.59%	1.48%	1.18%	0.24%	0.05%	0.00%	5.11%	8.82
N-NW	0.24%	0.99%	1.69%	1.32%	0.89%	0.30%	0.08%	5.51%	12.43
CALM	0.56%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.56%	0.00
Total	7.18%	30.73%	38.77%	17.40%	4.41%	0.83%	0.11%	100.00%	8.89

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-256
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
November 2001 – 2006

CP COL 2.3(1)

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.14%	1.12%	4.61%	2.46%	1.15%	0.09%	0.00%	9.57%	11.48
N-NE	0.26%	0.80%	1.69%	0.80%	0.43%	0.03%	0.00%	4.01%	10.20
NE	0.32%	0.74%	1.00%	0.14%	0.00%	0.00%	0.00%	2.21%	7.17
E-NE	0.11%	0.72%	1.17%	0.17%	0.03%	0.00%	0.00%	2.21%	7.72
E	0.29%	1.75%	1.58%	0.06%	0.00%	0.00%	0.00%	3.67%	6.47
E-SE	0.26%	2.23%	0.86%	0.06%	0.00%	0.00%	0.00%	3.41%	6.21
SE	0.20%	2.66%	5.76%	0.60%	0.00%	0.00%	0.00%	9.23%	8.20
S-SE	0.34%	2.23%	8.08%	3.44%	0.29%	0.00%	0.00%	14.38%	9.89
S	0.34%	1.40%	4.61%	5.16%	1.09%	0.11%	0.00%	12.72%	11.60
S-SW	0.54%	2.23%	1.83%	1.66%	0.26%	0.09%	0.00%	6.62%	9.05
SW	0.66%	1.26%	1.43%	0.69%	0.23%	0.03%	0.00%	4.30%	8.36
W-SW	0.52%	1.92%	1.00%	0.40%	0.09%	0.03%	0.00%	3.95%	7.04
W	0.34%	0.66%	0.34%	0.17%	0.11%	0.00%	0.00%	1.63%	6.98
W-NW	0.34%	0.97%	1.20%	0.92%	0.32%	0.06%	0.09%	3.90%	10.30
NW	0.34%	2.09%	2.12%	1.29%	1.17%	0.49%	0.26%	7.77%	11.85
N-NW	0.11%	1.29%	2.12%	3.64%	1.95%	0.72%	0.26%	10.09%	14.21
CALM	0.34%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.34%	0.00
Total	5.13%	24.10%	39.43%	21.66%	7.11%	1.63%	0.60%	100.00%	10.05

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-257
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
December 2001 – 2006

CP COL 2.3(1)

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.16%	1.05%	3.23%	2.53%	1.24%	0.19%	0.00%	8.40%	12.24
N-NE	0.08%	0.54%	2.13%	0.75%	0.16%	0.05%	0.00%	3.72%	10.61
NE	0.05%	0.40%	0.65%	0.08%	0.03%	0.00%	0.00%	1.21%	7.99
E-NE	0.27%	0.19%	0.43%	0.03%	0.00%	0.00%	0.00%	0.92%	6.64
E	0.13%	0.65%	0.40%	0.00%	0.00%	0.00%	0.00%	1.19%	5.83
E-SE	0.16%	1.72%	0.62%	0.08%	0.00%	0.00%	0.00%	2.59%	6.17
SE	0.40%	3.34%	4.17%	0.59%	0.11%	0.00%	0.00%	8.62%	7.79
S-SE	0.38%	2.80%	7.41%	3.56%	0.67%	0.03%	0.00%	14.84%	10.19
S	0.35%	2.59%	5.55%	3.82%	2.21%	0.08%	0.00%	14.60%	11.38
S-SW	0.73%	2.40%	2.18%	1.40%	0.57%	0.27%	0.00%	7.54%	9.37
SW	0.67%	1.89%	1.08%	0.51%	0.08%	0.11%	0.05%	4.39%	7.46
W-SW	0.78%	2.29%	2.18%	0.38%	0.08%	0.00%	0.00%	5.71%	6.97
W	0.51%	1.16%	0.75%	0.05%	0.03%	0.00%	0.00%	2.50%	6.14
W-NW	0.38%	1.75%	1.99%	0.94%	0.30%	0.11%	0.08%	5.55%	9.46
NW	0.19%	2.56%	3.42%	1.91%	1.62%	0.35%	0.03%	10.07%	11.34
N-NW	0.27%	0.73%	2.13%	1.99%	1.97%	0.62%	0.05%	7.76%	13.93
CALM	0.40%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.40%	0.00
Total	5.52%	26.04%	38.32%	18.64%	9.05%	1.80%	0.22%	100.00%	9.99

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-258
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Lower Level
Annual 2001 – 2006

CP COL 2.3(1)

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.19%	1.13%	2.92%	1.97%	0.93%	0.21%	0.01%	7.35%	11.66
N-NE	0.21%	1.13%	2.22%	1.06%	0.27%	0.05%	0.01%	4.95%	9.88
NE	0.25%	1.10%	1.49%	0.45%	0.09%	0.00%	0.00%	3.39%	8.34
E-NE	0.28%	1.14%	1.27%	0.25%	0.02%	0.00%	0.00%	2.95%	7.38
E	0.27%	1.91%	1.10%	0.08%	0.00%	0.00%	0.00%	3.35%	6.34
E-SE	0.34%	3.17%	1.66%	0.17%	0.01%	0.00%	0.00%	5.35%	6.45
SE	0.35%	3.83%	6.56%	1.31%	0.09%	0.01%	0.00%	12.15%	8.40
S-SE	0.41%	2.85%	8.28%	4.59%	1.21%	0.11%	0.00%	17.46%	10.66
S	0.41%	1.93%	5.58%	5.16%	2.08%	0.25%	0.05%	15.46%	12.08
S-SW	0.53%	1.71%	2.26%	1.24%	0.36%	0.04%	0.00%	6.13%	9.20
SW	0.62%	1.29%	1.20%	0.52%	0.11%	0.02%	0.00%	3.75%	7.69
W-SW	0.43%	1.26%	0.84%	0.31%	0.08%	0.02%	0.00%	2.94%	7.26
W	0.35%	0.56%	0.31%	0.08%	0.04%	0.00%	0.00%	1.34%	6.21
W-NW	0.35%	0.82%	0.75%	0.38%	0.10%	0.04%	0.01%	2.44%	8.32
NW	0.30%	1.43%	1.30%	0.82%	0.51%	0.12%	0.04%	4.51%	10.16
N-NW	0.18%	0.92%	1.52%	1.66%	1.16%	0.46%	0.09%	6.00%	13.39
CALM	0.47%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.47%	0.00
Total	5.46%	26.14%	39.27%	20.04%	7.06%	1.32%	0.23%	100.00%	9.77

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-259
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
January 2001 – 2006

CP COL 2.3(1)

January	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.18%	0.89%	2.86%	3.35%	1.75%	0.46%	0.12%	9.63%	13.50
N-NE	0.31%	0.89%	1.69%	1.82%	0.22%	0.00%	0.00%	4.92%	10.64
NE	0.28%	0.40%	0.71%	0.80%	0.15%	0.00%	0.00%	2.34%	10.13
E-NE	0.25%	0.58%	0.92%	0.49%	0.09%	0.00%	0.00%	2.34%	9.11
E	0.34%	0.86%	1.02%	0.25%	0.00%	0.00%	0.00%	2.46%	7.08
E-SE	0.22%	1.14%	1.17%	0.31%	0.00%	0.00%	0.00%	2.83%	7.64
SE	0.25%	1.29%	2.98%	2.58%	0.52%	0.00%	0.00%	7.63%	10.80
S-SE	0.12%	1.23%	2.71%	3.88%	2.22%	0.28%	0.00%	10.43%	13.28
S	0.12%	1.14%	2.31%	3.91%	2.03%	0.92%	0.00%	10.43%	14.02
S-SW	0.18%	0.52%	1.60%	2.68%	2.00%	1.17%	0.12%	8.28%	15.65
SW	0.22%	0.86%	1.20%	1.42%	1.57%	0.58%	0.09%	5.94%	14.22
W-SW	0.18%	0.68%	0.92%	1.02%	1.63%	1.32%	0.09%	5.85%	16.04
W	0.25%	0.52%	0.34%	0.43%	1.17%	0.28%	0.00%	2.98%	14.01
W-NW	0.03%	0.55%	0.65%	0.74%	0.43%	0.12%	0.00%	2.52%	12.38
NW	0.34%	0.77%	1.05%	3.11%	1.51%	0.86%	0.31%	7.94%	15.03
N-NW	0.09%	1.08%	2.92%	3.60%	2.92%	1.63%	0.95%	13.20%	16.05
CALM	0.28%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.28%	0.00
Total	3.35%	13.42%	25.05%	30.37%	18.22%	7.63%	1.69%	100.00%	13.46

NOTES:

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

2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-260
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
February 2001 – 2006

CP COL 2.3(1)

February	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.24%	1.04%	3.42%	4.14%	2.05%	1.55%	0.36%	12.80%	14.54
N-NE	0.33%	1.16%	2.44%	2.05%	1.01%	0.30%	0.03%	7.32%	11.90
NE	0.24%	1.04%	1.43%	0.74%	0.12%	0.00%	0.00%	3.57%	8.91
E-NE	0.33%	0.83%	1.58%	0.45%	0.00%	0.00%	0.00%	3.18%	8.35
E	0.21%	1.31%	1.07%	0.03%	0.00%	0.00%	0.00%	2.62%	6.54
E-SE	0.12%	2.83%	1.46%	0.30%	0.06%	0.00%	0.00%	4.76%	7.05
SE	0.15%	1.34%	3.69%	2.92%	1.07%	0.15%	0.06%	9.38%	11.84
S-SE	0.21%	0.77%	2.56%	4.11%	2.50%	1.31%	0.27%	11.73%	15.25
S	0.00%	0.51%	2.26%	3.66%	3.36%	1.10%	0.45%	11.34%	16.32
S-SW	0.12%	0.54%	0.77%	1.61%	1.79%	0.54%	0.06%	5.42%	15.34
SW	0.12%	0.33%	0.68%	0.65%	0.51%	0.09%	0.03%	2.41%	12.95
W-SW	0.12%	0.39%	0.48%	0.36%	0.36%	0.15%	0.09%	1.93%	13.01
W	0.09%	0.51%	0.27%	0.21%	0.12%	0.12%	0.00%	1.31%	10.72
W-NW	0.06%	0.48%	0.48%	0.68%	0.39%	0.09%	0.00%	2.17%	12.29
NW	0.12%	0.51%	1.22%	1.67%	1.10%	0.48%	0.27%	5.36%	14.85
N-NW	0.15%	0.89%	2.11%	4.17%	2.77%	2.20%	1.79%	14.08%	17.49
CALM	0.63%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.63%	0.00
Total	2.59%	14.46%	25.92%	27.74%	17.20%	8.07%	3.39%	100.00%	13.63

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-261
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
March 2001 – 2006

CP COL 2.3(1)

March	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.14%	1.08%	2.82%	2.44%	1.33%	0.41%	0.27%	8.48%	13.25
N-NE	0.30%	1.00%	2.95%	2.82%	0.84%	0.46%	0.03%	8.40%	12.34
NE	0.14%	0.68%	2.30%	2.19%	0.38%	0.16%	0.00%	5.85%	11.52
E-NE	0.19%	0.95%	1.79%	1.00%	0.24%	0.00%	0.03%	4.20%	9.85
E	0.08%	1.06%	1.60%	0.41%	0.03%	0.00%	0.00%	3.17%	8.47
E-SE	0.14%	1.60%	1.92%	0.30%	0.03%	0.00%	0.00%	3.98%	7.90
SE	0.11%	1.16%	2.68%	3.47%	0.70%	0.05%	0.05%	8.24%	11.94
S-SE	0.08%	1.11%	3.58%	4.85%	4.15%	1.46%	0.35%	15.58%	15.22
S	0.08%	1.06%	2.52%	3.68%	5.15%	2.52%	0.76%	15.77%	17.06
S-SW	0.03%	0.51%	0.68%	1.52%	1.27%	0.60%	0.08%	4.69%	15.52
SW	0.14%	0.19%	0.84%	0.87%	0.30%	0.08%	0.03%	2.44%	12.43
W-SW	0.11%	0.22%	0.60%	0.84%	0.76%	0.38%	0.16%	3.06%	15.92
W	0.08%	0.19%	0.33%	0.41%	0.27%	0.00%	0.03%	1.30%	12.45
W-NW	0.05%	0.27%	0.33%	0.51%	0.14%	0.05%	0.08%	1.44%	13.23
NW	0.05%	0.33%	0.81%	1.11%	0.62%	0.49%	0.22%	3.63%	15.56
N-NW	0.08%	0.81%	1.84%	2.66%	1.92%	1.84%	0.33%	9.48%	16.22
CALM	0.30%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.30%	0.00
Total	1.79%	12.22%	27.58%	29.07%	18.13%	8.51%	2.41%	100.00%	13.85

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-262
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
April 2001 – 2006

CP COL 2.3(1)

April	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.11%	0.55%	1.24%	2.04%	1.41%	0.40%	0.14%	5.89%	14.48
N-NE	0.09%	0.55%	1.52%	1.64%	0.52%	0.32%	0.11%	4.74%	13.39
NE	0.20%	0.57%	1.21%	0.78%	0.34%	0.00%	0.00%	3.10%	10.37
E-NE	0.11%	0.60%	1.24%	0.46%	0.06%	0.00%	0.00%	2.47%	9.03
E	0.17%	0.98%	1.21%	0.29%	0.00%	0.00%	0.00%	2.64%	7.82
E-SE	0.17%	2.01%	2.01%	0.26%	0.06%	0.00%	0.00%	4.51%	7.70
SE	0.06%	0.98%	2.56%	4.37%	1.21%	0.06%	0.03%	9.25%	12.89
S-SE	0.06%	1.18%	3.19%	8.47%	6.55%	2.30%	1.06%	22.81%	16.54
S	0.11%	0.75%	2.87%	6.29%	7.87%	5.11%	0.80%	23.81%	17.86
S-SW	0.20%	0.23%	1.38%	1.81%	1.58%	0.83%	0.03%	6.06%	15.45
SW	0.09%	0.26%	0.89%	0.57%	0.32%	0.03%	0.00%	2.15%	11.79
W-SW	0.03%	0.23%	0.55%	0.57%	0.52%	0.20%	0.03%	2.13%	14.65
W	0.06%	0.29%	0.49%	0.34%	0.17%	0.11%	0.00%	1.47%	12.17
W-NW	0.14%	0.29%	0.09%	0.34%	0.32%	0.09%	0.03%	1.29%	13.06
NW	0.00%	0.11%	0.78%	0.55%	0.55%	0.14%	0.00%	2.13%	14.07
N-NW	0.03%	0.46%	1.29%	1.58%	0.63%	0.78%	0.46%	5.23%	15.99
CALM	0.32%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.32%	0.00
Total	1.64%	10.03%	22.49%	30.36%	22.09%	10.37	2.70%	100.00%	14.79

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-263
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
May 2001 – 2006

CP COL 2.3(1)

May	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.19%	0.65%	1.67%	0.94%	0.75%	0.35%	0.16%	4.71%	13.37
N-NE	0.11%	1.02%	1.59%	1.37%	0.48%	0.16%	0.11%	4.84%	12.14
NE	0.22%	0.94%	1.99%	1.29%	0.30%	0.03%	0.03%	4.79%	10.70
E-NE	0.16%	1.10%	1.53%	0.78%	0.13%	0.11%	0.00%	3.82%	9.77
E	0.11%	0.89%	1.53%	0.38%	0.11%	0.00%	0.00%	3.01%	8.69
E-SE	0.22%	1.29%	2.34%	0.43%	0.00%	0.00%	0.05%	4.33%	8.42
SE	0.27%	1.05%	2.74%	4.09%	1.56%	0.11%	0.03%	9.84%	12.76
S-SE	0.30%	1.02%	2.72%	8.69%	7.48%	2.53%	0.13%	22.86%	16.07
S	0.11%	0.59%	2.58%	7.56%	7.80%	3.66%	0.35%	22.64%	17.30
S-SW	0.03%	0.30%	1.10%	2.34%	1.67%	0.73%	0.00%	6.16%	15.59
SW	0.13%	0.40%	0.67%	0.81%	0.56%	0.19%	0.03%	2.80%	13.33
W-SW	0.00%	0.27%	0.43%	0.30%	0.30%	0.03%	0.00%	1.32%	11.99
W	0.08%	0.22%	0.27%	0.16%	0.05%	0.00%	0.00%	0.78%	8.86
W-NW	0.00%	0.22%	0.16%	0.03%	0.05%	0.00%	0.24%	0.70%	30.90
NW	0.05%	0.48%	0.43%	0.32%	0.22%	0.00%	0.00%	1.51%	10.37
N-NW	0.03%	0.48%	0.99%	0.70%	0.99%	0.62%	0.32%	4.14%	15.87
CALM	1.77%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	1.77%	0.00
Total	1.99%	10.92%	22.75%	30.17%	22.45%	8.50%	1.45%	100.00%	14.17

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-264
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
June 2001 – 2006

CP COL 2.3(1)

June	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.17%	0.78%	1.54%	1.40%	0.25%	0.08%	0.00%	4.23%	10.73
N-NE	0.14%	1.48%	2.44%	1.68%	0.50%	0.22%	0.06%	6.52%	10.74
NE	0.34%	1.76%	1.96%	1.23%	0.25%	0.08%	0.03%	5.66%	9.45
E-NE	0.25%	1.37%	1.85%	0.70%	0.08%	0.00%	0.00%	4.26%	8.63
E	0.25%	1.71%	1.62%	0.31%	0.03%	0.00%	0.00%	3.92%	7.48
E-SE	0.22%	2.88%	3.22%	0.64%	0.03%	0.03%	0.00%	7.03%	7.74
SE	0.28%	2.63%	6.33%	7.06%	1.85%	0.14%	0.00%	18.29%	11.84
S-SE	0.11%	2.55%	6.64%	8.49%	4.20%	1.43%	0.36%	23.77%	13.79
S	0.22%	1.34%	3.50%	4.76%	3.50%	1.60%	0.17%	15.09%	14.57
S-SW	0.08%	0.78%	1.51%	1.09%	0.59%	0.11%	0.00%	4.17%	11.78
SW	0.17%	0.36%	0.36%	0.17%	0.06%	0.03%	0.03%	1.18%	9.05
W-SW	0.11%	0.34%	0.50%	0.17%	0.06%	0.06%	0.00%	1.23%	9.10
W	0.11%	0.25%	0.25%	0.08%	0.00%	0.00%	0.00%	0.70%	7.30
W-NW	0.08%	0.28%	0.28%	0.11%	0.00%	0.00%	0.00%	0.76%	7.55
NW	0.20%	0.14%	0.39%	0.06%	0.03%	0.06%	0.00%	0.87%	8.26
N-NW	0.25%	0.67%	0.56%	0.31%	0.22%	0.06%	0.00%	2.07%	9.36
CALM	0.25%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.25%	0.00
Total	3.00%	19.35%	32.96%	28.26%	11.65%	3.89%	0.64%	100.00%	11.62

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-265
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
July 2001 – 2006

CP COL 2.3(1)

July	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.05%	0.40%	0.81%	0.57%	0.13%	0.00%	0.00%	1.97%	10.41
N-NE	0.19%	0.35%	0.57%	0.70%	0.11%	0.00%	0.00%	1.91%	10.27
NE	0.05%	0.75%	0.73%	0.70%	0.00%	0.00%	0.00%	2.24%	9.17
E-NE	0.22%	0.92%	0.27%	0.43%	0.05%	0.00%	0.00%	1.89%	7.73
E	0.30%	0.73%	1.08%	0.13%	0.03%	0.00%	0.00%	2.26%	7.35
E-SE	0.13%	1.83%	2.32%	0.38%	0.03%	0.00%	0.00%	4.69%	7.84
SE	0.16%	1.70%	5.20%	5.47%	0.92%	0.00%	0.00%	13.44%	11.53
S-SE	0.30%	2.18%	7.84%	9.61%	2.99%	0.24%	0.03%	23.19%	12.58
S	0.22%	1.70%	7.68%	9.94%	3.12%	0.30%	0.00%	22.95%	12.76
S-SW	0.30%	1.48%	3.69%	5.84%	1.70%	0.08%	0.00%	13.09%	12.45
SW	0.19%	1.29%	2.07%	2.18%	0.65%	0.00%	0.00%	6.38%	11.05
W-SW	0.16%	0.48%	1.40%	0.67%	0.11%	0.00%	0.00%	2.83%	9.85
W	0.11%	0.40%	0.51%	0.11%	0.00%	0.00%	0.00%	1.13%	7.53
W-NW	0.03%	0.16%	0.19%	0.00%	0.03%	0.00%	0.00%	0.40%	8.07
NW	0.05%	0.13%	0.24%	0.00%	0.03%	0.00%	0.00%	0.46%	7.54
N-NW	0.11%	0.19%	0.43%	0.19%	0.03%	0.00%	0.00%	0.94%	8.57
CALM	0.24%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.24%	0.00
Total	2.56%	14.71%	35.01%	36.92%	9.91%	0.62%	0.03%	100.00%	11.53

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-266
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
August 2001 – 2006

CP COL 2.3(1)

August	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.27%	0.99%	0.59%	0.27%	0.00%	0.05%	0.00%	2.18%	7.44
N-NE	0.24%	1.24%	0.99%	0.62%	0.13%	0.08%	0.03%	3.33%	8.97
NE	0.40%	1.75%	1.26%	0.75%	0.03%	0.08%	0.00%	4.27%	8.08
E-NE	0.43%	2.12%	1.91%	0.97%	0.13%	0.05%	0.00%	5.62%	8.30
E	0.32%	2.12%	1.96%	0.48%	0.05%	0.00%	0.00%	4.95%	7.63
E-SE	0.22%	3.68%	3.82%	0.38%	0.03%	0.00%	0.00%	8.12%	7.38
SE	0.24%	2.23%	4.70%	3.76%	0.97%	0.08%	0.00%	11.99%	10.96
S-SE	0.27%	2.58%	6.05%	7.53%	2.15%	0.13%	0.00%	18.71%	12.10
S	0.30%	1.77%	5.67%	9.09%	3.31%	0.08%	0.03%	20.24%	12.95
S-SW	0.24%	1.42%	3.15%	3.92%	0.81%	0.00%	0.00%	9.54%	11.49
SW	0.19%	0.94%	1.51%	0.86%	0.16%	0.00%	0.00%	3.66%	9.60
W-SW	0.19%	0.86%	0.59%	0.16%	0.05%	0.00%	0.00%	1.85%	7.38
W	0.16%	0.65%	0.32%	0.00%	0.00%	0.00%	0.00%	1.13%	5.60
W-NW	0.08%	0.59%	0.40%	0.03%	0.00%	0.00%	0.00%	1.10%	6.48
NW	0.19%	0.48%	0.32%	0.05%	0.05%	0.00%	0.00%	1.10%	6.67
N-NW	0.16%	0.81%	0.54%	0.19%	0.05%	0.00%	0.00%	1.75%	7.44
CALM	0.46%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.46%	0.00
Total	3.90%	24.25%	33.79%	29.06%	7.93%	0.56%	0.05%	100.00%	10.37

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-267
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
September 2001 – 2006

CP COL 2.3(1)

September	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.25%	1.62%	2.70%	2.17%	1.17%	0.03%	0.00%	7.94%	11.21
N-NE	0.42%	2.03%	3.51%	1.73%	1.25%	0.08%	0.00%	9.02%	10.64
NE	0.64%	1.67%	2.37%	2.01%	0.14%	0.03%	0.00%	6.85%	9.36
E-NE	0.53%	2.53%	2.53%	1.20%	0.22%	0.00%	0.00%	7.02%	8.36
E	0.53%	1.78%	2.53%	0.78%	0.11%	0.03%	0.00%	5.76%	8.20
E-SE	0.31%	3.48%	3.73%	0.14%	0.00%	0.00%	0.00%	7.66%	7.11
SE	0.36%	2.92%	5.76%	4.68%	0.67%	0.06%	0.00%	14.45%	10.54
S-SE	0.33%	2.26%	5.76%	5.04%	1.45%	0.25%	0.03%	15.12%	11.52
S	0.36%	1.75%	2.98%	2.98%	2.01%	0.72%	0.03%	10.83%	12.76
S-SW	0.22%	0.72%	1.45%	1.06%	0.67%	0.19%	0.00%	4.32%	11.76
SW	0.31%	0.61%	0.75%	0.31%	0.17%	0.00%	0.00%	2.14%	8.58
W-SW	0.14%	0.19%	0.33%	0.25%	0.00%	0.00%	0.00%	0.92%	8.72
W	0.08%	0.22%	0.08%	0.03%	0.00%	0.00%	0.00%	0.42%	5.79
W-NW	0.08%	0.39%	0.25%	0.11%	0.00%	0.00%	0.00%	0.84%	7.29
NW	0.19%	0.28%	0.50%	0.72%	0.36%	0.06%	0.06%	2.17%	12.37
N-NW	0.17%	0.84%	1.11%	1.00%	0.58%	0.03%	0.00%	3.73%	11.22
CALM	0.81%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.81%	0.00
Total	4.93%	23.31%	36.37%	24.20%	8.80%	1.48%	0.11%	100.00%	10.29

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-268
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
October 2001 – 2006

CP COL 2.3(1)

October	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.32%	1.18%	1.80%	2.02%	1.45%	0.46%	0.13%	7.37%	12.99
N-NE	0.35%	0.99%	2.45%	1.99%	0.48%	0.13%	0.11%	6.51%	11.08
NE	0.43%	1.26%	1.86%	0.62%	0.19%	0.00%	0.00%	4.36%	8.73
E-NE	0.24%	1.16%	1.37%	0.46%	0.08%	0.00%	0.00%	3.31%	8.09
E	0.40%	1.37%	1.96%	0.13%	0.00%	0.00%	0.00%	3.87%	7.19
E-SE	0.40%	2.53%	1.91%	0.05%	0.00%	0.00%	0.00%	4.89%	6.36
SE	0.13%	2.26%	4.33%	3.15%	0.38%	0.03%	0.00%	10.27%	10.31
S-SE	0.16%	1.91%	4.84%	7.37%	2.55%	0.19%	0.00%	17.02%	12.76
S	0.24%	0.94%	3.31%	5.22%	3.85%	0.78%	0.00%	14.33%	14.39
S-SW	0.13%	1.18%	1.51%	2.37%	1.29%	0.32%	0.00%	6.80%	12.68
SW	0.13%	0.59%	1.26%	1.75%	0.56%	0.05%	0.00%	4.36%	12.11
W-SW	0.16%	0.35%	0.91%	0.56%	0.46%	0.05%	0.00%	2.50%	11.70
W	0.13%	0.38%	0.83%	0.46%	0.08%	0.11%	0.00%	1.99%	10.41
W-NW	0.05%	0.32%	0.54%	0.19%	0.13%	0.03%	0.11%	1.37%	11.76
NW	0.13%	0.56%	0.89%	1.56%	0.81%	0.13%	0.13%	4.22%	13.64
N-NW	0.51%	0.73%	1.64%	1.48%	1.13%	0.56%	0.32%	6.37%	13.56
CALM	0.46%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.46%	0.00
Total	3.95%	17.72%	31.41%	29.36%	13.44%	2.85%	0.81%	100.00%	11.70

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-269
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
November 2001 – 2006

CP COL 2.3(1)

November	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.17%	0.81%	3.29%	3.36%	1.29%	0.17%	0.00%	9.08%	12.58
N-NE	0.17%	1.29%	2.00%	1.22%	0.44%	0.07%	0.00%	5.19%	10.26
NE	0.37%	0.61%	1.15%	0.61%	0.20%	0.07%	0.00%	3.02%	9.73
E-NE	0.27%	0.68%	1.29%	0.71%	0.07%	0.03%	0.00%	3.05%	9.60
E	0.20%	1.56%	2.44%	0.20%	0.00%	0.00%	0.00%	4.41%	7.73
E-SE	0.14%	1.49%	1.53%	0.51%	0.00%	0.00%	0.00%	3.66%	7.83
SE	0.07%	1.25%	3.15%	2.95%	0.68%	0.00%	0.00%	8.10%	11.44
S-SE	0.20%	1.22%	3.53%	6.44%	3.19%	0.24%	0.07%	14.88%	13.67
S	0.10%	0.71%	2.71%	4.85%	3.02%	0.47%	0.03%	11.90%	14.48
S-SW	0.14%	0.75%	1.76%	1.59%	1.63%	0.41%	0.14%	6.41%	13.81
SW	0.24%	0.95%	0.68%	1.12%	1.02%	0.17%	0.00%	4.17%	12.53
W-SW	0.14%	0.47%	1.29%	0.71%	0.64%	0.44%	0.07%	3.76%	13.21
W	0.14%	0.31%	0.68%	0.34%	0.20%	0.14%	0.03%	1.83%	11.73
W-NW	0.07%	0.37%	0.34%	0.75%	0.58%	0.03%	0.03%	2.17%	13.34
NW	0.07%	0.47%	0.98%	1.66%	0.95%	0.92%	0.71%	5.76%	17.30
N-NW	0.17%	1.08%	2.54%	4.34%	2.61%	1.32%	0.44%	12.51%	15.22
CALM	0.10%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.10%	0.00
Total	2.64%	14.03%	29.36%	31.36%	16.51%	4.47%	1.53%	100.00%	12.88

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yrs (2001 – 2004, 2006).

Table 2.3-270
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
December 2001 – 2006

CP COL 2.3(1)

December	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.34%	0.88%	2.67%	3.04%	2.06%	0.61%	0.00%	9.58%	13.43
N-NE	0.13%	0.47%	1.89%	1.75%	0.61%	0.17%	0.07%	5.10%	12.56
NE	0.07%	0.17%	0.81%	0.13%	0.00%	0.03%	0.00%	1.21%	9.16
E-NE	0.13%	0.34%	0.37%	0.24%	0.00%	0.00%	0.00%	1.08%	8.11
E	0.10%	0.24%	0.71%	0.07%	0.00%	0.00%	0.00%	1.11%	8.20
E-SE	0.13%	0.81%	0.57%	0.17%	0.07%	0.00%	0.00%	1.75%	7.32
SE	0.10%	1.01%	1.55%	1.52%	0.44%	0.17%	0.00%	4.79%	11.62
S-SE	0.20%	1.01%	3.44%	5.20%	2.26%	0.78%	0.00%	12.89%	13.84
S	0.17%	1.65%	4.72%	5.50%	2.46%	0.78%	0.03%	15.32%	13.12
S-SW	0.17%	1.42%	2.97%	3.14%	1.45%	0.78%	0.24%	10.16%	13.43
SW	0.24%	1.11%	1.75%	1.65%	0.51%	0.17%	0.20%	5.64%	11.99
W-SW	0.07%	1.01%	1.05%	0.91%	0.71%	0.10%	0.03%	3.88%	11.65
W	0.07%	0.37%	0.54%	0.61%	0.88%	0.20%	0.00%	2.67%	14.32
W-NW	0.03%	0.40%	0.84%	1.28%	0.57%	0.03%	0.00%	3.17%	12.78
NW	0.13%	0.84%	2.13%	1.65%	0.94%	0.71%	0.20%	6.61%	13.78
N-NW	0.24%	1.32%	1.65%	3.21%	2.13%	2.36%	0.47%	11.37%	15.91
CALM	3.64%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	3.64%	0.00
Total	2.33%	13.06%	27.67%	30.07%	15.09%	6.88%	1.25%	100.00%	12.72

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-271
Percentage Frequency of Wind Direction and Speed (MPH)
CPNPP, Upper Level
Annual 2001 – 2006

CP COL 2.3(1)

All Months	Wind Speed (mph)								
	0-3	4-7	8-12	13-17	18-22	23-27	≥28		
Direction From	Frequency of Occurrence (%)							Total (%)	Avg. Speed
N	0.20%	0.91%	2.07%	2.08%	1.10%	0.37%	0.10%	6.83%	12.92
N-NE	0.23%	1.04%	2.00%	1.61%	0.55%	0.17%	0.05%	5.65%	11.36
NE	0.28%	0.99%	1.50%	1.01%	0.18%	0.04%	0.00%	4.01%	9.68
E-NE	0.26%	1.13%	1.41%	0.67%	0.10%	0.02%	0.00%	3.58%	8.76
E	0.25%	1.23%	1.57%	0.29%	0.03%	0.00%	0.00%	3.38%	7.73
E-SE	0.20%	2.17%	2.22%	0.32%	0.02%	0.00%	0.00%	4.94%	7.49
SE	0.18%	1.68%	3.86%	3.90%	0.93%	0.07%	0.01%	10.63%	11.51
S-SE	0.20%	1.61%	4.47%	6.71%	3.51%	0.94%	0.19%	17.63%	13.95
S	0.17%	1.16%	3.62%	5.68%	4.01%	1.53%	0.23%	16.40%	14.97
S-SW	0.15%	0.82%	1.79%	2.43%	1.36%	0.47%	0.05%	7.07%	13.58
SW	0.18%	0.65%	1.06%	1.02%	0.52%	0.11%	0.03%	3.56%	11.87
W-SW	0.12%	0.45%	0.74%	0.53%	0.45%	0.22%	0.04%	2.54%	12.71
W	0.11%	0.36%	0.41%	0.26%	0.23%	0.07%	0.00%	1.44%	11.10
W-NW	0.06%	0.36%	0.37%	0.37%	0.20%	0.04%	0.04%	1.44%	12.43
NW	0.13%	0.42%	0.78%	0.99%	0.57%	0.30%	0.15%	3.34%	14.13
N-NW	0.17%	0.76%	1.43%	1.86%	1.27%	0.91%	0.41%	6.80%	15.27
CALM	0.74%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.74%	0.00
Total	2.90%	15.73%	29.30%	29.74%	15.03%	5.24%	1.31%	100.00%	12.56

NOTES:

1. Calm is classified as a wind speed less than 1 mph.
2. Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-272
Maximum Number of Consecutive Hours With Wind From a Single Sector
Dallas Fort Worth Airport

CP COL 2.3(1)

Sector	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	21	11	13	13	17	21	26	21	14	23	26	18.0
NNE	11	14	10	15	12	10	5	9	9	10	15	10.5
NE	7	8	5	7	4	5	4	5	9	7	9	6.1
ENE	7	6	4	10	7	8	5	4	11	5	11	6.7
E	6	8	11	13	10	11	9	9	10	10	13	9.7
ESE	6	7	6	7	8	7	8	11	6	11	11	7.7
SE	13	12	10	8	8	13	9	9	7	10	13	9.9
SSE	11	14	10	20	11	10	17	19	11	13	20	13.6
S	31	24	39	36	52	40	48	28	31	43	52	37.2
SSW	9	8	9	9	6	10	13	5	10	7	13	8.6
SW	4	6	5	5	4	3	7	5	3	6	7	4.8
WSW	5	6	8	6	7	4	4	5	4	9	9	5.8
W	8	8	8	7	6	6	8	12	12	10	12	8.5
WNW	5	8	8	7	6	7	9	6	10	8	10	7.4
NW	15	12	9	10	8	10	12	9	9	10	15	10.4
NNW	21	12	9	15	9	20	22	15	20	10	22	15.3

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-273
Maximum Number of Consecutive Hours With Wind From Three Adjacent Sectors
Dallas Fort Worth Airport

CP COL 2.3(1)

Sector	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	34	56	44	42	49	56	75	54	41	66	75	51.7
NNE	40	35	43	48	71	44	29	38	51	66	71	46.5
NE	21	41	18	18	25	26	21	13	26	16	41	22.5
ENE	28	28	24	28	24	40	15	28	35	29	40	27.9
E	29	27	32	31	27	41	16	23	28	26	41	28.0
ESE	25	27	39	23	27	44	33	33	36	39	44	32.6
SE	44	32	23	38	31	27	29	40	39	45	45	34.8
SSE	80	110	110	86	106	89	73	152	96	158	158	106.0
S	80	114	99	95	93	105	92	131	84	96	131	98.9
SSW	59	79	52	55	57	57	48	44	64	69	79	58.4
SW	16	15	23	12	16	16	29	27	19	15	29	18.8
WSW	17	21	18	15	28	16	12	21	20	12	28	18.0
W	23	30	21	13	22	17	17	27	23	21	30	21.4
WNW	47	28	36	24	17	37	34	37	38	22	47	32.0
NW	35	49	41	24	31	38	34	35	38	30	49	35.5
NNW	67	47	51	41	62	53	50	49	46	39	67	50.5

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-274
Maximum Number of Consecutive Hours With Wind From Five Adjacent Sectors
Dallas Fort Worth Airport

CP COL 2.3(1)

Sector	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	67	64	51	83	74	69	88	72	87	66	88	72.1
NNE	50	72	48	84	71	67	80	58	63	66	84	65.9
NE	59	73	55	74	72	74	39	47	70	66	74	62.9
ENE	51	86	49	45	46	71	27	34	36	36	86	48.1
E	38	39	39	44	41	62	34	33	41	39	62	41.0
ESE	58	50	72	38	60	68	38	43	42	45	72	51.4
SE	91	110	110	131	132	125	110	152	96	170	170	122.7
SSE	96	140	140	156	156	185	123	232	136	304	304	166.8
S	105	210	145	138	168	152	92	233	124	274	274	164.1
SSW	81	236	181	136	119	128	107	131	89	96	236	130.4
SW	67	81	54	55	57	59	48	44	72	69	81	60.6
WSW	28	30	31	21	41	22	29	27	27	30	41	28.6
W	49	31	50	29	28	37	35	37	47	27	50	37.0
WNW	57	72	51	51	40	44	39	50	54	34	72	49.2
NW	85	62	56	57	65	53	55	61	49	58	85	60.1
NNW	74	75	56	49	76	57	82	73	57	66	82	66.5

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-275
Maximum Number of Consecutive Hours With Wind From a Single Sector
Mineral Wells Airport

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	18	22	25	16	17	14	25	18.7
NNE	8	8	9	6	5	5	9	6.8
NE	7	6	3	8	4	6	8	5.7
ENE	7	4	8	9	8	5	9	6.8
E	8	12	5	7	9	8	12	8.2
ESE	8	9	7	5	4	8	9	6.8
SE	18	8	15	13	8	10	18	12.0
SSE	18	17	15	15	13	16	18	15.7
S	14	17	17	24	15	17	24	17.3
SSW	5	7	6	7	5	7	7	6.2
SW	5	3	5	7	6	4	7	5.0
WSW	3	3	5	4	5	3	5	3.8
W	10	11	7	8	8	10	11	9.0
WNW	5	4	4	10	8	5	10	6.0
NW	5	7	10	10	9	9	10	8.3
NNW	10	10	10	9	15	13	15	11.2

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.
3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-276
Maximum Number of Consecutive Hours With Wind From Three Adjacent Sectors
Mineral Wells Airport

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	54	35	50	47	48	37	54	45.2
NNE	29	28	26	31	24	30	31	28.0
NE	17	23	18	20	28	21	28	21.2
ENE	20	23	13	24	24	22	24	21.0
E	30	52	13	16	21	14	52	24.3
ESE	34	37	21	27	17	30	37	27.7
SE	48	39	39	45	49	58	58	46.3
SSE	83	118	95	124	90	107	124	102.8
S	104	84	89	100	71	107	107	92.5
SSW	28	22	28	24	24	26	28	25.3
SW	8	11	12	17	11	10	17	11.5
WSW	24	12	17	16	12	24	24	17.5
W	16	14	19	16	21	24	24	18.3
WNW	20	27	18	19	26	20	27	21.7
NW	27	23	37	34	29	28	37	29.7
NNW	38	55	41	37	40	29	55	40.0

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-277
Maximum Number of Consecutive Hours With Wind From Five Adjacent Sectors
Mineral Wells Airport

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2005	2006	Maximum	Avg.
N	65	55	70	72	51	39	72	58.7
NNE	54	42	52	55	64	66	66	55.5
NE	43	35	29	40	37	56	56	40.0
ENE	34	56	27	26	29	30	56	33.7
E	46	52	23	29	35	36	52	36.8
ESE	49	52	48	49	54	63	63	52.5
SE	90	137	95	138	100	129	138	114.8
SSE	140	184	98	169	175	216	216	114.8
S	142	179	109	211	109	194	211	157.3
SSW	104	84	91	110	71	134	134	99.0
SW	31	31	31	31	24	46	46	32.3
WSW	29	23	23	28	24	26	29	25.5
W	37	29	31	20	35	40	40	32.0
WNW	32	31	37	53	45	49	53	41.2
NW	46	57	41	63	56	48	63	51.8
NNW	66	56	70	48	51	38	70	54.8

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-278 Maximum Number of Consecutive Hours With Wind From a Single Sector CPNPP, Lower Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	18	19	21	27	41	41	25.2
NNE	15	13	12	12	11	15	12.6
NE	14	6	7	6	10	14	8.6
ENE	8	10	8	9	12	10	9.4
E	9	8	9	8	11	9	9.0
ESE	8	8	10	12	11	12	9.8
SE	11	19	20	19	14	20	16.6
SSE	21	24	18	15	17	24	19.0
S	23	31	26	19	13	31	22.4
SSW	9	11	11	12	13	12	11.2
SW	10	13	10	10	9	13	10.4
WSW	10	6	10	9	11	10	9.2
W	4	4	5	5	5	5	4.6
WNW	8	10	8	8	8	10	8.4
NW	13	20	16	15	12	20	15.2
NNW	21	17	32	18	14	32	20.4

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.
2. CPNPP SITE DATA, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-279
Maximum Number of Consecutive Hours With Wind From Three Adjacent Sectors
CPNPP, Lower Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	56	68	70	53	65	70	62.4
NNE	69	49	32	43	46	69	47.8
NE	38	25	21	21	28	38	26.6
ENE	26	25	18	21	28	26	23.6
E	38	26	15	25	29	38	26.6
ESE	35	55	43	48	37	55	43.6
SE	60	114	49	104	70	114	79.4
SSE	83	127	106	146	120	146	116.4
S	116	124	90	92	176	124	119.6
SSW	48	39	34	28	39	48	37.6
SW	29	25	26	37	33	37	30.0
WSW	29	18	19	18	29	29	22.6
W	32	26	13	31	16	32	23.6
WNW	19	26	25	23	21	26	22.8
NW	34	39	36	43	51	43	40.6
NNW	60	61	79	44	48	79	58.4

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.
2. CPNPP Data, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-280
Maximum Number of Consecutive Hours With Wind From Five Adjacent Sectors
CPNPP, Lower Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	79	69	81	71	68	81	73.6
NNE	69	69	70	75	68	75	70.2
NE	90	57	41	73	55	90	63.2
ENE	72	45	35	45	53	72	50.0
E	49	65	55	69	44	69	56.4
ESE	72	114	61	117	74	117	87.6
SE	134	143	157	148	120	157	140.4
SSE	169	203	162	146	321	203	200.2
S	155	253	106	146	279	253	187.8
SSW	116	127	104	92	176	127	123.0
SW	49	59	49	54	48	59	51.8
WSW	42	29	36	48	38	48	38.6
W	47	37	35	35	40	47	38.8
WNW	34	48	43	61	53	61	47.8
NW	60	61	79	45	61	79	61.2
NNW	75	69	79	70	65	79	71.6

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.
2. CPNPP Data, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-281
Maximum Number of Consecutive Hours With Wind From a Single Sector
CPNPP, Upper Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	16	23	31	16	23	31	21.8
NNE	15	17	11	14	15	17	14.4
NE	10	9	9	9	8	10	9.0
ENE	9	9	9	6	12	9	9.0
E	7	11	7	9	11	11	9.0
ESE	11	11	9	15	9	15	11.0
SE	17	15	20	18	16	20	17.2
SSE	31	18	19	16	30	31	22.8
S	21	20	18	22	19	22	20.0
SSW	17	12	10	12	14	17	13.0
SW	10	9	13	12	11	13	11.0
WSW	11	9	13	14	9	14	11.2
W	5	6	12	6	5	12	6.8
WNW	6	7	6	6	6	7	6.2
NW	15	18	12	16	17	18	15.6
NNW	24	40	16	19	22	40	24.2

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.

2. CPNPP SITE DATA, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-282
Maximum Number of Consecutive Hours With Wind From Three Adjacent Sectors
CPNPP, Upper Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	56	68	63	70	65	70	64.4
NNE	52	60	50	45	44	60	50.2
NE	42	58	34	30	28	58	38.4
ENE	26	25	20	25	22	26	23.6
E	41	34	17	32	29	41	30.6
ESE	35	56	21	48	37	56	39.4
SE	71	40	57	103	68	103	67.8
SSE	122	121	136	122	118	136	123.8
S	99	162	88	204	184	204	147.4
SSW	30	105	52	40	73	105	60.0
SW	28	34	35	36	38	36	34.2
WSW	32	25	26	37	29	37	29.8
W	20	19	17	26	17	26	19.8
WNW	18	18	14	26	27	26	20.6
NW	37	47	31	55	35	55	41.0
NNW	61	58	44	43	58	61	52.8

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.
2. CPNPP Data, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-283
Maximum Number of Consecutive Hours With Wind From Five Adjacent Sectors
CPNPP, Upper Level

CP COL 2.3(1)

Sector	2001	2002	2003	2004	2006	Maximum	Avg.
N	81	75	65	88	68	88	75.4
NNE	72	102	66	84	68	102	78.4
NE	69	98	61	73	47	98	69.6
ENE	67	58	41	76	53	76	59.0
E	49	65	31	69	67	69	56.2
ESE	89	68	60	114	69	114	80.0
SE	153	146	136	123	118	153	135.2
SSE	169	204	161	240	321	240	219.0
S	163	256	167	243	283	256	222.4
SSW	123	162	104	204	184	204	155.4
SW	49	106	63	62	76	106	71.2
WSW	37	38	42	57	42	57	43.2
W	46	32	43	47	36	47	40.8
WNW	37	58	36	61	51	61	48.6
NW	61	63	44	67	64	67	59.8
NNW	77	69	65	88	67	88	73.2

NOTES:

1. Wind values which were either not provided or had a zero speed value were not included, and assumed to break any consecutive wind direction count.

2. CPNPP Data, Period of Record – 5 yr (2001 – 2004, 2006).

Table 2.3-284 (Sheet 1 of 2)
Comparison of Average Wind Persistence

CP COL 2.3(1)

Wind Persistence (hrs)

	Single Sector				Three Adjacent Sectors				Five Adjacent Sectors			
Sector	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth
N	25.2	21.8	18.7	18.0	62.4	64.4	45.2	51.7	73.6	75.4	58.7	72.1
NNE	12.6	14.4	6.8	10.5	47.8	50.2	28.0	46.5	70.2	78.4	55.5	65.9
NE	8.6	9.0	5.7	6.1	26.6	38.4	21.2	22.5	63.2	69.6	40.0	62.9
ENE	9.4	9.0	6.8	6.7	23.6	23.6	21.0	27.9	50.0	59.0	33.7	48.1
E	9.0	9.0	8.2	9.7	26.6	30.6	24.3	28.0	56.4	56.2	36.8	41.0
ESE	9.8	11.0	6.8	7.7	43.6	39.4	27.7	32.6	87.6	80.0	52.5	51.4
SE	16.6	17.2	12.0	9.9	79.4	67.8	46.3	34.8	140.4	135.2	114.8	122.7
SSE	19.0	22.8	15.7	13.6	116.4	123.8	102.8	106.0	200.2	219.0	163.7	166.8
S	22.4	20.0	17.3	37.2	119.6	147.4	92.5	98.9	187.8	222.4	157.3	164.1
SSW	11.2	13.0	6.2	8.6	37.6	60.0	25.3	58.4	123.0	155.4	99.0	130.4
SW	10.4	11.0	5.0	4.8	30.0	34.2	11.5	18.8	51.8	71.2	32.3	60.6
WSW	9.2	11.2	3.8	5.8	22.6	29.8	17.5	18.0	38.6	43.2	25.5	28.6
W	4.6	6.8	9.0	8.5	23.6	19.8	18.3	21.4	38.8	40.8	32.0	37.0

Table 2.3-284 (Sheet 2 of 2)
Comparison of Average Wind Persistence

CP COL 2.3(1)

Wind Persistence (hrs)

	Single Sector				Three Adjacent Sectors				Five Adjacent Sectors			
Sector	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth	CPNPP Lower Level	CPNPP Upper Level	Mineral Wells	Fort Worth
WNW	8.4	6.2	6.0	7.4	22.8	20.6	21.7	32.0	47.8	48.6	41.2	49.2
NW	15.2	15.6	8.3	10.4	40.6	41.0	29.7	35.5	61.2	59.8	51.8	60.1
NNW	20.4	24.2	11.2	15.3	58.4	52.8	40.0	50.5	71.6	73.2	54.8	66.5

NOTES:

1. Wind values which were either not provided, had a zero speed value, or a VRB wind direction were not included, and assumed to break any consecutive wind direction count.
2. Wind persistence values above are the average persistence durations for the period of record.
3. Period of record at CPNPP site and Mineral Wells Airport, 2001 – 2004, 2006.
4. Period of record at Mineral Wells Airport, 2001 – 2006.

Table 2.3-285
CPNPP Normal Temperatures

CP COL 2.3(1)

	Daily Minimum	Daily Mean	Daily Maximum
JAN	22.3	49.6	89.0
FEB	19.2	48.9	84.6
MAR	32.9	58.3	93.0
APR	49.4	69.2	100.2
MAY	47.5	75.2	98.9
JUN	65.0	80.3	100.2
JUL	72.7	84.9	103.1
AUG	66.6	85.1	105.0
SEP	56.8	77.4	97.8
OCT	42.3	68.4	93.2
NOV	28.0	58.0	88.0
DEC	18.6	50.8	78.5
Annual	43.4	67.2	94.3

Reference: CPNPP site data 2001-2004 and 2006.

Table 2.3-286
Relative Humidity Dallas Fort Worth Airport
for 4 Time Periods Per Day

CP COL 2.3(1)

1997 – 2006

Time	00:00-06:00	06:00-12:00	12:00-18:00	18:00-24:00
Jan	76%	72%	56%	66%
Feb	78%	74%	58%	67%
Mar	76%	69%	54%	65%
Apr	76%	67%	52%	63%
May	80%	70%	55%	66%
Jun	80%	70%	54%	65%
Jul	72%	62%	44%	55%
Aug	69%	60%	43%	54%
Sep	72%	63%	45%	58%
Oct	77%	69%	52%	65%
Nov	78%	71%	54%	67%
Dec	75%	69%	53%	65%
Annual	76%	68%	52%	63%

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

Table 2.3-287
Relative Humidity Mineral Wells Airport for 4 Time Periods Per Day

CP COL 2.3(1)

2001 – 2005

Time	00:00-06:00	06:00-12:00	12:00-18:00	18:00-24:00
Jan	81%	75%	53%	70%
Feb	83%	76%	55%	72%
Mar	81%	72%	53%	67%
Apr	91%	68%	54%	75%
May	88%	73%	56%	74%
Jun	88%	72%	53%	72%
Jul	83%	64%	44%	64%
Aug	82%	65%	45%	65%
Sep	82%	66%	44%	66%
Oct	86%	73%	53%	75%
Nov	81%	70%	49%	69%
Dec	77%	67%	43%	64%
Annual	84%	70%	50%	69%

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

Table 2.3-288
Monthly Mean and Extreme Maximum and Minimum Dewpoint Temperatures
Mineral Wells

CP COL 2.3(1)

Month	Dewpoint (°F)		
	Mean	Maximum	Minimum
Jan	32	36.3	24.9
Feb	35	39.2	29.7
Mar	39	48.5	33.6
Apr	50	58.2	41.3
May	60	65.6	52.1
Jun	66	68.9	62.2
Jul	67	69.7	62.9
Aug	66	69.0	62.3
Sep	61	67.3	53.3
Oct	52	59.2	45.2
Nov	41	50.2	29.6
Dec	33	41.1	27.0
Annual	50	69.7	24.9

Notes:

1. NCDC Data Mineral Wells AP, WBAN Station ID 93985, Mineral Wells data 1949 – 2006.

Table 2.3-289
Hourly Meteorological Data Dallas Fort Worth Airport Worst 1-Day

CP COL 2.3(1)

MAY 26, 1997

Hour	Dry Bulb Temperature (°F)	Wet Bulb Temperature (°F)
1	78	76
2	78	76
3	78	76
4	77	75
5	77	75
6	76	75
7	78	75
8	80	76
9	83	78
10	86	79
11	88	81
12	89	81
13	90	82
14	92	82
15	92	82
16	91	83
17	89	82
18	88	81
19	86	80
20	84	79
21	83	79
22	83	79
23	81	78
24	80	77
AVERAGE	83.6	78.6

NOTES:

1. The average wet bulb temperature above (78.6°F) is calculated from 24 hourly observations for this date.
2. Period of Record – 10 yr (1997 – 2006).
3. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

Table 2.3-290
Daily Average Meteorological Data
Dallas Fort Worth Airport
Worst 5 Consecutive Day Period

CP COL 2.3(1)

Date	Dry Bulb Temperature (°F)	Wet Bulb Temperature (°F)
6/29/1997	83	77
6/30/1997	84	77
7/1/1997	85	77
7/2/1997	85	77
7/3/1997	86	78
AVERAGE	84.6	77.4

NOTES:

1. Period of Record – 10 yr (1997 – 2006).

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

Table 2.3-291 (Sheet 1 of 2)
Daily Average Meteorological Data Dallas Fort Worth Airport
Worst 30 Consecutive Day Period

CP COL 2.3(1)

Yr	Month	Day	Daily Average Dry Bulb (°F)	Daily Average Wet Bulb (°F)
2001	7	4	85	75
2001	7	5	86	76
2001	7	6	87	77
2001	7	7	86	76
2001	7	8	85	76
2001	7	9	87	76
2001	7	10	87	76
2001	7	11	88	76
2001	7	12	88	75
2001	7	13	89	76
2001	7	14	85	78
2001	7	15	86	78
2001	7	16	88	77
2001	7	17	88	77
2001	7	18	88	76
2001	7	19	87	76
2001	7	20	88	76
2001	7	21	89	76
2001	7	22	89	74
2001	7	23	89	75
2001	7	24	89	77
2001	7	25	88	76
2001	7	26	87	76
2001	7	27	86	76
2001	7	28	87	78
2001	7	29	87	77
2001	7	30	89	76
2001	7	31	88	76
2001	8	1	89	75
2001	8	2	87	76

Table 2.3-291 (Sheet 2 of 2)
Daily Average Meteorological Data Dallas Fort Worth Airport
Worst 30 Consecutive Day Period

CP COL 2.3(1)

			Daily Average	
Yr	Month	Day	Dry Bulb (°F)	Wet Bulb (°F)
Average	-	-	87.4	76.1

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

2. Period of Record – 10 yr (1997 – 2006).

Table 2.3-292
Hourly Meteorological Data Mineral Wells Airport Worst 1-Day

CP COL 2.3(1)

June 24, 2003

Hour	Dry Bulb Temperature (°F)	Wet Bulb Temperature (°F)
1	80	75
2	80	76
3	79	76
4	77	76
5	76	75
6	76	75
7	78	75
8	79	75
9	83	77
10	86	77
11	86	77
12	88	77
13	90	78
14	91	79
15	93	79
16	93	79
17	92	78
18	91	78
19	89	78
20	86	77
21	85	78
22	84	78
23	83	78
24	81	77
AVERAGE	84.4	77.0

NOTES:

1. The average wet bulb temperature above (77.0°F) is calculated from 24 hourly observations for this date.

2. Period of Record – 6 yr (2001 – 2006).

3. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC

Table 2.3-293
Daily Average Meteorological Data
Mineral Wells Airport
Worst 5 Consecutive Day Period

CP COL 2.3(1)

Date	Dry Bulb Temperature (°F)	Wet Bulb Temperature (°F)
6/21/2003	80.6	74.3
6/22/2003	83.2	75.5
6/23/2003	83.8	76.3
6/24/2003	84.4	77.0
6/25/2003	84.5	76.1
AVERAGE	83.3	75.8

NOTES:

1. Period of Record – 6 yr (2001 – 2006).

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC

Table 2.3-294 (Sheet 1 of 2)
Daily Average Meteorological Data
Mineral Wells Airport
Worst 30 Consecutive Day Period

CP COL 2.3(1)

Yr	Month	Day	Daily Average Dry Bulb (°F)	Daily Average Wet Bulb (°F)
2001	7	14	87.2	75.8
2001	7	15	89.5	75.4
2001	7	16	90.8	74.5
2001	7	17	89.2	74.7
2001	7	18	89.8	74.6
2001	7	19	88.3	75.1
2001	7	20	89.2	74.7
2001	7	21	90.9	73.3
2001	7	22	90.1	71.6
2001	7	23	89.9	72.1
2001	7	24	89.3	74.6
2001	7	25	88.4	73.8
2001	7	26	88.4	74.1
2001	7	27	87.7	75.4
2001	7	28	86.7	75.3
2001	7	29	88.0	76.0
2001	7	30	90.0	74.7
2001	7	31	89.8	73.6
2001	8	1	89.0	73.1
2001	8	2	87.5	72.8
2001	8	3	84.9	71.0
2001	8	4	86.6	70.0
2001	8	5	89.5	70.8
2001	8	6	88.1	71.3
2001	8	7	83.8	73.1
2001	8	8	87.7	74.3
2001	8	9	90.0	73.6
2001	8	10	84.5	75.5
2001	8	11	84.7	75.0
2001	8	12	88.8	74.5

**Table 2.3-294 (Sheet 2 of 2)
Daily Average Meteorological Data
Mineral Wells Airport
Worst 30 Consecutive Day Period**

CP COL 2.3(1)

			Daily Average	
Yr	Month	Day	Dry Bulb (°F)	Wet Bulb (°F)
Average	-	-	88.3	73.8

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC
2. Period of Record – 6 yr (2001 – 2006).

Table 2.3-295
Precipitation Data CPNPP

CP COL 2.3(1)

Precipitation Measurements

Month	Monthly Mean	Max 48-hr Precipitation	Monthly (hrs)	Max 24 hour rain (in)	Number of Days >0.01 in
Jan	2.2	2.5	32	2.5	5
Feb	3.6	3.0	37	2.8	8
Mar	3.7	4.5	31	3.8	7
Apr	2.5	2.8	19	1.8	7
May	2.6	2.0	23	1.7	8
Jun	3.3	2.6	22	1.8	6
Jul	0.7	1.2	8	1.2	4
Aug	3.4	2.7	29	2.5	7
Sep	3.1	2.5	28	2.5	7
Oct	2.7	1.9	28	1.9	6
Nov	1.2	0.8	19	0.7	6
Dec	1.4	1.7	30	1.5	5
Annual	30.3	4.5	307	3.8	74

NOTES:

1. CPNPP site data 2001, 2003, 2006.

Table 2.3-296
Rainfall Frequency Distribution
Dallas Fort Worth Airport

CP COL 2.3(1)

NUMBER OF HOURS PER MONTH, AVERAGE YR

Rainfall (in/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	9	12	12	6	5	7	2	4	4	10	11	10
0.02-.099	16	25	15	10	11	15	4	7	8	14	16	18
0.10-0.249	5	6	6	5	6	4	2	3	3	6	4	6
0.25-0.499	1	2	2	2	2	2	1	1	1	2	2	2
0.50-0.99	0	1	1	1	2	1	1	1	0	1	0	0
1.00-1.99	0	0	1	0	1	0	0	0	0	0	0	0
2.0 & over	0	0	0	0	0	0	0	0	0	0	0	0
Total	32	45	35	24	26	29	10	15	16	34	33	37

NOTES:

1. Instances of "trace" precipitation were not counted in determining hours of precipitation.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-297
Rainfall Frequency Distribution
Mineral Wells

CP COL 2.3(1)

NUMBER OF HOURS PER MONTH, AVERAGE YR

Rainfall (in/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	10	10	16	9	14	5	5	10	10	14	15	10
0.02-.099	17	23	15	8	11	13	7	11	8	15	16	10
0.10-0.249	3	6	6	4	4	7	2	3	2	4	5	3
0.25-0.499	0	1	2	1	2	2	1	1	0	1	2	1
0.50-0.99	0	0	1	1	1	2	1	0	1	1	0	0
1.00-1.99	0	0	0	0	0	0	0	0	0	0	0	0
2.0 & over	0	0	0	0	0	0	0	0	0	0	0	0
Total	31	41	39	22	31	29	16	25	20	35	38	24

NOTES:

1. Instances of "trace" precipitation were not counted in determining hours of precipitation.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-298
Rainfall Frequency Distribution
CPNPP

CP COL 2.3(1)

NUMBER OF HOURS PER MONTH, AVERAGE YR

Rainfall (in/hr)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.01-0.019	9	11	8	7	6	6	4	7	8	7	6	11
0.02-.099	17	16	13	7	10	8	3	12	15	15	9	15
0.10-0.249	4	7	5	2	4	5	0	6	3	2	3	3
0.25-0.499	1	2	2	1	3	2	0	3	0	3	1	1
0.50-0.99	1	1	1	1	1	1	0	1	1	1	0	0
1.00-1.99	0	0	1	0	0	1	0	0	1	0	0	0
2.0 & over	0	0	0	0	0	0	0	0	0	0	0	0
Total	32	37	31	19	23	22	8	29	28	28	19	30

NOTES:

1. CPNPP site data 2001, 2003, 2006

Table 2.3-299 (Sheet 1 of 2)
Percent of Total Observations (by Month) of Indicated Wind Directions and Precipitation
Dallas Fort Worth Airport

CP COL 2.3(1)

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Total
N	2.06	2.59	1.56	0.75	1.23	0.98	0.65	0.50	0.75	1.57	2.06	1.90	16.60
N-NE	0.76	1.12	0.80	0.56	0.53	0.45	0.20	0.37	0.56	0.61	0.81	1.09	7.87
NE	0.28	0.78	0.59	0.20	0.34	0.25	0.03	0.16	0.31	0.55	0.72	0.65	4.86
E-NE	0.67	0.81	0.78	0.39	0.30	0.41	0.05	0.28	0.25	0.45	0.64	0.78	5.80
E	1.06	1.18	1.42	0.59	0.67	0.59	0.27	0.36	0.64	0.62	0.51	0.64	8.56
E-SE	0.87	0.95	0.90	0.55	0.47	0.89	0.36	0.33	0.42	0.64	0.51	0.73	7.62
SE	0.64	1.11	0.95	0.84	0.65	1.00	0.41	0.31	0.23	0.90	0.69	0.55	8.28
S-SE	0.53	0.70	0.86	0.98	0.75	1.08	0.31	0.31	0.27	1.39	0.62	0.47	8.26
S	0.94	1.20	0.61	1.04	1.06	1.15	0.42	0.47	0.30	1.18	0.59	0.61	9.57
S-SW	0.27	0.19	0.31	0.30	0.28	0.34	0.19	0.25	0.12	0.22	0.20	0.22	2.88
SW	0.08	0.16	0.22	0.20	0.09	0.16	0.12	0.19	0.08	0.11	0.09	0.12	1.62
W-SW	0.08	0.14	0.14	0.16	0.09	0.11	0.08	0.11	0.08	0.16	0.11	0.17	1.42
W	0.09	0.14	0.25	0.30	0.16	0.19	0.05	0.23	0.22	0.22	0.19	0.30	2.32

Table 2.3-299 (Sheet 2 of 2)
Percent of Total Observations (by Month) of Indicated Wind Directions and Precipitation
Dallas Fort Worth Airport

CP COL 2.3(1)

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Total
W-NW	0.41	0.20	0.30	0.17	0.09	0.08	0.02	0.03	0.14	0.25	0.30	0.19	2.17
NW	0.42	0.41	0.64	0.37	0.27	0.19	0.09	0.08	0.20	0.55	0.67	0.53	4.41
N-NW	0.97	0.97	0.69	0.31	0.51	0.20	0.28	0.16	0.48	0.76	1.23	1.17	7.73
Total	10.12	12.64	11.01	7.72	7.50	8.06	3.54	4.13	5.05	10.18	9.95	10.12	100

NOTES:

1. Instances of "trace" precipitation were counted as precipitation.
2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.
3. Period of Record – 10 yr (1997 – 2006).

Table 2.3-300 (Sheet 1 of 2)
Percent of Total Observations (by Month) of Indicated Wind Directions and Precipitation
Mineral Wells Airport

CP COL 2.3(1)

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Total
N	0.99	2.59	1.30	1.44	1.16	0.76	0.56	0.96	0.73	2.54	2.62	1.75	17.39
N-NE	0.34	0.51	0.51	0.23	0.51	0.34	0.23	0.14	0.37	0.51	0.56	0.65	4.88
NE	0.25	0.73	0.85	0.14	0.28	0.54	0.08	0.31	0.42	0.42	0.42	0.28	4.74
E-NE	0.37	0.54	0.68	0.25	0.48	0.31	0.23	0.34	0.25	0.31	0.45	0.28	4.48
E	1.04	0.65	1.10	0.28	0.39	0.45	0.25	0.51	0.51	0.70	0.99	0.45	7.33
E-SE	0.65	0.68	1.16	0.39	0.70	0.56	0.25	0.68	0.37	0.62	0.76	0.76	7.58
SE	1.24	1.47	1.44	1.27	1.41	1.55	0.85	0.59	0.37	1.04	1.21	1.16	13.59
S-SE	1.07	1.49	0.99	1.35	1.10	1.27	0.70	0.73	0.45	1.35	1.04	0.48	12.04
S	0.54	0.70	0.39	0.68	1.04	0.54	0.65	0.70	0.51	0.37	0.23	0.25	6.60
S-SW	0.08	0.06	0.28	0.17	0.23	0.20	0.23	0.20	0.17	0.17	0.06	0.08	1.92
SW	0.08	0.08	0.11	0.14	0.14	0.14	0.08	0.20	0.11	0.14	0.11	0.08	1.44
W-SW	0.03	0.06	0.14	0.06	0.23	0.06	0.11	0.17	0.17	0.11	0.14	0.03	1.30
W	0.23	0.23	0.20	0.23	0.34	0.06	0.08	0.31	0.20	0.11	0.20	0.25	2.42
W-NW	0.23	0.14	0.11	0.31	0.17	0.08	0.00	0.31	0.17	0.20	0.48	0.45	2.65
NW	0.28	0.25	0.39	0.34	0.20	0.08	0.17	0.23	0.34	0.34	1.04	0.42	4.09

Table 2.3-300 (Sheet 2 of 2)
Percent of Total Observations (by Month) of Indicated Wind Directions and Precipitation
Mineral Wells Airport

CP COL 2.3(1)

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Total
N-NW	0.87	0.96	0.85	0.45	0.45	0.25	0.23	0.28	0.34	0.73	1.44	0.70	7.56
Total	8.29	11.14	10.49	7.72	8.82	7.19	4.71	6.65	5.47	9.67	11.76	8.09	100.00

NOTES:

1. Instances of "trace" precipitation were counted as precipitation.

2. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

3. Period of Record – 6 yr (2001 – 2006).

Table 2.3-301
Percent of Total Observations (by Month) of Indicated Wind Directions and Precipitation CPNPP

CP COL 2.3(1)

Sector	January	February	March	April	May	June	July	August	September	October	November	December	Total
N	0.33	2.51	1.42	0.65	0.87	0.98	0.00	0.33	0.76	1.09	1.42	1.09	11.44
N-NE	0.98	0.98	0.44	0.22	0.33	0.22	0.22	0.76	1.09	0.65	0.33	0.44	6.64
NE	0.54	0.76	0.44	0.11	0.54	0.76	0.11	0.65	0.54	0.33	0.22	0.22	5.23
E-NE	0.65	0.22	0.76	0.11	0.54	0.33	0.00	0.65	0.00	0.22	0.00	0.22	3.70
E	1.42	0.00	0.76	0.00	0.54	0.11	0.54	0.98	0.98	0.54	0.33	0.54	6.75
E-SE	1.85	1.20	1.09	0.54	0.33	0.33	0.22	0.44	1.09	0.44	0.33	1.42	9.26
SE	1.09	1.53	0.65	0.76	0.87	0.44	0.44	1.31	0.98	1.63	0.33	0.98	11.00
S-SE	0.76	1.09	0.87	0.76	0.87	0.76	0.22	0.33	0.87	1.09	0.76	0.54	8.93
S	0.54	0.65	0.11	0.76	0.54	0.33	0.33	0.54	0.44	0.54	0.00	0.11	4.90
S-SW	0.33	0.00	0.33	0.33	0.00	0.54	0.22	0.33	0.22	0.33	0.00	0.00	2.61
SW	0.11	0.11	0.54	0.22	0.44	0.54	0.33	0.76	0.11	0.22	0.22	0.44	4.03
W-SW	0.22	0.11	0.54	0.11	0.00	0.33	0.00	0.33	0.44	0.11	0.22	0.44	2.83
W	0.00	0.11	0.22	0.54	0.22	0.22	0.00	0.87	0.00	0.00	0.00	0.33	2.51
W-NW	0.22	0.22	0.54	0.22	0.22	0.22	0.11	0.22	0.54	0.76	0.00	0.65	3.92
NW	0.98	0.65	0.76	0.11	0.54	0.33	0.00	0.76	0.22	0.33	0.00	0.87	5.56
N-NW	0.44	2.07	0.54	0.76	0.65	0.65	0.00	0.33	0.87	0.87	2.07	1.42	10.68
Total	10.46	12.20	10.02	6.21	7.52	7.08	2.72	9.59	9.15	9.15	6.21	9.69	100.00

NOTES:

1. Instances of "trace" precipitation were counted as precipitation.
2. Period of Record – 3 yr (2001, 2003, 2006).

Table 2.3-302
Average Hours of Fog and Haze
Dallas Fort Worth Airport

CP COL 2.3(1)

Month	Fog (Average hours/month)	Haze (Average hours/month)
Jan	6.3	5.3
Feb	1.8	5.4
Mar	1.7	4.7
Apr	0.8	4.9
May	0.0	13.8
Jun	0.1	4.2
Jul	0.2	4.2
Aug	0.0	4.7
Sep	0.0	9.2
Oct	1.3	2.4
Nov	1.9	1.8
Dec	2.1	1.2
Annual (hours/yr)	16.2	61.8

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Dallas Fort Worth International Airport, Station No. 03927.

2. Period of Record – 10 yr (1997 – 2006).

CP COL 2.3(1)

Table 2.3-303
Average Hours of Fog and Haze
Mineral Wells Airport

Month	Fog (Average hours/month)	Haze (Average hours/month)
Jan	12.5	9.0
Feb	8.2	2.2
Mar	5.8	4.7
Apr	3.8	5.8
May	1.7	11.8
Jun	0.3	6.7
Jul	0.3	7.5
Aug	0.7	5.5
Sep	0.3	13.0
Oct	5.5	3.8
Nov	4.3	5.5
Dec	3.2	5.5
Annual (hours/yr)	46.7	81.0

NOTES:

1. Data from Local Climatological Data, National Oceanic and Atmospheric Administration, U. S. Department of Commerce, Asheville, NC, Mineral Wells Airport, Station No. 93985.

2. Period of Record – 6 yr (2001 – 2006).

Table 2.3-304
CPNPP Monthly and Annual Stability Class Percent Frequency Distributions

CP COL 2.3(1)

	Stability Class						
Month	A	B	C	D	E	F	G
JAN	0.3%	0.5%	0.5%	4.1%	1.8%	0.7%	0.6%
FEB	0.4%	0.4%	0.5%	4.2%	1.4%	0.5%	0.3%
MAR	0.7%	0.5%	0.6%	4.2%	1.7%	0.5%	0.3%
APR	0.6%	0.6%	0.6%	4.1%	1.9%	0.3%	0.2%
MAY	1.0%	0.7%	0.7%	3.9%	1.7%	0.3%	0.1%
JUN	0.8%	0.7%	0.8%	3.7%	2.0%	0.2%	0.0%
JUL	1.1%	0.9%	0.7%	3.6%	2.0%	0.2%	0.0%
AUG	1.2%	0.8%	0.6%	3.6%	2.1%	0.2%	0.0%
SEP	0.7%	0.5%	0.7%	3.4%	2.5%	0.4%	0.1%
OCT	0.5%	0.5%	0.6%	4.0%	2.0%	0.6%	0.3%
NOV	0.3%	0.4%	0.5%	4.1%	1.9%	0.6%	0.4%
DEC	0.2%	0.4%	0.5%	3.9%	2.2%	0.8%	0.5%
Annual	7.8%	6.8%	7.3%	46.7%	23.2%	5.3%	2.9%

Reference: CPNPP site data 2001 – 2004, 2006.

CP COL 2.3(1)

Table 2.3-305 (Sheet 1 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS A			HRS			wind speed in m/sec							
DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.175	.000	.175	.175	.175	.524	1.398	4.892	5.940	5.066	11.355	3.843	33.716	5.741
NNE	.000	.349	.175	.349	.524	.699	2.271	3.494	5.066	5.940	3.843	.699	23.409	4.626
NE	.000	.175	.524	.000	1.398	1.223	7.512	7.512	6.638	2.795	1.572	.349	29.698	3.679
ENE	.000	.000	.000	.349	1.223	1.572	5.940	8.211	2.970	1.747	1.048	.000	23.060	3.415
E	.000	.000	.000	.175	.699	2.795	6.988	6.464	2.620	.175	.349	.000	20.265	3.023
ESE	.000	.000	.524	.524	.873	1.048	14.150	20.090	8.910	4.018	1.572	.000	51.710	3.519
SE	.000	.175	.175	.000	.349	1.048	7.337	16.422	22.885	15.199	6.988	1.398	71.975	4.496
SSE	.000	.000	.349	.175	.699	1.398	8.385	15.897	25.855	26.379	26.205	8.036	113.378	5.227
S	.000	.175	.000	.175	.175	1.048	5.590	15.199	20.440	28.126	38.608	18.867	128.402	5.840
SSW	.000	.000	.349	.000	.349	.175	4.717	7.163	10.831	11.355	18.518	5.416	58.873	5.512
SW	.000	.000	.000	.000	.000	.524	3.319	6.638	4.717	1.922	2.620	1.048	20.789	4.398
WSW	.000	.000	.175	.000	.175	.349	.873	1.398	1.747	1.398	1.048	1.572	8.735	5.249
W	.000	.000	.175	.000	.000	.175	.349	.524	.349	.175	.349	.524	2.620	4.861
WNW	.000	.000	.175	.000	.000	.524	.175	.175	.524	.175	1.572	.699	4.018	5.823
NW	.000	.000	.175	.000	.000	.000	.524	1.398	.873	1.922	3.843	6.813	15.548	7.443
NNW	.000	.000	.000	.000	.175	.000	2.795	3.494	5.940	6.289	13.626	18.168	50.487	7.155
CALM	.87												.87	.38

CP COL 2.3(1)

Table 2.3-305 (Sheet 2 of 10)
Annual Stability Class Frequency Distribution for CPNPP
(Upper Bound of Wind Speed Category Listed)

STABILITY CLASS A HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
TOTAL	1.05	.87	2.97	1.92	6.81	13.10	72.32	118.97	126.31	112.68	133.12	67.43	657.56	
HOURS OF CALM			.87											

STABILITY CLASS B HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.000	.175	.524	.000	.349	.699	3.145	4.367	5.241	4.892	10.307	5.765	35.463	5.741
NNE	.000	.175	.000	.349	.524	2.620	5.241	5.416	4.018	4.892	4.717	3.843	31.795	4.799
NE	.000	.000	.349	.699	.1.398	4.717	8.735	6.114	3.145	2.271	1.747	1.572	30.747	3.458
ENE	.000	.175	.000	.873	1.223	4.193	9.434	4.193	1.922	1.572	1.048	.000	24.632	2.952
E	.000	.000	.349	.699	1.048	1.048	6.114	3.843	1.747	.524	.000	.000	15.373	2.823
ESE	.000	.175	.175	.524	.000	2.271	10.482	8.036	2.970	1.048	.349	.000	26.030	3.110
SE	.000	.000	.000	.000	.524	1.398	11.006	7.861	10.132	6.114	3.145	.699	40.879	4.018
SSE	.000	.000	.349	.349	.699	1.572	8.385	10.831	10.132	13.976	20.265	7.687	74.246	5.303
S	.000	.175	.349	.349	.524	1.048	6.988	9.608	17.120	22.187	41.578	21.837	121.764	6.125
SSW	.000	.000	.000	.175	.175	1.398	3.843	7.512	7.337	7.337	9.259	4.717	41.753	5.246
SW	.000	.000	.175	.699	.524	1.048	6.464	9.259	6.638	6.988	4.018	.873	36.686	4.259

CP COL 2.3(1)

Table 2.3-305 (Sheet 3 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS B			HRS			wind speed in m/sec								
DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
WSW	.000	.000	.000	.000	.000	.873	2.271	4.717	2.446	1.223	2.970	1.572	16.072	4.742
W	.000	.000	.175	.000	.175	.349	.873	.000	1.223	.000	.349	.000	3.145	3.522
WNW	.000	.000	.000	.000	.000	.349	.000	.524	.699	.699	1.223	.699	4.193	5.925
NW	.000	.349	.000	.000	.175	.000	.524	1.747	2.620	3.145	6.114	4.892	19.566	6.401
NNW	.000	.000	.000	.175	.175	1.223	5.416	6.114	6.813	6.988	10.132	16.596	53.632	6.405
CALM	.70												.70	.35
TOTAL	.70	1.22	2.45	4.89	7.51	24.81	88.92	90.14	84.20	83.85	117.22	70.75	576.67	
HOURS OF CALM			.70											

STABILITY CLASS C			HRS			wind speed in m/sec								
DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.000	.175	.699	.873	1.048	1.572	5.416	6.289	7.163	7.687	12.403	10.132	53.457	5.531
NNE	.000	.175	.524	1.223	1.223	4.018	3.843	5.066	4.542	4.717	4.892	4.367	34.590	4.575
NE	.000	.175	.699	1.747	3.494	4.717	9.608	3.843	1.922	2.970	2.970	.349	32.494	3.128
ENE	.175	.175	1.223	1.223	3.319	6.289	8.036	4.018	4.367	1.747	.873	.175	31.620	2.806
E	.000	.524	.699	1.223	1.048	4.367	6.638	3.494	1.223	.349	.349	.000	19.915	2.480

CP COL 2.3(1)

Table 2.3-305 (Sheet 4 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS C			HRS			wind speed in m/sec							
DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
ESE	.175	.349	.349	.175	.524	4.367	14.500	6.464	2.271	1.398	.524	.000	31.096	2.875
SE	.000	.524	.000	.175	.873	1.398	8.910	8.211	9.608	8.385	4.018	.349	42.451	4.114
SSE	.000	.175	.524	.349	.349	4.367	7.687	12.054	9.783	16.946	20.964	8.385	81.583	5.250
S	.000	.175	.000	.873	.873	1.048	6.114	8.910	15.548	24.283	43.849	25.506	127.179	6.226
SSW	.000	.349	.524	.524	.873	2.271	5.416	2.795	7.163	7.687	12.403	4.717	44.722	5.233
SW	.000	.000	.349	.349	.175	1.398	5.940	5.416	4.542	4.717	5.241	1.747	29.873	4.597
WSW	.000	.000	.349	.349	.175	2.096	2.970	4.542	4.542	3.145	4.367	1.922	24.458	4.647
W	.000	.349	.349	.000	.699	.524	1.398	1.223	2.446	1.747	1.048	.699	10.482	4.272
WNW	.175	.349	.000	.000	.000	.524	1.398	1.048	1.223	1.048	1.398	2.271	9.434	5.422
NW	.000	.000	.349	.524	.175	.524	1.572	3.145	3.843	3.494	6.114	9.608	29.349	6.397
NNW	.000	.175	1.048	.175	.699	2.446	8.211	6.289	4.367	5.241	11.006	16.771	56.427	6.016
CALM	1.57												1.57	.36
TOTAL	2.10	3.67	7.69	9.78	15.55	41.93	97.66	82.81	84.55	95.56	132.42	87.00	660.70	
HOURS OF CALM			1.57											

CP COL 2.3(1)

Table 2.3-305 (Sheet 5 of 10)
Annual Stability Class Frequency Distribution for CPNPP
(Upper Bound of Wind Speed Category Listed)

STABILITY CLASS D HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.000	.873	1.223	1.223	3.494	9.783	42.626	73.897	86.126	83.854	103.071	51.885	458.055	5.293
NNE	.000	1.223	2.620	2.620	2.620	10.482	35.463	50.837	57.301	45.596	42.626	13.277	264.666	4.584
NE	.349	1.048	2.620	4.193	5.416	9.434	26.379	36.337	42.102	29.873	16.247	2.271	176.269	3.998
ENE	.175	1.398	3.319	5.066	5.241	11.181	28.301	38.608	35.289	20.090	7.337	.349	156.354	3.605
E	.000	1.572	2.446	3.669	6.988	15.373	58.873	53.807	18.693	6.464	1.922	.000	169.805	2.990
ESE	.175	1.223	3.669	2.795	9.434	24.108	78.439	53.632	23.759	10.657	2.795	.349	211.034	2.986
SE	.000	.873	2.620	2.620	5.416	14.849	73.547	110.408	103.246	76.168	35.289	3.145	428.182	4.104
SSE	.000	.699	2.620	2.446	3.145	11.879	41.753	88.397	149.715	161.944	197.757	55.728	716.083	5.394
S	.175	1.048	1.572	2.096	2.795	8.560	30.397	64.813	114.601	143.601	195.835	76.867	642.360	5.745
SSW	.000	.524	2.096	2.271	1.398	7.512	25.331	27.427	27.427	23.759	23.409	6.988	148.143	4.421
SW	.175	.873	2.096	1.572	2.620	6.988	14.325	11.879	9.783	7.861	8.910	4.717	71.800	4.004
WSW	.175	1.048	1.922	1.572	2.271	6.813	13.801	7.337	4.367	3.843	7.163	1.572	51.885	3.540
W	.000	.699	2.096	.175	1.398	1.398	7.512	4.717	4.367	2.795	3.319	1.922	30.397	3.827
WNW	.000	.524	.873	1.048	1.922	2.446	11.355	14.150	12.753	12.054	17.120	8.385	82.632	4.940
NW	.175	1.048	1.223	.524	1.223	4.193	15.548	16.771	19.391	24.458	41.753	27.078	153.384	5.711
NNW	.175	.873	1.747	1.223	2.446	7.512	19.741	26.554	37.036	50.662	93.812	76.168	317.948	6.192

CP COL 2.3(1)

Table 2.3-305 (Sheet 6 of 10)
Annual Stability Class Frequency Distribution for CPNPP
(Upper Bound of Wind Speed Category Listed)

STABILITY CLASS D HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
CALM	9.96												9.96	.33
TOTAL	11.53	15.55	34.76	35.11	57.82	152.51	523.39	679.57	745.96	703.68	798.36	330.70	4088.95	

HOURS OF CALM 9.96

STABILITY CLASS E HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.349	1.398	1.398	2.096	1.922	5.590	12.403	15.373	13.102	7.861	2.271	.175	63.939	3.423
NNE	.000	.873	.699	1.223	1.922	5.416	12.928	20.440	13.452	4.717	.524	.175	62.367	3.359
NE	.000	.524	.524	1.398	1.398	3.494	6.813	4.367	3.319	.524	.349	.000	22.711	2.755
ENE	.175	1.048	1.048	.873	1.048	3.494	5.241	2.620	1.747	.349	.000	.000	17.644	2.361
E	.000	1.398	2.446	1.747	3.145	10.657	25.506	11.530	1.747	.349	.000	.000	58.523	2.385
ESE	.349	.699	2.795	3.494	8.735	22.361	64.987	26.554	3.843	.175	.175	.000	134.167	2.454
SE	.175	3.145	2.271	4.193	8.910	23.584	113.902	163.516	80.535	14.675	1.572	.000	416.477	3.290
SSE	.699	2.271	3.494	3.669	5.416	17.994	84.204	149.715	144.125	54.505	17.295	2.795	486.181	3.845
S	.349	2.970	4.018	3.669	5.416	9.084	34.066	57.825	55.379	32.144	15.548	2.271	222.739	3.901
SSW	.524	2.795	4.542	3.843	6.114	8.560	26.030	28.476	26.030	16.946	10.482	1.398	135.739	3.627

CP COL 2.3(1)

Table 2.3-305 (Sheet 7 of 10)
Annual Stability Class Frequency Distribution for CPNPP
(Upper Bound of Wind Speed Category Listed)

STABILITY CLASS E HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
SW	.524	2.620	6.988	4.542	5.066	8.910	11.705	12.403	8.735	7.861	8.560	1.398	79.312	3.272
WSW	.175	1.747	2.970	2.446	4.193	6.464	9.783	5.940	5.765	4.717	3.843	.175	48.216	3.071
W	.175	1.223	1.747	2.096	3.319	4.367	7.337	3.843	3.319	1.572	.349	.000	29.349	2.519
WNW	.000	.873	3.669	3.319	2.446	4.892	13.976	13.452	11.181	4.193	2.620	.524	61.144	3.189
NW	.175	1.223	1.747	2.970	5.241	9.958	26.903	26.554	19.391	5.590	1.922	.524	102.198	3.173
NNW	.349	.349	1.572	1.747	3.494	5.241	12.229	13.277	9.434	5.940	3.145	2.271	59.048	3.561
CALM	23.76												23.76	.35
TOTAL	27.78	25.16	41.93	43.32	67.78	150.06	468.01	555.89	401.10	162.12	68.66	11.70	2023.51	

HOURS OF CALM 23.76

STABILITY CLASS F HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.000	.349	.000	.524	.349	.524	.175	.873	.699	.175	.000	.000	3.669	2.686
NNE	.000	.000	.175	.175	.000	.000	.349	1.747	.699	.349	.000	.000	3.494	3.668
NE	.000	.349	.175	.349	.000	.000	.000	.175	.000	.000	.000	.000	1.048	1.296
ENE	.000	.349	.000	.000	.000	.000	.000	.175	.000	.000	.000	.000	.524	1.565

CP COL 2.3(1)

Table 2.3-305 (Sheet 8 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS F			HRS			wind speed in m/sec								
DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
E	.000	.000	.000	.175	.175	.524	.699	.000	.000	.000	.000	.000	1.572	1.912
ESE	.000	.699	.699	.699	.524	1.398	1.572	1.572	.175	.000	.175	.000	7.512	2.091
SE	.175	.699	2.271	1.398	2.446	4.018	22.361	20.265	4.193	.175	.175	.000	58.174	2.776
SSE	.000	.873	3.319	2.620	2.620	5.940	16.422	17.295	5.590	.349	.000	.000	55.030	2.684
S	.349	1.223	3.669	1.572	6.464	9.608	15.548	13.102	5.765	1.223	.175	.175	58.873	2.545
SSW	.175	1.922	5.066	5.590	6.638	8.211	13.801	9.958	6.114	1.922	.349	.000	59.746	2.418
SW	.000	1.747	3.843	8.385	5.940	8.036	9.608	6.464	6.638	1.572	.349	.000	52.584	2.335
WSW	.175	2.096	4.018	5.241	5.416	5.590	14.150	6.813	4.193	1.922	.349	.000	49.963	2.345
W	.175	1.747	3.494	3.843	2.446	5.241	4.018	3.145	1.223	.699	.175	.000	26.205	2.000
WNW	.175	1.572	2.970	3.319	4.367	5.066	7.687	5.765	2.795	.524	.349	.000	34.590	2.262
NW	.000	1.223	1.922	2.096	4.193	6.638	24.807	15.199	2.096	.175	.000	.000	58.349	2.552
NNW	.000	.524	1.223	.699	.349	1.922	2.620	1.747	.175	.175	.000	.000	9.434	2.090
CALM	9.96												9.96	.36
TOTAL	11.18	15.37	32.84	36.69	41.93	62.72	133.82	104.29	40.35	9.26	2.10	.17	490.72	
HOURS OF CALM			9.96											

CP COL 2.3(1)

Table 2.3-305 (Sheet 9 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS G HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
N	.000	.175	.000	.175	.175	.000	.000	.000	.000	.000	.000	.000	.524	1.013
NNE	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000
NE	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000
ENE	.175	.000	.000	.000	.000	.000	.175	.000	.000	.000	.000	.000	.349	1.699
E	.000	.175	.000	.000	.000	.000	.175	.000	.000	.000	.000	.000	.349	1.654
ESE	.000	.175	.175	.000	.699	.175	.175	.000	.175	.000	.000	.000	1.572	1.704
SE	.000	.000	.873	.524	.349	1.747	2.271	1.398	.000	.000	.175	.000	7.337	2.235
SSE	.000	.873	1.048	1.223	1.922	.699	.699	.524	.000	.000	.000	.524	7.512	2.137
S	.000	.349	1.223	1.223	2.446	5.066	4.717	1.922	.699	.000	.524	.524	18.693	2.440
SSW	.000	.873	.524	3.319	6.464	11.006	8.735	5.416	2.096	.524	.349	.000	39.307	2.231
SW	.175	.524	2.271	3.494	4.542	8.560	12.229	5.590	2.271	.175	.175	.000	40.006	2.212
WSW	.000	1.048	2.271	3.145	4.542	11.355	18.168	11.530	5.241	1.048	.175	.000	58.523	2.537
W	.000	1.048	2.096	4.018	3.669	4.892	3.843	1.398	.175	.175	.000	.524	21.837	1.903
WNW	.175	1.048	4.018	3.319	3.494	5.940	4.717	.699	.175	.349	.000	.000	23.933	1.651
NW	.000	1.048	1.223	1.572	4.018	5.765	15.897	3.843	.349	.000	.000	.000	33.716	2.175
NNW	.000	.000	.175	.873	.175	1.398	1.572	.349	.000	.000	.000	.000	4.542	1.950

CP COL 2.3(1)

Table 2.3-305 (Sheet 10 of 10) Annual Stability Class Frequency Distribution for CPNPP (Upper Bound of Wind Speed Category Listed)

STABILITY CLASS G HRS wind speed in m/sec

DIR	0.5	0.75	1	1.25	1.5	2.00	3	4.0	5.0	6	8	16	TOTAL	AVE SPEED
CALM	3.67												3.67	.34
TOTAL	4.19	7.34	15.90	22.89	32.49	56.60	73.37	32.67	11.18	2.27	1.40	1.57	261.87	

HOURS OF CALM 3.67

Table 2.3-306
Inversion Heights and Strengths, Fort Worth
January 2000 – 2005

CP COL 2.3(1)

January	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	10	1323	0.401	19	577	0.444
2001	21	1327	0.336	33	932	0.355
2002	4	1634	0.184	13	761	0.456
2003	9	1254	0.406	18	600	0.487
2004	6	1270	0.393	16	736	0.357
2005	6	912	0.384	17	703	0.462
Total	56	1286	0.359	116	743	0.417

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive, http://raob.fsl.noaa.gov/.

Table 2.3-307
Inversion Heights and Strengths, Fort Worth
February 2000 – 2005

CP COL 2.3(1)

February	Mornings with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)	Afternoons with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)
2000	4	1509	0.260	16	705	0.729
2001	17	1234	0.294	29	776	0.543
2002	2	1876	0.392	12	360	0.531
2003	8	874	0.417	14	729	0.375
2004	6	1463	0.173	11	746	0.645
2005	4	1005	0.238	12	655	0.353
Total	41	1233	0.296	94	685	0.535

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-308
Inversion Heights and Strengths, Fort Worth
March 2000 – 2005

CP COL 2.3(1)

March	Mornings with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)	Afternoons with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)
2000	4	1326	0.511	10	664	0.300
2001	13	1434	0.307	20	808	0.472
2002	7	1183	0.297	13	852	0.397
2003	7	1507	0.335	10	754	0.384
2004	2	1537	0.524	8	535	0.666
2005				9	1010	0.207
Total	33	1390	0.349	70	783	0.409

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-309
Inversion Heights and Strengths, Fort Worth
April 2000 – 2005

CP COL 2.3(1)

April	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	7	1249	0.324	14	602	0.448
2001	16	1853	0.370	26	1184	0.375
2002	3	1850	0.387	10	1294	0.379
2003	2	1235	0.438	15	793	0.464
2004	5	1616	0.468	8	1273	0.328
2005	2	1677	0.443	4	814	0.362
Total	35	1652	0.385	77	1006	0.401

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-310
Inversion Heights and Strengths, Fort Worth
May 2000 – 2005

CP COL 2.3(1)

May	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	5	1250	0.343	8	989	0.383
2001	3	1898	0.301	13	1147	0.415
2002	4	1636	0.181	2	1317	0.361
2003	6	1750	0.361	7	1150	0.275
2004	4	1372	0.552	7	1013	0.188
2005				2	1278	0.262
Total	22	1567	0.351	39	1107	0.332

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-311
Inversion Heights and Strengths, Fort Worth
June 2000 – 2005

CP COL 2.3(1)

June	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	1	1454	0.357	2	908	0.315
2001	2	1949	0.175	15	822	0.319
2002	1	1996	0.381	1	196	0.532
2003	2	945	0.222	2	655	0.308
2004	1	2285	0.545	1	1496	0.200
2005	1	2097	0.375	2	1398	0.284
Total	8	1703	0.307	23	867	0.319

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-312
Inversion Heights and Strengths, Fort Worth
July 2000 – 2005

CP COL 2.3(1)

July	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000				2	402	0.352
2001	1	2602	0.250	2	603	0.345
2002				1	196	1.333
2003	1	1753	0.200	2	844	0.304
2004	2	1464	0.318	1	2110	0.125
2005				1	1932	0.055
Total	4	1821	0.271	9	882	0.391

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-313
Inversion Heights and Strengths, Fort Worth
August 2000 – 2005

CP COL 2.3(1)

August	Mornings with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)	Afternoons with Inversions[(a)]	Average Height[(b)] (m)	Average Strength[(c)] (0.1ºC/m)
2000				1	196	0.364
2001	3	1085	0.301	3	325	0.254
2002				2	1075	0.145
2003						
2004				1	662	0.486
2005						
Total	3	1085	0.301	7	569	0.254

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-314
Inversion Heights and Strengths, Fort Worth
September 2000 – 2005

CP COL 2.3(1)

September	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000				7	585	0.609
2001	7	1560	0.370	19	857	0.296
2002						
2003				4	1665	0.318
2004	1	2761	0.435	1	1533	0.273
2005	3	2451	0.275	1	2382	0.364
Total	11	1912	0.350	32	967	0.312

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-315
Inversion Heights and Strengths, Fort Worth
October 2000 – 2005

CP COL 2.3(1)

October	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	2	1293	0.189	6	1009	0.286
2001	18	1530	0.257	28	715	0.418
2002	3	790	0.338	5	1133	0.228
2003	4	1759	0.169	7	556	0.406
2004	2	1278	0.381	3	468	0.514
2005	3	1919	0.236	8	1106	0.426
Total	32	1495	0.255	57	805	0.392

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-316
Inversion Heights and Strengths, Fort Worth
November 2000 – 2005

CP COL 2.3(1)

November	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	6	1342	0.151	5	798	0.404
2001	9	1403	0.313	19	727	0.371
2002	6	1124	0.468	14	530	0.328
2003	7	1021	0.363	14	658	0.391
2004	3	1132	0.157	6	906	0.301
2005	2	1295	0.103	7	605	0.332
Total	33	1229	0.295	65	678	0.358

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-317
Inversion Heights and Strengths, Fort Worth
December 2000 – 2005

CP COL 2.3(1)

December	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	8	1124	0.486	15	804	0.321
2001	12	1330	0.317	26	672	0.410
2002	3	725	0.233	15	560	0.292
2003	4	1465	0.246	17	625	0.315
2004	4	1360	0.223	13	636	0.212
2005	5	1045	0.240	13	645	0.323
Total	36	1213	0.319	99	659	0.325

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-318
Inversion Heights and Strengths, Fort Worth
Annual 2000 – 2005

CP COL 2.3(1)

Annual	Mornings with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)	Afternoons with Inversions[a]	Average Height[b] (m)	Average Strength[c] (0.1ºC/m)
2000	47	1290	0.353	105	707	0.447
2001	122	1472	0.313	233	847	0.400
2002	33	1332	0.315	88	737	0.389
2003	50	1309	0.344	110	744	0.390
2004	36	1456	0.354	76	822	0.385
2005	26	1379	0.287	76	833	0.353
Total	314	1395	0.327	688	791	0.397

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

b) Balloons were released each day at 0000 Universal Time Coordinated (UTC) and 1200 UTC. Height is defined as elevation in meters where temperature first increases and is averaged only over those days with inversions.

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

d) Data from: FSL/NCDC Radiosonde Data Archive http://raob.fsl.noaa.gov/.

Table 2.3-319
Cooling Tower Visible Plume Length by Season

CP COL 2.3(1)

(Length in meters)

	Winter	Spring	Summer	Fall	Annual
Plume from LMDCT moving in the indicated direction					
S	6060	3660	2210	3540	4360
SSW	5590	2950	1670	2730	3210
SW	6210	3000	1400	2500	2990
WSW	5830	3590	1160	2770	2930
W	6140	3430	1170	3730	3360
WNW	6140	3100	1460	2630	3050
NW	6120	3130	1720	2840	3130
NNW	4970	2360	1060	2390	2410
N	4140	2090	930	2260	2190
NNE	3700	2300	1040	2690	2420
NE	4070	3140	1490	4210	3260
ENE	4960	3880	2040	4960	4220
E	4900	3000	2060	3970	3770
ESE	5440	3820	2910	4630	4600
SE	4620	3400	2260	3940	4020
SSE	4910	3190	1860	3690	3960
All	5050	2780	1330	3050	3050

Average Plume Lengths in Mi

	Winter	Spring	Summer	Fall	Annual
Plume from LMDCT moving in the indicated direction					
S	3.77	2.27	1.37	2.2	2.71
SSW	3.47	1.83	1.04	1.7	1.99
SW	3.86	1.86	0.87	1.55	1.86
WSW	3.62	2.23	0.72	1.72	1.82
W	3.82	2.13	0.73	2.32	2.09
WNW	3.82	1.93	0.91	1.63	1.9
NW	3.8	1.94	1.07	1.76	1.94
NNW	3.09	1.47	0.66	1.49	1.5
N	2.57	1.3	0.58	1.4	1.36
NNE	2.3	1.43	0.65	1.67	1.5
NE	2.53	1.95	0.93	2.62	2.03
ENE	3.08	2.41	1.27	3.08	2.62
E	3.04	1.86	1.28	2.47	2.34
ESE	3.38	2.37	1.81	2.88	2.86
SE	2.87	2.11	1.4	2.45	2.5
SSE	3.05	1.98	1.16	2.29	2.46
All	3.14	1.73	0.83	1.9	1.9

Table 2.3-320
Winter Plume Percent Frequency by Length and Direction

CP COL 2.3(1)

	(0 - <500 m (0 to 1/3 mi)	500 - <1000 m (1/3 - 2/3 mi)	1000 - <8000 m (2/3 - 5 mi)	8000 m and longer (>5 mi)	Total Freq
Plume from LMDCT moving in the indicated direction					
S	2.46	0.4	1.73	4.6	9.19
SSW	0.97	0.21	0.8	1.5	3.48
SW	0.32	0.17	0.59	0.84	1.92
WSW	0.27	0.18	0.34	0.65	1.44
W	0.7	0.19	0.76	1.59	3.24
WNW	0.83	0.27	1.03	1.98	4.11
NW	1.3	1.47	1.53	3.99	8.29
NNW	3.41	1.71	1.88	4.14	11.14
N	7.47	0.94	2.88	5.12	16.41
NNE	2.73	0.45	1.19	1.42	5.79
NE	1.73	0.25	1.43	0.87	4.28
ENE	1.45	0.34	1.31	1.46	4.56
E	1.09	0.29	0.78	1.14	3.3
ESE	1.42	0.28	1.1	1.95	4.75
SE	2.64	1.24	1.34	2.61	7.83
SSE	3.62	0.97	1.14	3.88	9.61
All	32.4	9.4	19.8	37.7	99.35

Table 2.3-321
Spring Plume Percent Frequency by Length and Direction

CP COL 2.3(1)

	(0 - <500 m (0 to 1/3 mi)	500 - <1000 m (1/3 - 2/3 mi)	1000 - <8000 m (2/3 - 5 mi)	8000 m and longer (>5 mi)	Total Freq
Plume from LMDCT moving in the indicated direction					
S	2.93	0.31	1.13	1.47	5.84
SSW	2.53	0.32	0.78	0.87	4.5
SW	1.8	0.2	0.94	0.45	3.39
WSW	1.13	0.17	0.72	0.5	2.52
W	2.15	0.28	0.72	1	4.15
WNW	2.69	0.23	0.7	1.01	4.63
NW	4.69	1.42	1.49	1.75	9.35
NNW	11.64	1.91	1.88	2.68	18.11
N	17.52	0.97	2.79	3.37	24.65
NNE	2.61	0.21	0.6	0.53	3.95
NE	1.66	0.19	0.66	0.52	3.03
ENE	1.12	0.1	0.65	0.53	2.4
E	1.22	0.12	0.38	0.38	2.1
ESE	1.03	0.14	0.41	0.58	2.16
SE	1.65	0.59	0.45	0.84	3.53
SSE	2.92	0.54	0.68	1.15	5.29
All	59.3	7.7	15.0	17.7	99.62

Table 2.3-322
Summer Plume Percent Frequency by Length and Direction

CP COL 2.3(1)

	(0 - <500 m (0 to 1/3 mi)	500 - <1000 m (1/3 - 2/3 mi)	1000 - <8000 m (2/3 - 5 mi)	8000 m and longer (>5 mi)	Total Freq
Plume from LMDCT moving in the indicated direction					
S	1.82	0.12	0.39	0.37	2.7
SSW	2.41	0.18	0.28	0.33	3.2
SW	2.32	0.18	0.3	0.22	3.02
WSW	2.43	0.09	0.25	0.18	2.95
W	4.15	0.26	0.36	0.32	5.09
WNW	5.3	0.24	0.59	0.56	6.69
NW	10.59	1.42	1.38	1.31	14.7
NNW	15.96	1.72	1.07	0.97	19.72
N	20.73	0.73	1.25	1.17	23.88
NNE	4.84	0.33	0.5	0.28	5.95
NE	2.55	0.29	0.67	0.16	3.67
ENE	1.38	0.19	0.44	0.18	2.19
E	1.04	0.11	0.29	0.15	1.59
ESE	0.75	0.1	0.28	0.24	1.37
SE	0.75	0.19	0.17	0.15	1.26
SSE	1.21	0.16	0.2	0.16	1.73
All	78.2	6.3	8.4	6.7	99.71

Table 2.3-323
Fall Plume Percent Frequency by Length and Direction

CP COL 2.3(1)

	(0 - <500 m (0 to 1/3 mi)	500 - <1000 m (1/3 - 2/3 mi)	1000 - <8000 m (2/3 - 5 mi)	8000 m and longer (>5 mi)	Total Freq
Plume from LMDCT moving in the indicated direction					
S	3.73	0.41	1.1	1.83	7.07
SSW	2.82	0.2	0.44	0.91	4.37
SW	1.91	0.23	0.47	0.47	3.08
WSW	1.84	0.11	0.33	0.55	2.83
W	2.91	0.31	0.66	1.65	5.53
WNW	3.38	0.3	0.59	1.06	5.33
NW	6.83	1.5	1.35	2.3	11.98
NNW	8.37	1.83	1.59	1.97	13.76
N	11.03	0.87	2.24	2.31	16.45
NNE	2.87	0.48	0.98	0.8	5.13
NE	1.53	0.19	1.02	0.89	3.63
ENE	1.01	0.19	0.74	1.03	2.97
E	0.99	0.26	0.59	0.66	2.5
ESE	1.15	0.27	0.68	1.16	3.26
SE	1.99	0.68	0.88	1.22	4.77
SSE	3.3	0.58	0.69	1.81	6.38
All	55.7	8.4	14.4	20.6	99.03

Table 2.3-324 (Sheet 1 of 2) Annual Hours/Yr of Plume Shadow

CP COL 2.3(1)

Directions are directions from the tower.

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.25	400	2028	2437	3125	4075	5017	4273	3678	3262	3107	3166	3889	4534	3584	2464	1964	1895
0.37	600	1392	1436	1723	2385	2986	2827	2592	2329	2037	1840	1938	2329	1981	1600	1400	1422
0.5	800	1096	994	1137	1681	2096	2184	2100	1861	1495	1289	1277	1541	1311	1185	1131	1128
0.62	1000	950	832	848	1340	1601	1810	1783	1537	1240	1013	945	1167	972	945	918	994
0.75	1200	822	744	706	1110	1286	1562	1573	1337	1085	838	724	898	754	819	768	881
0.87	1400	703	604	590	939	1087	1323	1377	1139	956	685	557	668	608	690	651	758
0.99	1600	546	527	533	834	894	1129	1253	1016	815	568	470	570	473	605	561	669
1.12	1800	454	464	490	757	776	1013	1165	892	716	461	409	497	382	521	514	606
1.24	2000	394	404	457	696	674	896	1083	809	632	383	357	436	342	463	452	534
1.37	2200	341	359	430	624	606	798	1008	733	561	337	307	375	298	415	409	469
1.49	2400	304	331	397	586	555	744	921	649	504	303	259	336	257	372	353	419
1.62	2600	279	304	376	557	512	694	840	580	470	274	224	304	233	343	320	379
1.74	2800	246	286	349	523	473	644	775	528	447	260	199	268	209	309	306	337
1.86	3000	229	270	328	487	439	604	730	500	417	250	174	243	194	289	292	305
1.99	3200	214	253	312	453	411	567	680	470	386	239	163	222	183	266	272	285
2.11	3400	204	239	297	411	385	539	643	437	363	232	157	198	170	244	260	269
2.24	3600	192	226	287	375	366	516	606	417	344	225	149	174	161	234	244	254
2.36	3800	186	214	275	354	354	496	578	404	333	221	139	161	148	229	234	241
2.49	4000	174	206	265	329	340	468	544	389	318	214	133	144	140	221	224	230
2.61	4200	163	193	254	309	332	450	515	374	304	203	126	131	131	209	212	219

Table 2.3-324 (Sheet 2 of 2)
Annual Hours/Yr of Plume Shadow

CP COL 2.3(1)

Directions are directions from the tower.

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
2.73	4400	153	179	243	289	325	429	498	354	293	196	122	122	128	196	206	209
2.86	4600	143	170	237	279	317	412	484	336	283	190	118	118	125	190	195	201
2.98	4800	134	155	230	272	310	394	465	324	272	186	108	108	123	181	187	190
3.11	5000	127	148	217	261	304	382	457	313	266	179	103	100	118	170	176	178
3.23	5200	116	141	209	247	297	373	445	304	260	177	99	89	113	163	170	170
3.36	5400	110	133	195	237	294	360	433	296	253	175	97	87	110	156	159	160
3.48	5600	108	121	189	228	286	347	417	290	243	164	94	83	105	146	152	152
3.6	5800	95	117	177	222	278	335	412	276	233	158	91	79	102	141	141	145
3.73	6000	90	113	170	214	273	317	399	270	221	152	88	75	99	137	133	140
3.85	6200	84	108	156	207	269	311	392	259	210	147	86	71	95	134	131	131
3.98	6400	76	104	148	195	266	296	376	256	197	144	83	68	94	129	126	127
4.1	6600	69	97	145	182	261	283	362	249	192	138	81	65	93	124	117	118
4.23	6800	62	90	142	175	255	275	352	239	181	133	77	60	91	121	112	113
4.35	7000	57	82	135	161	252	265	343	230	171	127	74	55	89	114	108	107
4.47	7200	55	81	133	157	245	255	330	219	158	124	72	52	81	108	101	105
4.6	7400	48	79	129	152	241	246	319	216	149	117	66	52	77	102	98	97
4.72	7600	40	71	124	148	239	243	301	208	142	108	65	50	76	99	90	89
4.85	7800	33	60	118	141	237	234	291	203	133	99	62	45	76	98	87	85
4.97	8000	31	50	110	138	234	229	275	188	121	93	59	43	70	95	82	76

Table 2.3-325
Annual Hours/Yr of Fogging

CP COL 2.3(1)

Directions are directions from the tower.

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	22	1.1	6.5	1.1	0.4	2.8	24.3	49.7	6.5	1.6	9.8	2.8	0.0	8.8	82.6	152.7
0.12	200	90.8	31.4	7.4	2.2	2.0	5.9	31	94.5	76.7	30.6	10.2	4.4	1.4	11.2	137.5	290.1
0.19	300	61.2	11.0	5.9	2.1	1.0	3.1	24.1	77.4	53.0	11.4	8.9	4.3	0.5	3.5	129.4	350.3
0.25	400	45.1	5.2	4.4	1.0	1.0	0.7	12.4	50.9	26.6	5.7	6.3	2.5	0.4	2.3	100.6	313.4
0.31	500	90.0	9.4	4.1	1.0	1.9	2.3	0.0	15.4	60.9	8.2	6.3	2.5	0.4	4.8	37.4	157.9
0.37	600	60.8	4.1	4.1	0.5	1.7	0.7	0.0	7.6	27.3	3.3	6.3	1.7	0.3	2.5	13.8	63.5
0.43	700	50.5	2.0	4.0	0.0	1.5	0.0	0.0	4.1	19.0	1.0	6.3	1.0	0.3	2.0	2.0	17.4
0.5	800	42.3	2.0	4.0	0.0	1.1	0.0	0.0	1.0	19.0	1.0	6.3	1.0	0.3	2.0	0.2	2.8
0.56	900	33.7	1.6	4.0	0.0	0.7	0.0	0.0	0.0	16.3	1.0	6.0	1.0	0.0	2.0	0.0	0.0
0.62	1000	28.5	1.5	2.0	0.0	0.5	0.0	0.0	0.0	15.5	1.0	3.0	1.0	0.0	2.0	0.0	0.0
0.68	1100	27.9	1.1	2.0	0.0	0.5	0.0	0.0	0.0	12.8	1.0	3.0	1.0	0.0	2.0	0.0	0.0
0.75	1200	20.5	1.0	2.0	0.0	0.5	0.0	0.0	0.0	7.5	0.5	3.0	1.0	0.0	1.5	0.0	0.0
0.81	1300	20.5	1.0	0.0	0.0	0.5	0.0	0.0	0.0	7.5	0.5	0.0	1.0	0.0	1.5	0.0	0.0
0.87	1400	20.5	1.0	0.0	0.0	0.5	0.0	0.0	0.0	7.5	0.5	0.0	1.0	0.0	1.5	0.0	0.0
0.93	1500	19.4	0.6	0.0	0.0	0.5	0.0	0.0	0.0	6.4	0.5	0.0	1.0	0.0	1.1	0.0	0.0
0.99	1600	19	0.5	0.0	0.0	0.5	0.0	0.0	0.0	6.0	0.5	0.0	0.6	0.0	1.0	0.0	0.0

Table 2.3-326
Annual Hours/Yr of RIME Icing

CP COL 2.3(1)

Directions are directions from the tower.

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	9	1.1	6.5	1.1	0	2.8	24.2	45.2	6.4	1.6	9.8	2.8	0	7.5	57.2	77.2
0.12	200	52.7	21.6	7	1.9	1.2	5.6	30.6	84.5	66.9	28.5	10.2	4.4	1.2	5.2	80.4	133.2
0.19	300	36	7.5	5.7	2	0	3	24.1	63.4	43	10	8.9	4.3	0.4	2.5	67.2	125
0.25	400	19.7	2.2	4	1	0	0.7	12.4	38.2	16.6	3.8	6.3	2.5	0.3	1.3	36.4	84.8
0.31	500	42.9	5.2	4	1	0	2.3	0	7.1	42.1	7.9	6.3	2.5	0.3	2.9	2.4	27.4
0.37	600	20.5	1.8	4	0.5	0	0.7	0	2.6	16.9	3.1	6.3	1.7	0.3	1.3	0.9	10
0.43	700	15.5	0	4	0	0	0	0	1.5	9	1	6.3	1	0.3	1	0.5	5.5
0.5	800	15	0	4	0	0	0	0	0	9	1	6.3	1	0.3	1	0	0.4
0.56	900	14.5	0	4	0	0	0	0	0	9	1	6	1	0	1	0	0
0.62	1000	14.5	0	2	0	0	0	0	0	9	1	3	1	0	1	0	0
0.68	1100	14.5	0	2	0	0	0	0	0	9	1	3	1	0	1	0	0
0.75	1200	7.5	0	2	0	0	0	0	0	4.5	0.5	3	1	0	0.5	0	0
0.81	1300	7.5	0	0	0	0	0	0	0	4.5	0.5	0	1	0	0.5	0	0
0.87	1400	7.5	0	0	0	0	0	0	0	4.5	0.5	0	1	0	0.5	0	0
0.93	1500	7.5	0	0	0	0	0	0	0	4.5	0.5	0	1	0	0.5	0	0
0.99	1600	7.5	0	0	0	0	0	0	0	4.5	0.5	0	0.6	0	0.5	0	0

Table 2.3-327 (Sheet 1 of 3)
Cooling Tower Salt Deposition Rate

CP COL 2.3(1)

Directions are directions to which the plume is headed

$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	24.41	23.80	49.39	41.11	26.96	30.99	66.47	86.01	137.3	30.79	64.35	42.38	10.97	10.24	19.96	31.85
0.12	200	20.60	18.04	28.58	24.38	17.19	20.56	42.02	61.28	127.2	25.41	34.31	23.97	7.70	8.26	14.18	19.06
0.19	300	7.20	5.41	16.70	12.60	3.65	4.94	33.28	52.17	49.19	8.60	22.56	15.76	2.07	2.54	11.45	15.01
0.25	400	6.25	4.69	7.09	5.42	3.26	4.43	25.57	40.43	42.50	7.51	9.34	6.36	1.84	2.21	9.02	11.50
0.31	500	6.28	4.69	0.23	0.17	3.46	4.74	10.67	13.03	42.80	7.61	0.44	0.37	1.79	2.22	4.05	4.65
0.37	600	6.17	4.59	0.15	0.13	3.35	4.57	8.79	10.91	42.08	7.42	0.26	0.22	1.67	2.18	3.19	3.90
0.43	700	5.55	4.12	0.10	0.10	2.97	3.85	5.98	8.05	37.63	6.42	0.17	0.15	1.41	1.98	2.04	2.80
0.5	800	3.77	2.78	0.08	0.07	1.84	2.33	4.55	6.52	24.97	4.25	0.14	0.13	0.97	1.36	1.42	2.02
0.56	900	0.33	0.19	0.07	0.06	0.25	0.28	4.54	6.51	0.76	0.27	0.14	0.13	0.14	0.17	1.42	2.02
0.62	1000	0.31	0.18	0.07	0.06	0.24	0.27	4.58	6.57	0.74	0.26	0.14	0.13	0.14	0.17	1.43	2.03
0.68	1100	0.31	0.18	0.07	0.06	0.24	0.27	5.33	7.62	0.73	0.26	0.14	0.13	0.14	0.17	1.66	2.25
0.75	1200	0.31	0.18	0.07	0.06	0.24	0.27	5.85	8.44	0.73	0.26	0.14	0.13	0.14	0.17	1.87	2.48
0.81	1300	0.31	0.18	0.07	0.06	0.24	0.27	6.04	8.65	0.73	0.26	0.14	0.13	0.14	0.17	1.92	2.53
0.87	1400	0.31	0.18	0.07	0.06	0.24	0.27	6.04	8.65	0.73	0.26	0.14	0.13	0.14	0.17	1.92	2.53
0.93	1500	0.31	0.18	0.07	0.06	0.24	0.27	5.94	8.58	0.73	0.26	0.14	0.13	0.14	0.17	1.90	2.45
0.99	1600	0.31	0.18	0.07	0.06	0.24	0.27	5.75	8.43	0.73	0.26	0.14	0.13	0.14	0.17	1.86	2.27
1.06	1700	0.31	0.18	0.06	0.05	0.24	0.27	5.75	8.43	0.73	0.26	0.12	0.12	0.14	0.17	1.86	2.27
1.12	1800	0.31	0.18	0.05	0.05	0.24	0.27	5.75	8.43	0.73	0.26	0.10	0.10	0.14	0.17	1.86	2.27

Table 2.3-327 (Sheet 2 of 3)
Cooling Tower Salt Deposition Rate

CP COL 2.3(1)

Directions are directions to which the plume is headed
$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
1.18	1900	0.31	0.18	0.05	0.04	0.24	0.27	5.75	8.43	0.73	0.26	0.09	0.09	0.14	0.17	1.86	2.27
1.24	2000	0.31	0.18	0.05	0.04	0.24	0.27	5.56	8.18	0.73	0.26	0.09	0.09	0.14	0.17	1.78	2.21
1.3	2100	0.30	0.17	0.04	0.04	0.23	0.25	5.28	7.83	0.71	0.25	0.08	0.08	0.14	0.16	1.68	2.13
1.37	2200	0.27	0.15	0.04	0.03	0.20	0.22	5.10	7.58	0.64	0.23	0.06	0.07	0.13	0.15	1.59	2.05
1.43	2300	0.19	0.10	0.03	0.02	0.12	0.13	4.49	6.62	0.41	0.14	0.05	0.05	0.09	0.11	1.35	1.79
1.49	2400	0.12	0.05	0.02	0.02	0.07	0.07	2.29	2.91	0.22	0.08	0.04	0.04	0.05	0.07	0.93	0.78
1.55	2500	0.10	0.05	0.02	0.02	0.05	0.06	2.28	2.89	0.18	0.06	0.03	0.03	0.04	0.06	0.93	0.78
1.62	2600	0.08	0.04	0.02	0.02	0.05	0.05	2.28	2.89	0.15	0.05	0.03	0.03	0.03	0.04	0.93	0.78
1.68	2700	0.08	0.04	0.02	0.02	0.05	0.05	2.28	2.89	0.15	0.05	0.03	0.03	0.03	0.04	0.93	0.78
1.74	2800	0.06	0.03	0.02	0.02	0.04	0.04	2.28	2.89	0.11	0.04	0.03	0.03	0.02	0.03	0.93	0.78
1.8	2900	0.05	0.02	0.02	0.02	0.03	0.03	2.28	2.89	0.09	0.03	0.03	0.03	0.02	0.03	0.93	0.78
1.86	3000	0.05	0.02	0.02	0.02	0.03	0.03	2.13	2.71	0.09	0.03	0.03	0.03	0.02	0.03	0.87	0.73
1.93	3100	0.05	0.02	0.02	0.01	0.03	0.03	1.80	2.28	0.09	0.03	0.03	0.03	0.02	0.03	0.75	0.63
1.99	3200	0.05	0.02	0.01	0.01	0.03	0.03	1.88	2.31	0.09	0.03	0.03	0.02	0.02	0.03	0.77	0.66
2.05	3300	0.04	0.02	0.01	0.01	0.03	0.03	1.93	2.34	0.08	0.03	0.03	0.02	0.02	0.02	0.79	0.67
2.11	3400	0.04	0.02	0.01	0.01	0.02	0.03	1.93	2.34	0.07	0.02	0.03	0.02	0.01	0.02	0.79	0.67
2.17	3500	0.04	0.02	0.01	0.01	0.02	0.03	1.93	2.34	0.07	0.02	0.03	0.02	0.01	0.02	0.79	0.67
2.24	3600	0.04	0.02	0.01	0.01	0.02	0.03	1.54	1.76	0.07	0.02	0.03	0.02	0.01	0.02	0.58	0.51

Table 2.3-327 (Sheet 3 of 3)
Cooling Tower Salt Deposition Rate

CP COL 2.3(1)

Directions are directions to which the plume is headed
$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
2.3	3700	0.04	0.02	0.01	0.01	0.02	0.03	1.27	1.38	0.07	0.02	0.03	0.02	0.01	0.02	0.45	0.41
2.36	3800	0.04	0.02	0.01	0.01	0.02	0.03	1.20	1.30	0.07	0.02	0.03	0.02	0.01	0.02	0.43	0.39
2.42	3900	0.04	0.02	0.01	0.01	0.02	0.03	1.06	1.24	0.07	0.02	0.03	0.02	0.01	0.02	0.38	0.34
2.49	4000	0.04	0.02	0.01	0.01	0.02	0.03	0.97	1.20	0.07	0.02	0.03	0.02	0.01	0.02	0.35	0.31
2.55	4100	0.04	0.02	0.01	0.01	0.02	0.03	0.97	1.20	0.07	0.02	0.03	0.02	0.01	0.02	0.35	0.31
2.61	4200	0.04	0.02	0.01	0.01	0.02	0.03	0.97	1.20	0.07	0.02	0.03	0.02	0.01	0.02	0.35	0.31

Table 2.3-328 (Sheet 1 of 3)
Chlorides Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.

$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	37.6	36.6	75.8	63.1	41.4	47.6	102.1	132.1	211.9	47.4	98.7	65.0	16.9	15.8	30.7	48.9
0.12	200	31.8	27.8	43.9	37.4	26.4	31.6	64.6	94.2	196.3	39.2	52.8	36.9	11.8	12.7	21.8	29.3
0.19	300	10.90	8.18	25.75	19.44	5.54	7.50	51.15	80.13	74.44	13.03	34.76	24.27	3.14	3.85	17.61	23.06
0.25	400	9.56	7.17	10.62	8.11	4.99	6.80	39.20	61.93	65.02	11.50	14.00	9.53	2.82	3.39	13.85	17.62
0.31	500	9.60	7.17	0.36	0.27	5.30	7.26	16.17	19.74	65.46	11.65	0.69	0.58	2.74	3.40	6.12	7.06
0.37	600	9.43	7.01	0.23	0.21	5.13	6.98	13.45	16.70	64.27	11.34	0.41	0.35	2.55	3.34	4.88	5.96
0.43	700	8.49	6.29	0.15	0.15	4.55	5.90	9.15	12.30	57.52	9.82	0.26	0.24	2.16	3.03	3.12	4.28
0.5	800	5.77	4.26	0.13	0.12	2.80	3.55	6.98	10.01	38.16	6.50	0.23	0.21	1.49	2.08	2.18	3.11
0.56	900	0.50	0.29	0.11	0.10	0.39	0.43	6.96	9.99	1.17	0.42	0.23	0.21	0.22	0.26	2.18	3.10
0.62	1000	0.48	0.28	0.11	0.10	0.38	0.42	7.04	10.09	1.14	0.41	0.23	0.21	0.22	0.26	2.21	3.12
0.68	1100	0.48	0.27	0.11	0.10	0.38	0.41	8.17	11.68	1.14	0.41	0.23	0.21	0.22	0.26	2.54	3.45
0.75	1200	0.48	0.27	0.11	0.10	0.38	0.41	8.97	12.94	1.14	0.41	0.23	0.21	0.22	0.26	2.86	3.80
0.81	1300	0.48	0.27	0.11	0.10	0.38	0.41	9.26	13.26	1.14	0.41	0.23	0.21	0.22	0.26	2.95	3.89
0.87	1400	0.48	0.27	0.11	0.10	0.38	0.41	9.26	13.26	1.14	0.41	0.23	0.21	0.22	0.26	2.95	3.89
0.93	1500	0.48	0.27	0.11	0.10	0.38	0.41	9.07	13.11	1.14	0.41	0.23	0.21	0.22	0.26	2.90	3.72
0.99	1600	0.48	0.27	0.11	0.10	0.38	0.41	8.81	12.91	1.14	0.41	0.23	0.21	0.22	0.26	2.85	3.48
1.06	1700	0.48	0.27	0.10	0.09	0.38	0.41	8.81	12.91	1.14	0.41	0.20	0.19	0.22	0.26	2.85	3.48
1.12	1800	0.48	0.27	0.09	0.08	0.38	0.41	8.81	12.91	1.14	0.41	0.17	0.16	0.22	0.26	2.85	3.48

Table 2.3-328 (Sheet 2 of 3) Chloride s Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.
$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
1.18	1900	0.48	0.27	0.08	0.07	0.38	0.41	8.81	12.91	1.14	0.41	0.15	0.14	0.22	0.26	2.85	3.48
1.24	2000	0.48	0.27	0.08	0.07	0.38	0.41	8.52	12.54	1.14	0.41	0.15	0.14	0.22	0.26	2.74	3.39
1.3	2100	0.45	0.26	0.07	0.06	0.34	0.38	8.09	11.99	1.08	0.38	0.13	0.12	0.21	0.25	2.58	3.26
1.37	2200	0.42	0.24	0.06	0.05	0.31	0.34	7.83	11.62	0.99	0.35	0.11	0.11	0.20	0.24	2.44	3.15
1.43	2300	0.29	0.15	0.05	0.04	0.18	0.20	6.90	10.16	0.62	0.22	0.08	0.09	0.13	0.17	2.07	2.75
1.49	2400	0.19	0.09	0.04	0.04	0.11	0.12	3.53	4.48	0.36	0.12	0.08	0.07	0.09	0.11	1.44	1.20
1.55	2500	0.16	0.07	0.04	0.03	0.09	0.10	3.51	4.46	0.29	0.10	0.06	0.06	0.07	0.09	1.43	1.20
1.62	2600	0.13	0.06	0.03	0.03	0.08	0.08	3.51	4.46	0.25	0.08	0.06	0.06	0.05	0.07	1.43	1.20
1.68	2700	0.13	0.06	0.03	0.03	0.08	0.08	3.51	4.46	0.25	0.08	0.06	0.06	0.05	0.07	1.43	1.20
1.74	2800	0.09	0.04	0.03	0.03	0.06	0.07	3.51	4.46	0.17	0.06	0.06	0.06	0.04	0.05	1.43	1.20
1.8	2900	0.08	0.04	0.03	0.03	0.05	0.06	3.51	4.46	0.15	0.05	0.06	0.06	0.03	0.05	1.43	1.20
1.86	3000	0.08	0.04	0.03	0.03	0.05	0.06	3.29	4.18	0.15	0.05	0.06	0.06	0.03	0.05	1.35	1.13
1.93	3100	0.08	0.04	0.03	0.03	0.05	0.06	2.78	3.53	0.15	0.05	0.06	0.05	0.03	0.05	1.16	0.98
1.99	3200	0.08	0.04	0.03	0.03	0.05	0.06	2.90	3.58	0.15	0.05	0.06	0.05	0.03	0.05	1.20	1.01
2.05	3300	0.07	0.03	0.03	0.03	0.05	0.05	2.98	3.61	0.13	0.05	0.06	0.05	0.03	0.04	1.22	1.04
2.11	3400	0.06	0.03	0.03	0.03	0.04	0.05	2.97	3.61	0.12	0.04	0.06	0.05	0.03	0.04	1.22	1.04
2.17	3500	0.06	0.03	0.03	0.03	0.04	0.05	2.97	3.61	0.12	0.04	0.06	0.05	0.03	0.03	1.22	1.04
2.24	3600	0.06	0.03	0.03	0.03	0.04	0.05	2.28	2.60	0.11	0.04	0.06	0.05	0.02	0.03	0.85	0.76

Table 2.3-328 (Sheet 3 of 3)
Chlorides Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.
kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
2.3	3700	0.06	0.03	0.03	0.03	0.04	0.05	1.98	2.16	0.11	0.04	0.06	0.05	0.02	0.03	0.70	0.63
2.36	3800	0.06	0.03	0.03	0.03	0.04	0.05	1.87	2.04	0.11	0.04	0.06	0.05	0.02	0.03	0.67	0.60
2.42	3900	0.06	0.03	0.03	0.03	0.04	0.05	1.65	1.95	0.11	0.04	0.06	0.05	0.02	0.03	0.60	0.53
2.49	4000	0.06	0.03	0.03	0.03	0.04	0.05	1.51	1.88	0.11	0.04	0.06	0.05	0.02	0.03	0.55	0.49
2.55	4100	0.06	0.03	0.03	0.03	0.04	0.05	1.51	1.88	0.11	0.04	0.06	0.05	0.02	0.03	0.55	0.49
2.61	4200	0.06	0.03	0.03	0.03	0.04	0.05	1.98	2.16	0.11	0.04	0.06	0.05	0.02	0.03	0.70	0.63

Table 2.3-329 (Sheet 1 of 3)
Total Dissolved Solids Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.
kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	73.7	71.8	150.0	125.0	81.0	93.3	203.4	264.2	415.5	93.1	195.1	128.5	33.1	31.0	61.1	97.3
0.12	200	62.2	54.3	85.6	72.9	51.5	61.6	130.2	190.3	384.9	76.6	103.1	72.1	23.1	24.9	43.8	59.1
0.19	300	21.0	15.8	50.8	38.4	10.8	14.6	101.2	158.8	143.4	25.2	68.5	47.8	6.1	7.4	34.7	45.7
0.25	400	18.7	14.0	19.7	15.0	9.8	13.4	75.2	118.1	126.8	22.5	26.0	17.7	5.5	6.6	26.9	33.8
0.31	500	18.7	14.0	0.7	0.6	10.3	14.2	31.4	38.3	127.6	22.7	1.4	1.2	5.4	6.7	11.9	13.7
0.37	600	18.4	13.7	0.5	0.4	10.0	13.6	25.9	32.1	125.1	22.1	0.9	0.7	5.0	6.5	9.4	11.5
0.43	700	16.6	12.3	0.3	0.3	8.9	11.5	17.3	23.3	112.1	19.2	0.6	0.5	4.2	5.9	5.8	8.0
0.5	800	11.1	8.2	0.3	0.3	5.4	6.8	13.6	19.4	73.3	12.5	0.5	0.5	2.9	4.0	4.3	6.0
0.56	900	1.0	0.6	0.3	0.2	0.8	0.9	13.6	19.4	2.4	0.9	0.5	0.5	0.5	0.5	4.3	6.0
0.62	1000	1.0	0.5	0.3	0.2	0.8	0.9	13.7	19.6	2.3	0.8	0.5	0.5	0.5	0.5	4.3	6.1
0.68	1100	1.0	0.5	0.3	0.2	0.8	0.9	16.0	22.9	2.3	0.8	0.5	0.5	0.5	0.5	5.0	6.8
0.75	1200	1.0	0.5	0.3	0.2	0.8	0.9	17.8	25.6	2.3	0.8	0.5	0.5	0.5	0.5	5.7	7.5
0.81	1300	1.0	0.5	0.3	0.2	0.8	0.9	18.3	26.2	2.3	0.8	0.5	0.5	0.5	0.5	5.8	7.7
0.87	1400	1.0	0.5	0.3	0.2	0.8	0.9	18.3	26.2	2.3	0.8	0.5	0.5	0.5	0.5	5.8	7.7
0.93	1500	1.0	0.5	0.3	0.2	0.8	0.9	17.9	25.9	2.3	0.8	0.5	0.5	0.5	0.5	5.7	7.3
0.99	1600	1.0	0.5	0.3	0.2	0.8	0.9	17.4	25.5	2.3	0.8	0.5	0.5	0.5	0.5	5.6	6.9
1.06	1700	1.0	0.5	0.2	0.2	0.8	0.9	17.4	25.5	2.3	0.8	0.4	0.4	0.5	0.5	5.6	6.9
1.12	1800	1.0	0.5	0.2	0.2	0.8	0.9	17.4	25.5	2.3	0.8	04	04	0.5	0.5	5.6	6.9

Table 2.3-329 (Sheet 2 of 3)
Total Dissolved Solids Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.

kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
1.18	1900	1.0	0.5	0.2	0.2	0.8	0.9	17.4	25.5	2.3	0.8	0.4	0.3	0.5	0.5	5.6	6.9
1.24	2000	1.0	0.5	0.2	0.2	0.8	0.9	16.8	24.8	2.3	0.8	0.4	0.3	0.5	0.5	5.4	6.7
1.3	2100	0.9	0.5	0.2	0.1	0.7	0.8	16.0	23.7	2.2	0.8	0.3	0.3	0.4	0.5	5.1	6.4
1.37	2200	0.8	0.5	0.1	0.1	0.6	0.7	15.3	22.8	2.0	0.7	0.3	0.3	0.4	0.5	4.7	6.2
1.43	2300	0.5	0.3	0.1	0.1	0.4	0.4	12.2	17.7	1.2	0.4	0.2	0.2	0.3	0.3	3.8	4.8
1.49	2400	0.4	0.2	0.1	0.1	0.2	0.3	6.9	8.8	0.8	0.3	0.2	0.2	0.2	0.2	2.8	2.4
1.55	2500	0.3	0.2	0.1	0.1	0.2	0.2	6.9	8.8	0.6	0.2	0.2	0.2	0.1	0.2	2.8	2.4
1.62	2600	0.3	0.1	0.1	0.1	0.2	0.2	6.9	8.8	0.6	0.2	0.2	0.2	0.1	0.2	2.8	2.4
1.68	2700	0.3	0.1	0.1	0.1	0.2	0.2	6.9	8.8	0.6	0.2	0.2	0.2	0.1	0.2	2.8	2.4
1.74	2800	0.2	0.1	0.1	0.1	0.1	0.2	6.9	8.8	0.4	0.1	0.2	0.2	0.1	0.1	2.8	2.4
1.8	2900	0.2	0.1	0.1	0.1	0.1	0.2	6.9	8.8	0.4	0.1	0.2	0.1	0.1	0.1	2.8	2.4
1.86	3000	0.2	0.1	0.1	0.1	0.1	0.2	6.5	8.2	0.4	0.1	0.2	0.1	0.1	0.1	2.6	2.2
1.93	3100	0.2	0.1	0.1	0.1	0.1	0.2	5.5	7.0	0.4	0.1	0.2	0.1	0.1	0.1	2.3	1.9
1.99	3200	0.2	0.1	0.1	0.1	0.1	0.2	5.7	7.1	0.4	0.1	0.2	0.1	0.1	0.1	2.4	2.0
2.05	3300	0.2	0.1	0.1	0.1	0.1	0.1	5.9	7.2	0.3	0.1	0.1	0.1	0.1	0.1	2.4	2.1
2.11	3400	0.1	0.1	0.1	0.1	0.1	0.1	5.9	7.2	0.3	0.1	0.1	0.1	0.1	0.1	2.4	2.1
2.17	3500	0.1	0.1	0.1	0.1	0.1	0.1	5.9	7.2	0.3	0.1	0.1	0.1	0.1	0.1	2.4	2.1
2.24	3600	0.1	0.1	0.1	0.1	0.1	0.1	4.4	5.0	0.3	0.1	0.1	0.1	0.1	0.1	1.6	1.4

Table 2.3-329 (Sheet 3 of 3)
Total Dissolved Solids Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.

kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
2.3	3700	0.1	0.1	0.1	0.1	0.1	0.1	4.0	4.4	0.3	0.1	0.1	0.1	0.1	0.1	1.4	1.3
2.36	3800	0.1	0.1	0.1	0.1	0.1	0.1	3.8	4.2	0.3	0.1	0.1	0.1	0.1	0.1	1.4	1.2
2.42	3900	0.1	0.1	0.1	0.1	0.1	0.1	3.4	4.0	0.3	0.1	0.1	0.1	0.1	0.1	1.2	1.1
2.49	4000	0.1	0.1	0.1	0.1	0.1	0.1	3.1	3.9	0.3	0.1	0.1	0.1	0.1	0.1	1.1	1.0
2.55	4100	0.1	0.1	0.1	0.1	0.1	0.1	3.1	3.9	0.3	0.1	0.1	0.1	0.1	0.1	1.1	1.0
2.61	4200	0.1	0.1	0.1	0.1	0.1	0.1	3.1	3.9	0.3	0.1	0.1	0.1	0.1	0.1	1.1	1.0

Table 2.3-330 (Sheet 1 of 3)
Water Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.
kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
0.06	100	8500	8200	18000	15000	9000	10000	23000	30000	49000	11000	23000	15000	3700	3500	6800	11000
0.12	200	7200	6300	10000	8600	5800	7000	15000	21000	45000	8800	12000	8500	2600	2800	4900	6600
0.19	300	2500	1900	6000	4500	1200	1700	11000	18000	17000	3000	8000	5600	690	870	3900	5100
0.25	400	2200	1600	2300	1800	1100	1500	8400	13000	15000	2600	3100	2100	620	780	3000	3800
0.31	500	2200	1600	75	54	1100	1500	3400	4200	15000	2600	150	130	610	780	1300	1500
0.37	600	2200	1600	45	40	1100	1500	2800	3500	15000	2600	80	74	580	760	1000	1300
0.43	700	1900	1400	29	28	970	1200	1900	2500	13000	2200	50	49	490	690	640	880
0.5	800	1300	960	24	22	610	770	1500	2100	8600	1500	45	44	340	470	460	650
0.56	900	110	62	23	20	83	91	1500	2100	250	90	44	44	47	56	460	650
0.62	1000	100	60	23	20	80	88	1500	2100	250	86	44	44	47	56	470	660
0.68	1100	100	59	23	20	80	88	1800	2600	240	86	44	44	47	56	560	77
0.75	1200	100	59	23	20	80	88	2100	3000	240	86	44	44	47	56	660	880
0.81	1300	100	59	23	20	80	88	2100	3100	240	86	44	44	47	56	680	900
0.87	1400	100	59	23	20	80	88	2100	3100	240	86	44	44	47	56	680	900
0.93	1500	100	59	23	20	80	88	2100	3000	240	86	44	44	47	56	670	860
0.99	1600	100	59	23	20	80	88	2000	3000	240	86	44	44	47	56	660	810
1.06	1700	100	59	19	17	80	88	2000	3000	240	86	35	36	47	56	660	810
1.12	1800	100	59	16	14	80	88	2000	3000	240	86	30	32	47	56	660	810

CP COL 2.3(1)

Table 2.3-330 (Sheet 2 of 3)
Water Deposition

Directions are directions to which the plume is headed.
kg/km^2/month

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
1.18	1900	100	59	14	12	80	88	2000	3000	240	86	27	28	47	56	660	810
1.24	2000	100	59	14	12	80	88	2000	2900	240	86	25	27	47	56	630	780
1.3	2100	98	56	12	11	72	79	1900	2800	230	81	20	22	45	53	590	750
1.37	2200	91	51	11	9.7	65	71	1800	2700	210	74	18	21	42	50	550	720
1.43	2300	58	28	8.2	7.8	34	38	1400	2100	120	40	13	16	25	32	440	560
1.49	2400	41	18	6.9	6.9	21	24	800	1000	74	24	11	12	18	22	320	270
1.55	2500	32	14	5.8	6	17	18	800	1000	55	18	8.9	8.6	12	16	320	270
1.62	2600	27	13	5.7	6	14	16	800	1000	48	15	8.8	8.5	10	14	320	270
1.68	2700	27	13	5.7	6	14	16	800	1000	48	15	8.8	8.5	10	14	320	270
1.74	2800	16	7.2	5.7	5.9	9.5	10	800	1000	27	8.3	8.8	8.4	5.6	7.9	320	270
1.8	2900	14	6.1	5.4	5.4	8.5	8.9	800	1000	24	6.9	8.5	8.2	4.8	7.1	320	270
1.86	3000	14	6.1	5.1	5	8.5	8.9	750	950	24	6.9	8.2	7.7	4.8	7.1	300	260
1.93	3100	14	6.1	3.8	3.3	8.5	8.9	630	800	24	6.9	7.1	5.8	4.8	7.1	260	220
1.99	3200	14	6.1	3.8	3.3	8.5	8.9	660	810	24	6.9	7.1	5.8	4.8	7.1	270	230
2.05	3300	10	4.7	3.4	3	6.4	7	680	820	18	5.2	5.3	5.2	3.5	4.9	280	240
2.11	3400	9.1	4.4	3.3	2.9	5.9	6.6	680	820	16	4.8	4.5	4.8	3.1	4.4	280	240
2.17	3500	9.1	4.4	3.3	2.9	5.9	6.6	680	820	16	4.8	4.5	4.8	3.1	4.4	280	240
2.24	3600	9.1	4.4	3.3	2.9	5.9	6.6	490	550	16	4.8	4.5	4.8	3.1	4.4	180	160
2.3	3700	9.1	4.4	3.3	2.9	5.9	6.6	440	470	16	4.8	4.5	4.8	3.1	4.4	150	140

Table 2.3-330 (Sheet 3 of 3) Water Deposition

CP COL 2.3(1)

Directions are directions to which the plume is headed.
$kg/km^2/month$

(mi)	(m)	S	SSW	SW	WSW	W	WNW	NW	NNW	N	NNE	NE	ENE	E	ESE	SE	SSE
2.36	3800	9.1	4.4	3.3	2.9	5.9	6.6	410	450	16	4.8	4.5	4.8	3.1	4.4	150	130
2.42	3900	9.1	4.4	3.3	2.9	5.9	6.6	360	430	16	4.8	4.5	4.8	3.1	4.4	130	120
2.49	4000	9.1	4.4	3.3	2.9	5.9	6.6	330	410	16	4.8	4.5	4.8	3.1	4.4	120	100
2.55	4100	9.1	4.4	3.3	2.9	5.9	6.6	330	410	16	4.8	4.5	4.8	3.1	4.4	120	100
2.61	4200	9.1	4.4	3.3	2.9	5.9	6.6	330	410	16	4.8	4.5	4.8	3.1	4.4	120	100

Note: These can be converted to inches per year of increased precipitation by multiplying by $4.7x10^{-7}$

Table 2.3-331
Visible Plume Length Summary

CP COL 2.3(1)

	Winter	Spring	Summer	Fall
Most Frequent Plume Heading Directions	N,NNW,SSE,S	N,NNW,NW	N,NNW,NW	N,NNW,NW,S
Percent of Plumes < 1/3 mi	32.4	59.3	78.2	55.7
Percent of Plumes >1/3 to 2/3 mi	9.4	7.7	6.3	8.4
Percent of Plumes >2/3 to 5 mi	19.8	15.0	8.4	14.4
Percent of Plumes >5 mi	37.7	17.7	6.7	20.6

CP COL 2.3(1)

Table 2.3-332
CPNPP Meteorological System Accuracies

Parameter	Recording Type	System Accuracy (ANSI/ANS-2.5-1984)[1]	Actual System Accuracy[2]	Resolution
Wind Speed	Digital	±0.5 mph, WS<5mph ±10%, otherwise	±0.39mph, WS<25mph ±1.10%, otherwise	0.1 mph
	Paperless Digital	±0.75mph, WS<5mph ±15%, otherwise	±0.58mph, WS<25mph ±1.18%, otherwise	
Wind Direction	Digital	±5°	±3.4°	1°
	Paperless Digital	±7.5°	±4.5°	
Temperature	Digital	±0.9°F	±0.6°F	0.1°F
	Paperless Digital	±0.9°F	±0.9°F	
Delta Temperature	Digital	±0.27°F	±0.17°F	0.01°F
	Paperless Digital	±0.27°F	±0.19°F	
Precipitation	Digital	Rain gauge with ±0.01 in resolution ±10% measured value for total accumulated catch greater than 0.2 in	Rain gauge with ±0.01 resolution ±0.011 in or ±1.1%	0.01 in
	Paperless Digital	Rain gauge with ±0.01 in resolution +10% measured value for total accumulated catch greater than 0.2 in	Rain gauge with ±0.01 resolution ±0.013 in or ±1.3%	

Notes:

1. Endorsed by Reg. Guide 1.23, Second Proposed Revision 1, April 1986.

2. Accuracy values shown were calculated for the original system. Calculations made for subsequent equipment upgrades computed uncertainties equal to or less than those stated. All uncertainties computed are within acceptance criteria.

CP COL 2.3(1)

Table 2.3-333
CPNPP Meteorological Delta Temperature System Accuracy

Instrument Accuracy	
1. Sensor Accuracy	
Signal Conditioner Accuracy	±0.13°F
Instrument Accuracy	±0.08°F
Temperature Coefficient	±0.05°F
2. Sq Root of the Sum of the Squared Tolerances	±0.09°F
3. Transmitter Accuracy	±0.04°F
4. Receiver Accuracy	±0.04°F
5. Current Driver Accuracy	±0.04°F
6. Digital Recorder Accuracy	
Input Resistor Accuracy	±0.05°F
Input Accuracy	±0.05°F
7. Sq Root of the Sum of the Squared Tolerances	±0.071°F
8. Analog Data Reduction Accuracy	±0.05°F
System Accuracy [a]	
Digital Recording	
Sq Root of the Sum of the Squared Tolerance of 1, 2, 3, 4, 5 and 6	±0.17°F

a) These values are well within the ±0.27°F criteria established by ASI/ANS-2.5-1984, which is endorsed by Regulatory Guide 1.23 and the criteria of Regulatory Guide 1.23 Second Proposed Revision 1, April 1986.

CP COL 2.3(1)

Table 2.3-334
Annual Meteorological System Data Recovery Rates

Yr	Data Recovery
2001	99.0%
2002	98.0%
2003	99.3%
2004	98.7%
2005	Not Used
2006	99.6%

Notes:

1. Data recovery based on data needed for dose analysis joint frequency distributions (delta temperature and lower level wind speed and wind direction).

2. Meteorological data for 2005 were not used because the data recovery was below the required 90% recovery given in Regulatory Guide 1.23. The meteorological computer was inoperable for 36 days resulting in low data recovery.

Table 2.3-335
Minimum Exclusion Area Boundary (EAB) and LPZ Distances

CP COL 2.3(2)

Boundary	Distance
EAB distance (from containment centerline)	0.5 mi
Release boundary (from containment centerline)	670 ft
Distance from release boundary to EAB	1970 ft (600 m)
LPZ distance (from center point between Units 3 and 4)	2 mi (10,560 ft, 3219 m)

Table 2.3-336
Receptor Locations Within Five Miles

CP COL 2.3(2)

Sector	Residence(1)	Garden	SCR(2)
S	5751		
SSW	4185		
SW	4185		
WSW	6132		
W	6132		
WNW	11959		517
NW	11532		517
NNW	11532		517
N	10504		517
NNE	10504		517
NE	12640		517
ENE	12675	15120	517
E	14598	15120	517
ESE	12804		517
SE	10320		
SSE	9653		

1. Distances, in ft, from the center point between Units 3 and 4 to the nearest receptor (residence, garden, or recreational use of SCR) in each sector.

2. SCR refers to Squaw Creek Reservoir.

Table 2.3-337 (Sheet 1 of 2)
Relative Concentration at CPNPP

CP COL 2.3(2)

Summary - Exclusion Area Boundary χ/Q Values (s/m^3)

	Exclusion Area Boundary χ/Q (s/m^3)		
	Direction Dependent χ/Q		Direction Independent χ/Q
Time Period	0.5% Max Sector χ/Q[(a)]	Sector/Distance	5% Overall Site Limit
0 – 2 Hrs	3.36E-04	ENE / 600 m	2.59E-04

Summary - Low Population Zone χ/Q Values (s/m^3)

	Low Population Zone χ/Q (s/m^3)		
	Direction Dependent χ/Q		Direction Independent χ/Q
Time Period	0.5% Max χ/Q[(a)]	Sector	5% Site Limit[(b)]
0 – 8 Hrs	2.08E-05	NNE	1.85E-05
8 – 24 Hrs	1.35E-05	NNE	1.28E-05
1 – 4 Days	5.33E-06	NNE	5.76E-06
4 – 30 Days	1.67E-06	NNW	1.83E-06

Comanche Peak Maximum Accident χ/Q Values (s/m^3)

	0 – 2 Hrs	0 – 8 Hrs	8 – 24 Hrs	24 – 96 Hrs	96 – 720 Hrs
EAB (ENE, 600 m)	3.70E-04				
LPZ (NNE, 3219 m)		2.29E-05	1.49E-05		
LPZ (5% Site Limit, 3219 m)				6.34E-06	2.01E-06

Comanche Peak Maximum 50% Probability-Level χ/Q Values (s/m^3)

	0 – 2 Hrs	0 – 8 Hrs	8 – 24 Hrs	24 – 96 Hrs	96 – 720 Hrs
EAB	5.75E-05				
LPZ		3.32E-06	2.75E-06	1.83E-06	1.01E-06

Table 2.3-337 (Sheet 2 of 2)
Relative Concentration at CPNPP

χ/Q VALUES (sec/m^3)

Site Values Compared to DCD Values

	0 – 2 Hrs	0 – 8 Hrs	8 – 24 Hrs	27 – 96 Hrs	96 – 720 Hrs
EAB (600m, ENE)	3.7×10^{-4}				
LPZ (3219 m, NNE)		2.29×10^{-5}	1.49×10^{-5}	6.34×10^{-6}	2.01×10^{-6}
US-APWR DCD χ/Q Values					
EAB	5.0×10^{-4}				
LPXZ		2.1×10^{-4}	1.3×10^{-4}	6.9×10^{-5}	2.8×10^{-5}

a) 0.5% χ/Q values represent the maximum for all sector-dependent values.

b) As identified in the PAVAN manual, the direction independent χ/Q envelope values (i.e. the 5% "SITE LIMIT" values) are considered approximations and require confirmation.

CP COL 2.3(2)

Table 2.3-338 (Sheet 1 of 3)
Main Control Room and TSC HVAC Intake Distances and Directions

Main Control Room/Class 1E Electrical Room East HVAC Intake Distances and Directions

Release Point	Distance (m)	Direction to Source (°)
Plant Vent	66.1	307.5°
Main Steam Line	17.1	243.5°
Fuel Handling Area	75.9	353.5°
Relief Valves	27.4	290.5°
Safety Valves	24.1	268.5°
Containment Shell	26.8	311.5°

Main Control Room/Class 1E Electrical Room West HVAC Intake Distances and Directions

Release Point	Distance (m)	Direction to Source (°)
Plant Vent	50.9	11.5°
Main Steam Line	24.7	86.5°
Fuel Handling Area	101.8	33.5°
Relief Valves	27.4	52.5°
Safety Valves	24.1	74.5°
Containment Shell	26.8	31.5°

Above Grade Elevations of the Main Control Room and Class 1E Electrical Room HVAC Intakes

Receptor	Elevation (m)
Control Room HVAC Intake	13.9
Class 1E Electrical Room HVAC Intake	13.9

CP COL 2.3(2)

Table 2.3-338 (Sheet 2 of 3)
Main Control Room and TSC HVAC Intake Distances and Directions

TSC HVAC Intake Distances and Directions

Release Point	Distance (m)	Direction to Source (°)
Plant Vent	55.5	73°
Main Steam Line	70.1	109°
Fuel Handling Area	111.9	63°
Relief Valves	62.5	89°
Safety Valves	63.4	94°
Containment Shell	46.3	83°

Above Grade Elevations of the TSC Intakes

Receptor	Lower Elevation (m)	Upper Elevation (m)
TSC HVAC Intake	23.3	25.4
1E		

CP COL 2.3(2)

Table 2.3-338 (Sheet 3 of 3)
Main Control Room and TSC HVAC Intake Distances and Directions

Release Heights

Release Point	Elevation Above Grade (m)
Plant Vent	69.9
Main Steam Line (East)	12.8
Main Steam Line (West)	26.3
Fuel Handling Area	5.9
Main Steam Relief Valve (East)	40.7
Main Steam Relief Valve (West)	40.7
Main Steam Safety Valve (East)	38.8
Main Steam Safety Valve (West)	38.8
Containment Shell	49.5

Note:

The sampling system line, air lock and equipment hatch release locations (sources) listed in the DCD (Figure 15A-1) are not considered above because they are interior to the Auxiliary Building. Likewise, the Reactor Building Door is not evaluated because it is an interior door. The Auxiliary Building intake location is not specifically evaluated because this pathway is bounded by the main control room HVAC pathway.

Table 2.3-339 (Sheet 1 of 2)
Main Control Room and TSC Atmospheric Dispersion Factors (χ/Q) for Accident Dose Analysis

CP COL 2.3(2)

Main Control Room χ/Q (s/m^3) at the East HVAC Intake

Time Interval	Plant Vent	Main Steam Line	Fuel Handling Area
0 – 2 hours	6.3E-04	1.6E-02	9.6E-04
2 – 8 hours	4.1E-04	8.3E-03	7.5E-04
8 – 24 hours	1.7E-04	3.5E-03	3.1E-04
1 – 4 days	1.1E-04	2.5E-03	2.0E-04
4 – 30 days	9.0E-05	1.7E-03	1.7E-04
Time Interval	**Main Steam Relief Valves**	**Main Steam Safety Valves**	**Containment Shell**
0 – 2 hours	2.9E-03	3.3E-03	7.5E-04
2 – 8 hours	1.7E-03	1.9E-03	5.1E-04
8 – 24 hours	6.9E-04	7.6E-04	2.2E-04
1 – 4 days	4.9E-04	5.4E-04	1.4E-04
4 – 30 days	3.9E-04	3.8E-04	1.2E-04

Main Control Room and TSC Atmospheric Dispersion Factors (χ/Q) for Accident Dose Analysis

Main Control Room χ/Q (s/m^3) at the West HVAC Intake

Time Interval	Plant Vent	Main Steam Line	Fuel Handling Area
0 – 2 hours	9.4E-04	6.6E-03	5.4E-04
2 – 8 hours	7.3E-04	4.3E-03	4.1E-04
8 – 24 hours	3.1E-04	1.8E-03	1.7E-04
1 – 4 days	1.9E-04	1.3E-03	1.1E-04
4 – 30 days	1.6E-04	8.9E-04	7.8E-05
Time Interval	**Main Steam Relief Valves**	**Main Steam Safety Valves**	**Containment Shell**
0 – 2 hours	3.4E-03	4.1E-03	8.7E-04
2 – 8 hours	2.4E-03	2.7E-03	6.1E-04
8 – 24 hours	9.9E-04	1.1E-03	2.7E-04
1 – 4 days	6.6E-04	8.1E-04	1.7E-04
4 – 30 days	4.5E-04	5.1E-04	1.4E-04

Table 2.3-339 (Sheet 2 of 2) Main Control Room and TSC Atmospheric Dispersion Factors (χ/Q) for Accident Dose Analysis

CP COL 2.3(2)

TSC χ/Q (s/m^3) at the TSC HVAC Intake

Time Interval	Plant Vent	Main Steam Line	Fuel Handling Area
0 – 2 hours	1.1E-03	1.3E-03	4.4E-04
2 – 8 hours	6.9E-04	9.6E-04	2.8E-04
8 – 24 hours	2.8E-04	3.9E-04	1.1E-04
1 – 4 days	2.1E-04	3.2E-04	8.5E-05
4 – 30 days	1.3E-04	2.4E-04	5.0E-05

Time Interval	Main Steam Relief Valves	Main Steam Safety Valves	Containment Shell
0 – 2 hours	1.3E-03	1.3E-03	8.0-E-04
2 – 8 hours	9.3E-04	9.6E-04	5.1E-04
8 – 24 hours	3.8E-04	3.9E-04	2.3E-04
1 – 4 days	2.7E-04	2.7E-04	1.6E-04
4 – 30 days	1.9E-04	2.0E-04	1.1E-04

CP COL 2.3(3)

Table 2.3-340 (Sheet 1 of 3)
Annual Average χ/Q (sec/m^3) for No Decay, Undepleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.31E-06	1.10E-06	6.09E-07	4.00E-07	2.19E-07	1.42E-07	1.02E-07	7.70E-08	6.10E-08	4.99E-08	4.17E-08
SSW	2.66E-06	8.71E-07	4.82E-07	3.16E-07	1.73E-07	1.12E-07	8.02E-08	6.09E-08	4.83E-08	3.95E-08	3.31E-08
SW	2.03E-06	6.61E-07	3.61E-07	2.35E-07	1.27E-07	8.16E-08	5.79E-08	4.38E-08	3.46E-08	2.82E-08	2.35E-08
WSW	1.93E-06	6.36E-07	3.49E-07	2.27E-07	1.23E-07	7.89E-08	5.60E-08	4.23E-08	3.34E-08	2.72E-08	2.27E-08
W	2.71E-06	8.79E-07	4.88E-07	3.22E-07	1.77E-07	1.16E-07	8.28E-08	6.30E-08	5.01E-08	4.10E-08	3.44E-08
WNW	4.41E-06	1.39E-06	7.70E-07	5.08E-07	2.83E-07	1.86E-07	1.35E-07	1.03E-07	8.24E-08	6.78E-08	5.72E-08
NW	8.85E-06	2.75E-06	1.51E-06	1.00E-06	5.65E-07	3.75E-07	2.73E-07	2.11E-07	1.69E-07	1.40E-07	1.18E-07
NNW	1.11E-05	3.44E-06	1.89E-06	1.25E-06	7.05E-07	4.67E-07	3.39E-07	2.62E-07	2.10E-07	1.74E-07	1.47E-07
N	7.85E-06	2.45E-06	1.32E-06	8.55E-07	4.76E-07	3.14E-07	2.28E-07	1.75E-07	1.41E-07	1.16E-07	9.85E-08
NNE	7.05E-06	2.12E-06	1.12E-06	7.24E-07	4.09E-07	2.73E-07	2.00E-07	1.57E-07	1.27E-07	1.06E-07	9.07E-08
NE	6.25E-06	1.86E-06	9.67E-07	6.25E-07	3.53E-07	2.36E-07	1.74E-07	1.36E-07	1.11E-07	9.32E-08	7.98E-08
ENE	4.86E-06	1.44E-06	7.25E-07	4.59E-07	2.57E-07	1.72E-07	1.27E-07	1.00E-07	8.24E-08	6.96E-08	5.99E-08
E	2.57E-06	7.66E-07	3.89E-07	2.48E-07	1.40E-07	9.40E-08	6.93E-08	5.48E-08	4.49E-08	3.78E-08	3.25E-08
ESE	3.44E-06	1.03E-06	5.31E-07	3.41E-07	1.92E-07	1.29E-07	9.45E-08	7.45E-08	6.09E-08	5.11E-08	4.39E-08
SE	4.47E-06	1.35E-06	7.04E-07	4.53E-07	2.56E-07	1.71E-07	1.25E-07	9.83E-08	8.01E-08	6.70E-08	5.73E-08
SSE	2.61E-06	8.39E-07	4.58E-07	2.98E-07	1.64E-07	1.07E-07	7.67E-08	5.85E-08	4.66E-08	3.82E-08	3.21E-08

CP COL 2.3(3)

Table 2.3-340 (Sheet 2 of 3)
Annual Average χ/Q (sec/m^3) for No Decay, Undepleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.56E-08	1.94E-08	1.26E-08	6.93E-09	4.56E-09	3.29E-09	2.53E-09	2.02E-09	1.67E-09	1.41E-09	1.21E-09
SSW	2.82E-08	1.54E-08	1.01E-08	5.55E-09	3.66E-09	2.66E-09	2.05E-09	1.64E-09	1.36E-09	1.15E-09	9.90E-10
SW	2.01E-08	1.09E-08	7.08E-09	3.89E-09	2.57E-09	1.87E-09	1.44E-09	1.16E-09	9.59E-10	8.13E-10	7.01E-10
WSW	1.93E-08	1.05E-08	6.79E-09	3.72E-09	2.45E-09	1.77E-09	1.36E-09	1.09E-09	9.02E-10	7.63E-10	6.57E-10
W	2.95E-08	1.62E-08	1.06E-08	5.87E-09	3.88E-09	2.82E-09	2.18E-09	1.75E-09	1.45E-09	1.23E-09	1.06E-09
WNW	4.91E-08	2.74E-08	1.81E-08	1.02E-08	6.80E-09	4.98E-09	3.86E-09	3.12E-09	2.59E-09	2.20E-09	1.91E-09
NW	1.02E-07	5.77E-08	3.85E-08	2.19E-08	1.48E-08	1.09E-08	8.46E-09	6.86E-09	5.72E-09	4.88E-09	4.23E-09
NNW	1.27E-07	7.14E-08	4.77E-08	2.71E-08	1.82E-08	1.34E-08	1.05E-08	8.48E-09	7.08E-09	6.03E-09	5.23E-09
N	8.48E-08	4.80E-08	3.22E-08	1.84E-08	1.24E-08	9.19E-09	7.19E-09	5.85E-09	4.89E-09	4.18E-09	3.64E-09
NNE	7.88E-08	4.59E-08	3.14E-08	1.84E-08	1.27E-08	9.50E-09	7.51E-09	6.17E-09	5.20E-09	4.47E-09	3.91E-09
NE	6.95E-08	4.09E-08	2.81E-08	1.67E-08	1.15E-08	8.69E-09	6.89E-09	5.67E-09	4.80E-09	4.14E-09	3.62E-09
ENE	5.24E-08	3.14E-08	2.19E-08	1.32E-08	9.23E-09	7.01E-09	5.61E-09	4.64E-09	3.94E-09	3.42E-09	3.01E-09
E	2.84E-08	1.69E-08	1.17E-08	7.01E-09	4.89E-09	3.70E-09	2.95E-09	2.43E-09	2.06E-09	1.78E-09	1.57E-09
ESE	3.82E-08	2.26E-08	1.56E-08	9.27E-09	6.43E-09	4.85E-09	3.86E-09	3.18E-09	2.69E-09	2.32E-09	2.04E-09
SE	4.98E-08	2.92E-08	2.00E-08	1.18E-08	8.13E-09	6.11E-09	4.84E-09	3.97E-09	3.35E-09	2.89E-09	2.53E-09
SSE	2.75E-08	1.52E-08	1.00E-08	5.61E-09	3.74E-09	2.73E-09	2.12E-09	1.71E-09	1.42E-09	1.21E-09	1.05E-09

CP COL 2.3(3)

Table 2.3-340 (Sheet 3 of 3)
Annual Average χ/Q (sec/m^3) for No Decay, Undepleted

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	6.24E-07	2.25E-07	1.03E-07	6.13E-08	4.19E-08	2.00E-08	7.14E-09	3.32E-09	2.03E-09	1.41E-09
SSW	4.95E-07	1.78E-07	8.10E-08	4.85E-08	3.32E-08	1.59E-08	5.72E-09	2.68E-09	1.65E-09	1.15E-09
SW	3.72E-07	1.31E-07	5.86E-08	3.48E-08	2.36E-08	1.12E-08	4.02E-09	1.89E-09	1.16E-09	8.15E-10
WSW	3.58E-07	1.26E-07	5.67E-08	3.36E-08	2.28E-08	1.08E-08	3.84E-09	1.79E-09	1.10E-09	7.65E-10
W	5.01E-07	1.82E-07	8.36E-08	5.03E-08	3.45E-08	1.66E-08	6.04E-09	2.85E-09	1.76E-09	1.23E-09
WNW	7.92E-07	2.90E-07	1.36E-07	8.28E-08	5.73E-08	2.81E-08	1.04E-08	5.02E-09	3.13E-09	2.21E-09
NW	1.56E-06	5.77E-07	2.75E-07	1.70E-07	1.19E-07	5.90E-08	2.24E-08	1.09E-08	6.88E-09	4.89E-09
NNW	1.95E-06	7.21E-07	3.42E-07	2.11E-07	1.47E-07	7.31E-08	2.77E-08	1.35E-08	8.51E-09	6.04E-09
N	1.36E-06	4.88E-07	2.30E-07	1.41E-07	9.87E-08	4.91E-08	1.88E-08	9.25E-09	5.87E-09	4.19E-09
NNE	1.16E-06	4.18E-07	2.02E-07	1.28E-07	9.09E-08	4.68E-08	1.88E-08	9.55E-09	6.18E-09	4.48E-09
NE	1.01E-06	3.61E-07	1.75E-07	1.12E-07	7.99E-08	4.16E-08	1.69E-08	8.73E-09	5.69E-09	4.14E-09
ENE	7.66E-07	2.64E-07	1.28E-07	8.26E-08	6.00E-08	3.18E-08	1.34E-08	7.04E-09	4.65E-09	3.42E-09
E	4.10E-07	1.43E-07	7.01E-08	4.51E-08	3.26E-08	1.72E-08	7.11E-09	3.71E-09	2.44E-09	1.79E-09
ESE	5.58E-07	1.97E-07	9.56E-08	6.10E-08	4.39E-08	2.30E-08	9.42E-09	4.87E-09	3.18E-09	2.32E-09
SE	7.37E-07	2.62E-07	1.27E-07	8.03E-08	5.74E-08	2.97E-08	1.20E-08	6.14E-09	3.98E-09	2.89E-09
SSE	4.72E-07	1.69E-07	7.75E-08	4.68E-08	3.22E-08	1.56E-08	5.76E-09	2.76E-09	1.72E-09	1.21E-09

CP COL 2.3(3)

Table 2.3-341 (Sheet 1 of 3)
Annual Average χ/Q (sec/m^3) For No Decay, Depleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.08E-06	9.96E-07	5.43E-07	3.50E-07	1.86E-07	1.18E-07	8.22E-08	6.12E-08	4.76E-08	3.83E-08	3.16E-08
SSW	2.48E-06	7.92E-07	4.30E-07	2.77E-07	1.47E-07	9.30E-08	6.50E-08	4.84E-08	3.77E-08	3.03E-08	2.50E-08
SW	1.89E-06	6.01E-07	3.22E-07	2.05E-07	1.08E-07	6.76E-08	4.69E-08	3.48E-08	2.70E-08	2.16E-08	1.78E-08
WSW	1.80E-06	5.78E-07	3.11E-07	1.99E-07	1.04E-07	6.54E-08	4.54E-08	3.36E-08	2.61E-08	2.09E-08	1.72E-08
W	2.52E-06	7.99E-07	4.35E-07	2.81E-07	1.50E-07	9.56E-08	6.71E-08	5.01E-08	3.91E-08	3.15E-08	2.60E-08
WNW	4.11E-06	1.27E-06	6.86E-07	4.45E-07	2.41E-07	1.54E-07	1.09E-07	8.19E-08	6.43E-08	5.21E-08	4.32E-08
NW	8.23E-06	2.50E-06	1.35E-06	8.76E-07	4.80E-07	3.11E-07	2.21E-07	1.67E-07	1.32E-07	1.07E-07	8.96E-08
NNW	1.03E-05	3.13E-06	1.69E-06	1.10E-06	5.98E-07	3.87E-07	2.75E-07	2.08E-07	1.64E-07	1.33E-07	1.11E-07
N	7.31E-06	2.23E-06	1.17E-06	7.48E-07	4.05E-07	2.60E-07	1.84E-07	1.39E-07	1.10E-07	8.94E-08	7.44E-08
NNE	6.56E-06	1.92E-06	9.93E-07	6.33E-07	3.47E-07	2.26E-07	1.62E-07	1.24E-07	9.92E-08	8.15E-08	6.86E-08
NE	5.82E-06	1.69E-06	8.62E-07	5.47E-07	3.00E-07	1.95E-07	1.41E-07	1.08E-07	8.67E-08	7.15E-08	6.03E-08
ENE	4.53E-06	1.31E-06	6.46E-07	4.02E-07	2.18E-07	1.42E-07	1.03E-07	7.97E-08	6.43E-08	5.34E-08	4.53E-08
E	2.39E-06	6.97E-07	3.47E-07	2.17E-07	1.19E-07	7.78E-08	5.62E-08	4.35E-08	3.51E-08	2.90E-08	2.46E-08
ESE	3.20E-06	9.39E-07	4.74E-07	2.98E-07	1.63E-07	1.06E-07	7.66E-08	5.91E-08	4.75E-08	3.93E-08	3.32E-08
SE	4.16E-06	1.23E-06	6.27E-07	3.97E-07	2.17E-07	1.42E-07	1.02E-07	7.81E-08	6.25E-08	5.14E-08	4.33E-08
SSE	2.43E-06	7.63E-07	4.08E-07	2.61E-07	1.39E-07	8.86E-08	6.21E-08	4.65E-08	3.63E-08	2.93E-08	2.43E-08

CP COL 2.3(3)

Table 2.3-341 (Sheet 2 of 3)
Annual Average χ/Q (sec/m^3) For No Decay, Depleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	2.66E-08	1.37E-08	8.49E-09	4.31E-09	2.66E-09	1.82E-09	1.33E-09	1.02E-09	8.10E-10	6.58E-10	5.46E-10
SSW	2.10E-08	1.09E-08	6.77E-09	3.45E-09	2.14E-09	1.47E-09	1.08E-09	8.30E-10	6.59E-10	5.37E-10	4.46E-10
SW	1.50E-08	7.68E-09	4.76E-09	2.42E-09	1.50E-09	1.03E-09	7.60E-10	5.85E-10	4.65E-10	3.80E-10	3.16E-10
WSW	1.44E-08	7.38E-09	4.56E-09	2.31E-09	1.43E-09	9.80E-10	7.18E-10	5.51E-10	4.38E-10	3.56E-10	2.96E-10
W	2.20E-08	1.14E-08	7.12E-09	3.65E-09	2.27E-09	1.56E-09	1.15E-09	8.83E-10	7.02E-10	5.73E-10	4.76E-10
WNW	3.66E-08	1.93E-08	1.22E-08	6.33E-09	3.97E-09	2.75E-09	2.04E-09	1.58E-09	1.26E-09	1.03E-09	8.59E-10
NW	7.61E-08	4.07E-08	2.59E-08	1.36E-08	8.61E-09	6.01E-09	4.46E-09	3.46E-09	2.77E-09	2.28E-09	1.91E-09
NNW	9.43E-08	5.03E-08	3.20E-08	1.69E-08	1.07E-08	7.43E-09	5.52E-09	4.28E-09	3.43E-09	2.82E-09	2.36E-09
N	6.32E-08	3.38E-08	2.16E-08	1.14E-08	7.25E-09	5.08E-09	3.79E-09	2.95E-09	2.37E-09	1.95E-09	1.64E-09
NNE	5.87E-08	3.24E-08	2.11E-08	1.15E-08	7.40E-09	5.26E-09	3.96E-09	3.11E-09	2.52E-09	2.09E-09	1.76E-09
NE	5.18E-08	2.88E-08	1.89E-08	1.04E-08	6.74E-09	4.81E-09	3.64E-09	2.87E-09	2.33E-09	1.93E-09	1.63E-09
ENE	3.91E-08	2.21E-08	1.47E-08	8.20E-09	5.39E-09	3.88E-09	2.96E-09	2.34E-09	1.91E-09	1.60E-09	1.35E-09
E	2.12E-08	1.19E-08	7.87E-09	4.36E-09	2.85E-09	2.05E-09	1.55E-09	1.23E-09	1.00E-09	8.32E-10	7.05E-10
ESE	2.85E-08	1.59E-08	1.05E-08	5.77E-09	3.76E-09	2.68E-09	2.03E-09	1.60E-09	1.30E-09	1.08E-09	9.17E-10
SE	3.71E-08	2.06E-08	1.34E-08	7.33E-09	4.75E-09	3.38E-09	2.55E-09	2.01E-09	1.63E-09	1.35E-09	1.14E-09
SSE	2.05E-08	1.07E-08	6.74E-09	3.49E-09	2.18E-09	1.51E-09	1.12E-09	8.64E-10	6.90E-10	5.65E-10	4.72E-10

CP COL 2.3(3)

Table 2.3-341 (Sheet 3 of 3) Annual Average χ/Q (sec/m^3) For No Decay, Depleted

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	5.58E-07	1.92E-07	8.32E-08	4.79E-08	3.17E-08	1.42E-08	4.51E-09	1.85E-09	1.03E-09	6.62E-10
SSW	4.42E-07	1.52E-07	6.58E-08	3.79E-08	2.51E-08	1.13E-08	3.61E-09	1.49E-09	8.36E-10	5.39E-10
SW	3.32E-07	1.12E-07	4.76E-08	2.72E-08	1.79E-08	8.00E-09	2.53E-09	1.05E-09	5.89E-10	3.81E-10
WSW	3.20E-07	1.08E-07	4.60E-08	2.62E-08	1.73E-08	7.69E-09	2.42E-09	9.95E-10	5.56E-10	3.58E-10
W	4.48E-07	1.55E-07	6.79E-08	3.93E-08	2.61E-08	1.18E-08	3.81E-09	1.58E-09	8.90E-10	5.75E-10
WNW	7.08E-07	2.48E-07	1.10E-07	6.47E-08	4.34E-08	2.00E-08	6.58E-09	2.79E-09	1.59E-09	1.03E-09
NW	1.39E-06	4.93E-07	2.23E-07	1.33E-07	8.99E-08	4.20E-08	1.41E-08	6.08E-09	3.49E-09	2.29E-09
NNW	1.74E-06	6.15E-07	2.78E-07	1.65E-07	1.12E-07	5.20E-08	1.75E-08	7.52E-09	4.31E-09	2.83E-09
N	1.22E-06	4.17E-07	1.87E-07	1.11E-07	7.47E-08	3.49E-08	1.18E-08	5.15E-09	2.97E-09	1.96E-09
NNE	1.04E-06	3.57E-07	1.64E-07	9.96E-08	6.88E-08	3.32E-08	1.18E-08	5.31E-09	3.13E-09	2.10E-09
NE	9.06E-07	3.08E-07	1.42E-07	8.71E-08	6.05E-08	2.95E-08	1.07E-08	4.85E-09	2.88E-09	1.94E-09
ENE	6.85E-07	2.25E-07	1.04E-07	6.46E-08	4.54E-08	2.26E-08	8.40E-09	3.91E-09	2.35E-09	1.60E-09
E	3.67E-07	1.22E-07	5.69E-08	3.52E-08	2.46E-08	1.22E-08	4.47E-09	2.06E-09	1.23E-09	8.35E-10
ESE	4.99E-07	1.68E-07	7.76E-08	4.77E-08	3.32E-08	1.63E-08	5.92E-09	2.71E-09	1.61E-09	1.09E-09
SE	6.59E-07	2.23E-07	1.03E-07	6.27E-08	4.34E-08	2.11E-08	7.54E-09	3.41E-09	2.02E-09	1.35E-09
SSE	4.22E-07	1.44E-07	6.29E-08	3.66E-08	2.44E-08	1.11E-08	3.63E-09	1.53E-09	8.71E-10	5.68E-10

CP COL 2.3(3)

Table 2.3-342 (Sheet 1 of 2)
χ/Q and D/Q Values for Normal Releases

No Decay, Undepleted and Depleted, at Each Receptor Location

Type of Location	Sector	Distance (mi)	Distance (meters)	χ/Q (m^3) No Decay Undepleted	χ/Q (m^3) No Decay Depleted	D/Q (m^{-2})
EAB	S	0.37	600	1.70E-06	1.50E-06	2.30E-08
EAB	SSW	0.37	600	1.30E-06	1.20E-06	1.50E-08
EAB	SW	0.37	600	1.00E-06	9.40E-07	1.10E-08
EAB	WSW	0.37	600	9.80E-07	9.00E-07	9.10E-09
EAB	W	0.37	600	1.40E-06	1.30E-06	1.10E-08
EAB	WNW	0.37	600	2.20E-06	2.00E-06	1.70E-08
EAB	NW	0.37	600	4.40E-06	4.10E-06	3.80E-08
EAB	NNW	0.37	600	5.50E-06	5.10E-06	5.50E-08
EAB	N	0.37	600	3.90E-06	3.60E-06	4.90E-08
EAB	NNE	0.37	600	3.50E-06	3.20E-06	1.90E-08
EAB	NE	0.37	600	3.10E-06	2.80E-06	1.20E-08
EAB	ENE	0.37	600	2.40E-06	2.20E-06	9.00E-09
EAB	E	0.37	600	1.30E-06	1.20E-06	4.00E-09
EAB	ESE	0.37	600	1.70E-06	1.60E-06	7.50E-09
EAB	SE	0.37	600	2.20E-06	2.00E-06	1.40E-08
EAB	SSE	0.37	600	1.30E-06	1.20E-06	1.90E-08
Residence	S	1.09	1753	3.50E-07	3.10E-07	3.90E-09
Residence	SSW	0.79	1276	4.40E-07	3.90E-07	4.50E-09
Residence	SW	0.79	1276	3.30E-07	3.00E-07	3.10E-09
Residence	WSW	1.16	1869	1.80E-07	1.60E-07	1.40E-09

CP COL 2.3(3)

Table 2.3-342 (Sheet 2 of 2)
χ/Q and D/Q Values for Normal Releases

No Decay, Undepleted and Depleted, at Each Receptor Location

Type of Location	Sector	Distance (mi)	Distance (meters)	χ/Q (m^3) No Decay Undepleted	χ/Q (m^3) No Decay Depleted	D/Q (m^{-2})
Residence	W	1.16	1869	2.60E-07	2.20E-07	1.60E-09
Residence	WNW	2.26	3645	1.60E-07	1.30E-07	8.00E-10
Residence	NW	2.18	3515	3.30E-07	2.70E-07	1.90E-09
Residence	NNW	2.18	3515	4.10E-07	3.40E-07	2.80E-09
Residence	N	1.99	3202	3.20E-07	2.60E-07	2.90E-09
Residence	NNE	1.99	3202	2.70E-07	2.30E-07	1.20E-09
Residence	NE	2.39	3853	1.80E-07	1.50E-07	5.20E-10
Residence	ENE	2.4	3863	1.30E-07	1.10E-07	3.90E-10
Residence	E	2.76	4449	6.10E-08	4.90E-08	1.40E-10
Residence	ESE	2.43	3903	9.80E-08	8.00E-08	3.20E-10
Residence	SE	1.95	3146	1.80E-07	1.50E-07	8.70E-10
Residence	SSE	1.83	2942	1.20E-07	1.00E-07	1.30E-09
Garden	ENE	2.86	4609	1.10E-07	8.50E-08	2.90E-10
Garden	E	2.86	4609	5.80E-08	4.60E-08	1.30E-10

CP COL 2.3(3)

Table 2.3-343 (Sheet 1 of 3) Annual Average χ/Q (sec/m^3) For a 2.26 Day Decay, Undepleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.31E-06	1.09E-06	6.08E-07	3.99E-07	2.18E-07	1.42E-07	1.01E-07	7.65E-08	6.06E-08	4.94E-08	4.13E-08
SSW	2.66E-06	8.69E-07	4.81E-07	3.15E-07	1.72E-07	1.12E-07	7.97E-08	6.05E-08	4.79E-08	3.91E-08	3.27E-08
SW	2.03E-06	6.60E-07	3.61E-07	2.34E-07	1.26E-07	8.11E-08	5.75E-08	4.34E-08	3.42E-08	2.78E-08	2.32E-08
WSW	1.93E-06	6.35E-07	3.48E-07	2.26E-07	1.22E-07	7.84E-08	5.56E-08	4.19E-08	3.30E-08	2.68E-08	2.24E-08
W	2.71E-06	8.77E-07	4.87E-07	3.21E-07	1.76E-07	1.15E-07	8.21E-08	6.25E-08	4.95E-08	4.05E-08	3.40E-08
WNW	4.41E-06	1.39E-06	7.68E-07	5.07E-07	2.82E-07	1.85E-07	1.34E-07	1.02E-07	8.15E-08	6.70E-08	5.64E-08
NW	8.84E-06	2.74E-06	1.51E-06	9.98E-07	5.63E-07	3.73E-07	2.71E-07	2.09E-07	1.68E-07	1.39E-07	1.17E-07
NNW	1.11E-05	3.43E-06	1.89E-06	1.25E-06	7.00E-07	4.63E-07	3.36E-07	2.58E-07	2.07E-07	1.71E-07	1.44E-07
N	7.85E-06	2.45E-06	1.31E-06	8.53E-07	4.75E-07	3.13E-07	2.26E-07	1.74E-07	1.40E-07	1.15E-07	9.73E-08
NNE	7.04E-06	2.11E-06	1.11E-06	7.18E-07	4.04E-07	2.69E-07	1.97E-07	1.53E-07	1.24E-07	1.03E-07	8.78E-08
NE	6.24E-06	1.85E-06	9.62E-07	6.20E-07	3.48E-07	2.32E-07	1.70E-07	1.33E-07	1.08E-07	9.02E-08	7.69E-08
ENE	4.86E-06	1.44E-06	7.22E-07	4.57E-07	2.56E-07	1.70E-07	1.25E-07	9.91E-08	8.13E-08	6.85E-08	5.88E-08
E	2.57E-06	7.64E-07	3.88E-07	2.46E-07	1.39E-07	9.31E-08	6.85E-08	5.41E-08	4.42E-08	3.71E-08	3.18E-08
ESE	3.43E-06	1.03E-06	5.30E-07	3.39E-07	1.91E-07	1.27E-07	9.36E-08	7.36E-08	6.00E-08	5.03E-08	4.30E-08
SE	4.47E-06	1.35E-06	7.02E-07	4.52E-07	2.55E-07	1.70E-07	1.25E-07	9.74E-08	7.92E-08	6.62E-08	5.65E-08
SSE	2.61E-06	8.38E-07	4.57E-07	2.98E-07	1.63E-07	1.06E-07	7.62E-08	5.81E-08	4.62E-08	3.79E-08	3.18E-08

CP COL 2.3(3)

Table 2.3-343 (Sheet 2 of 3)
Annual Average χ/Q (sec/m^3) For a 2.26 Day Decay, Undepleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.52E-08	1.91E-08	1.24E-08	6.71E-09	4.36E-09	3.12E-09	2.37E-09	1.87E-09	1.53E-09	1.28E-09	1.09E-09
SSW	2.79E-08	1.51E-08	9.82E-09	5.35E-09	3.49E-09	2.50E-09	1.90E-09	1.51E-09	1.23E-09	1.03E-09	8.76E-10
SW	1.98E-08	1.07E-08	6.88E-09	3.72E-09	2.42E-09	1.74E-09	1.32E-09	1.04E-09	8.52E-10	7.12E-10	6.05E-10
WSW	1.90E-08	1.02E-08	6.57E-09	3.54E-09	2.29E-09	1.63E-09	1.23E-09	9.74E-10	7.92E-10	6.59E-10	5.59E-10
W	2.90E-08	1.58E-08	1.03E-08	5.60E-09	3.65E-09	2.61E-09	1.98E-09	1.57E-09	1.28E-09	1.07E-09	9.05E-10
WNW	4.83E-08	2.67E-08	1.76E-08	9.71E-09	6.38E-09	4.60E-09	3.51E-09	2.79E-09	2.29E-09	1.91E-09	1.63E-09
NW	1.01E-07	5.66E-08	3.76E-08	2.11E-08	1.40E-08	1.02E-08	7.84E-09	6.28E-09	5.17E-09	4.35E-09	3.73E-09
NNW	1.24E-07	6.90E-08	4.55E-08	2.53E-08	1.67E-08	1.20E-08	9.20E-09	7.32E-09	6.00E-09	5.03E-09	4.29E-09
N	8.38E-08	4.71E-08	3.13E-08	1.77E-08	1.18E-08	8.60E-09	6.64E-09	5.33E-09	4.40E-09	3.71E-09	3.18E-09
NNE	7.60E-08	4.36E-08	2.93E-08	1.67E-08	1.12E-08	8.13E-09	6.26E-09	5.01E-09	4.12E-09	3.46E-09	2.96E-09
NE	6.67E-08	3.86E-08	2.61E-08	1.49E-08	1.00E-08	7.30E-09	5.62E-09	4.50E-09	3.70E-09	3.11E-09	2.66E-09
ENE	5.14E-08	3.05E-08	2.10E-08	1.24E-08	8.54E-09	6.36E-09	4.99E-09	4.05E-09	3.38E-09	2.87E-09	2.48E-09
E	2.77E-08	1.63E-08	1.12E-08	6.54E-09	4.45E-09	3.29E-09	2.56E-09	2.07E-09	1.71E-09	1.45E-09	1.24E-09
ESE	3.74E-08	2.19E-08	1.50E-08	8.70E-09	5.91E-09	4.36E-09	3.39E-09	2.74E-09	2.27E-09	1.92E-09	1.64E-09
SE	4.90E-08	2.85E-08	1.93E-08	1.12E-08	7.61E-09	5.62E-09	4.38E-09	3.54E-09	2.94E-09	2.49E-09	2.15E-09
SSE	2.72E-08	1.49E-08	9.78E-09	5.40E-09	3.56E-09	2.57E-09	1.97E-09	1.57E-09	1.29E-09	1.08E-09	9.24E-10

CP COL 2.3(3)

Table 2.3-343 (Sheet 3 of 3)
Annual Average χ/Q (sec/m^3) For a 2.26 Day Decay, Undepleted

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	6.23E-07	2.24E-07	1.02E-07	6.09E-08	4.15E-08	1.97E-08	6.92E-09	3.15E-09	1.88E-09	1.28E-09
SSW	4.94E-07	1.77E-07	8.05E-08	4.81E-08	3.28E-08	1.56E-08	5.52E-09	2.52E-09	1.51E-09	1.03E-09
SW	3.71E-07	1.30E-07	5.82E-08	3.44E-08	2.33E-08	1.10E-08	3.85E-09	1.75E-09	1.05E-09	7.14E-10
WSW	3.58E-07	1.26E-07	5.62E-08	3.32E-08	2.25E-08	1.06E-08	3.66E-09	1.65E-09	9.79E-10	6.61E-10
W	5.00E-07	1.81E-07	8.30E-08	4.98E-08	3.41E-08	1.63E-08	5.77E-09	2.64E-09	1.58E-09	1.07E-09
WNW	7.90E-07	2.89E-07	1.35E-07	8.19E-08	5.65E-08	2.74E-08	9.97E-09	4.64E-09	2.80E-09	1.92E-09
NW	1.56E-06	5.75E-07	2.74E-07	1.68E-07	1.17E-07	5.80E-08	2.16E-08	1.03E-08	6.30E-09	4.36E-09
NNW	1.95E-06	7.16E-07	3.39E-07	2.08E-07	1.44E-07	7.07E-08	2.60E-08	1.21E-08	7.36E-09	5.04E-09
N	1.36E-06	4.87E-07	2.28E-07	1.40E-07	9.76E-08	4.82E-08	1.81E-08	8.66E-09	5.35E-09	3.72E-09
NNE	1.16E-06	4.14E-07	1.99E-07	1.24E-07	8.80E-08	4.44E-08	1.70E-08	8.19E-09	5.03E-09	3.47E-09
NE	1.01E-06	3.57E-07	1.72E-07	1.08E-07	7.71E-08	3.93E-08	1.52E-08	7.35E-09	4.52E-09	3.12E-09
ENE	7.64E-07	2.63E-07	1.27E-07	8.15E-08	5.89E-08	3.09E-08	1.26E-08	6.39E-09	4.07E-09	2.88E-09
E	4.09E-07	1.43E-07	6.93E-08	4.43E-08	3.19E-08	1.66E-08	6.64E-09	3.31E-09	2.07E-09	1.45E-09
ESE	5.56E-07	1.96E-07	9.46E-08	6.02E-08	4.31E-08	2.23E-08	8.85E-09	4.39E-09	2.74E-09	1.92E-09
SE	7.35E-07	2.61E-07	1.26E-07	7.94E-08	5.66E-08	2.90E-08	1.14E-08	5.66E-09	3.55E-09	2.50E-09
SSE	4.71E-07	1.68E-07	7.70E-08	4.64E-08	3.19E-08	1.54E-08	5.55E-09	2.59E-09	1.58E-09	1.08E-09

CP COL 2.3(3)

Table 2.3-344 (Sheet 1 of 3)
Annual Average χ/Q (sec/M^3) for an 8.00 Day Decay, Depleted

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	3.08E-06	9.96E-07	5.43E-07	3.50E-07	1.86E-07	1.18E-07	8.21E-08	6.11E-08	4.75E-08	3.82E-08	3.15E-08
SSW	2.48E-06	7.92E-07	4.30E-07	2.77E-07	1.47E-07	9.28E-08	6.48E-08	4.83E-08	3.76E-08	3.02E-08	2.49E-08
SW	1.89E-06	6.01E-07	3.22E-07	2.05E-07	1.08E-07	6.74E-08	4.68E-08	3.47E-08	2.69E-08	2.16E-08	1.77E-08
WSW	1.80E-06	5.78E-07	3.11E-07	1.98E-07	1.04E-07	6.52E-08	4.53E-08	3.35E-08	2.60E-08	2.08E-08	1.71E-08
W	2.52E-06	7.99E-07	4.35E-07	2.81E-07	1.50E-07	9.54E-08	6.69E-08	4.99E-08	3.90E-08	3.14E-08	2.59E-08
WNW	4.11E-06	1.27E-06	6.86E-07	4.44E-07	2.40E-07	1.54E-07	1.09E-07	8.17E-08	6.41E-08	5.19E-08	4.30E-08
NW	8.23E-06	2.50E-06	1.35E-06	8.75E-07	4.79E-07	3.10E-07	2.21E-07	1.67E-07	1.32E-07	1.07E-07	8.93E-08
NNW	1.03E-05	3.13E-06	1.69E-06	1.09E-06	5.97E-07	3.86E-07	2.74E-07	2.07E-07	1.63E-07	1.33E-07	1.10E-07
N	7.31E-06	2.23E-06	1.17E-06	7.48E-07	4.04E-07	2.60E-07	1.84E-07	1.39E-07	1.10E-07	8.91E-08	7.42E-08
NNE	6.56E-06	1.92E-06	9.92E-07	6.32E-07	3.46E-07	2.25E-07	1.61E-07	1.24E-07	9.85E-08	8.09E-08	6.79E-08
NE	5.81E-06	1.69E-06	8.61E-07	5.46E-07	2.99E-07	1.95E-07	1.40E-07	1.08E-07	8.60E-08	7.09E-08	5.97E-08
ENE	4.52E-06	1.31E-06	6.45E-07	4.01E-07	2.18E-07	1.42E-07	1.02E-07	7.94E-08	6.41E-08	5.32E-08	4.51E-08
E	2.39E-06	6.96E-07	3.46E-07	2.16E-07	1.19E-07	7.76E-08	5.60E-08	4.34E-08	3.49E-08	2.89E-08	2.44E-08
ESE	3.20E-06	9.39E-07	4.73E-07	2.98E-07	1.63E-07	1.06E-07	7.63E-08	5.89E-08	4.73E-08	3.91E-08	3.30E-08
SE	4.16E-06	1.23E-06	6.27E-07	3.96E-07	2.17E-07	1.41E-07	1.01E-07	7.79E-08	6.23E-08	5.13E-08	4.31E-08
SSE	2.43E-06	7.63E-07	4.08E-07	2.61E-07	1.39E-07	8.84E-08	6.20E-08	4.64E-08	3.63E-08	2.93E-08	2.42E-08

CP COL 2.3(3)

Table 2.3-344 (Sheet 2 of 3)
Annual Average X/Q (sec/M^3) for an 8.00 Day Decay, Depleted

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	2.65E-08	1.36E-08	8.44E-09	4.27E-09	2.63E-09	1.79E-09	1.31E-09	9.99E-10	7.89E-10	6.40E-10	5.29E-10
SSW	2.10E-08	1.08E-08	6.72E-09	3.42E-09	2.11E-09	1.45E-09	1.06E-09	8.09E-10	6.40E-10	5.20E-10	4.31E-10
SW	1.49E-08	7.63E-09	4.72E-09	2.39E-09	1.48E-09	1.01E-09	7.41E-10	5.68E-10	4.50E-10	3.65E-10	3.03E-10
WSW	1.44E-08	7.33E-09	4.52E-09	2.28E-09	1.40E-09	9.57E-10	6.98E-10	5.34E-10	4.22E-10	3.42E-10	2.83E-10
W	2.19E-08	1.13E-08	7.05E-09	3.60E-09	2.23E-09	1.53E-09	1.12E-09	8.56E-10	6.78E-10	5.50E-10	4.56E-10
WNW	3.64E-08	1.92E-08	1.21E-08	6.25E-09	3.90E-09	2.69E-09	1.98E-09	1.53E-09	1.21E-09	9.88E-10	8.21E-10
NW	7.58E-08	4.04E-08	2.57E-08	1.35E-08	8.49E-09	5.90E-09	4.37E-09	3.38E-09	2.69E-09	2.20E-09	1.84E-09
NNW	9.37E-08	4.98E-08	3.16E-08	1.65E-08	1.04E-08	7.18E-09	5.30E-09	4.09E-09	3.25E-09	2.65E-09	2.21E-09
N	6.30E-08	3.37E-08	2.14E-08	1.13E-08	7.15E-09	4.99E-09	3.70E-09	2.87E-09	2.30E-09	1.89E-09	1.58E-09
NNE	5.81E-08	3.19E-08	2.07E-08	1.11E-08	7.12E-09	5.01E-09	3.75E-09	2.92E-09	2.34E-09	1.92E-09	1.61E-09
NE	5.12E-08	2.83E-08	1.85E-08	1.00E-08	6.46E-09	4.56E-09	3.42E-09	2.67E-09	2.14E-09	1.76E-09	1.48E-09
ENE	3.88E-08	2.20E-08	1.45E-08	8.06E-09	5.27E-09	3.77E-09	2.86E-09	2.25E-09	1.83E-09	1.52E-09	1.28E-09
E	2.10E-08	1.18E-08	7.77E-09	4.27E-09	2.78E-09	1.98E-09	1.49E-09	1.17E-09	9.48E-09	7.84E-10	6.60E-10
ESE	2.83E-08	1.58E-08	1.04E-08	5.66E-09	3.67E-09	2.60E-09	1.96E-09	1.54E-09	1.24E-09	1.03E-09	8.62E-10
SE	3.70E-08	2.04E-08	1.33E-08	7.23E-09	4.66E-09	3.30E-09	2.48E-09	1.94E-09	1.57E-09	1.29E-09	1.09E-09
SSE	2.04E-08	1.07E-08	6.69E-09	3.45E-09	2.15E-09	1.49E-09	1.10E-09	8.43E-10	6.71E-10	5.47E-10	4.55E-10

CP COL 2.3(3)

Table 2.3-344 (Sheet 3 of 3)
Annual Average X/Q (sec/M^3) for an 8.00 Day Decay, Depleted

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	5.58E-07	1.92E-07	8.31E-08	4.78E-08	3.16E-08	1.42E-08	4.47E-09	1.82E-09	1.01E-09	6.43E-10
SSW	4.42E-07	1.52E-07	6.57E-08	3.78E-08	2.50E-08	1.13E-08	3.57E-09	1.47E-09	8.16E-10	5.23E-10
SW	3.32E-07	1.11E-07	4.75E-08	2.71E-08	1.78E-08	7.95E-09	2.50E-09	1.03E-09	5.72E-10	3.67E-10
WSW	3.20E-07	1.08E-07	4.59E-08	2.62E-08	1.72E-08	7.64E-09	2.39E-09	9.72E-10	5.38E-10	3.43E-10
W	4.48E-07	1.55E-07	6.77E-08	3.92E-08	2.60E-08	1.18E-08	3.76E-09	1.55E-09	8.63E-10	5.53E-10
WNW	7.07E-07	2.47E-07	1.10E-07	6.45E-08	4.32E-08	1.98E-08	6.50E-09	2.73E-09	1.54E-09	9.93E-10
NW	1.39E-06	4.92E-07	2.23E-07	1.32E-07	8.96E-08	4.18E-08	1.40E-08	5.98E-09	3.40E-09	2.21E-09
NNW	1.74E-06	6.14E-07	2.77E-07	1.64E-07	1.11E-07	5.15E-08	1.71E-08	7.28E-09	4.12E-09	2.67E-09
N	1.22E-06	4.16E-07	1.86E-07	1.10E-07	7.45E-08	3.48E-08	1.17E-08	5.05E-09	2.89E-09	1.89E-09
NNE	1.04E-06	3.56E-07	1.63E-07	9.89E-08	6.81E-08	3.27E-08	1.15E-08	5.07E-09	2.93E-09	1.93E-09
NE	9.05E-07	3.07E-07	1.41E-07	8.64E-08	5.99E-08	2.90E-08	1.03E-08	4.61E-09	2.68E-09	1.77E-09
ENE	6.85E-07	2.25E-07	1.04E-07	6.43E-08	4.52E-08	2.24E-08	8.26E-09	3.81E-09	2.27E-09	1.52E-09
E	3.66E-07	1.22E-07	5.67E-08	3.50E-08	2.45E-08	1.21E-08	4.39E-09	2.00E-09	1.18E-09	7.87E-10
ESE	4.99E-07	1.68E-07	7.73E-08	4.75E-08	3.31E-08	1.62E-08	5.82E-09	2.63E-09	1.55E-09	1.03E-09
SE	6.58E-07	2.23E-07	1.03E-07	6.25E-08	4.33E-08	2.09E-08	7.44E-09	3.34E-09	1.95E-09	1.30E-09
SSE	4.22E-07	1.44E-07	6.28E-08	3.65E-08	2.43E-08	1.11E-08	3.59E-09	1.51E-09	8.49E-10	5.50E-10

CP COL 2.3(3)

Table 2.3-345 (Sheet 1 of 3)
D/Q (M-2) at Each 22.5° Sector for Each Distance (Mi) Shown at the Top

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	4.25E-08	1.44E-08	7.38E-09	4.53E-09	2.26E-09	1.37E-09	9.27E-10	6.71E-10	5.11E-10	4.02E-10	3.26E-10
SSW	2.85E-08	9.65E-09	4.96E-09	3.04E-09	1.52E-09	9.20E-10	6.22E-10	4.51E-10	3.43E-10	2.70E-10	2.19E-10
SW	1.99E-08	6.73E-09	3.45E-09	2.12E-09	1.06E-09	6.41E-10	4.34E-10	3.14E-10	2.39E-10	1.88E-10	1.52E-10
WSW	1.68E-08	5.69E-09	2.92E-09	1.80E-09	8.95E-10	5.43E-10	3.67E-10	2.66E-10	2.02E-10	1.59E-10	1.29E-10
W	1.95E-08	6.59E-09	3.39E-09	2.08E-09	1.04E-09	6.28E-10	4.25E-10	3.08E-10	2.34E-10	1.84E-10	1.49E-10
WNW	3.09E-08	1.04E-08	5.36E-09	3.29E-09	1.64E-09	9.95E-10	6.73E-10	4.88E-10	3.71E-10	2.92E-10	2.36E-10
NW	7.04E-08	2.38E-08	1.22E-08	7.50E-09	3.74E-09	2.27E-09	1.53E-09	1.11E-09	8.45E-10	6.66E-10	5.39E-10
NNW	1.02E-07	3.45E-08	1.77E-08	1.09E-08	5.43E-09	3.29E-09	2.23E-09	1.61E-09	1.23E-09	9.66E-10	7.82E-10
N	8.99E-08	3.04E-08	1.56E-08	9.58E-09	4.78E-09	2.90E-09	1.96E-09	1.42E-09	1.08E-09	8.50E-10	6.88E-10
NNE	3.60E-08	1.22E-08	6.26E-09	3.84E-09	1.92E-09	1.16E-09	7.85E-10	5.69E-10	4.33E-10	3.41E-10	2.76E-10
NE	2.20E-08	7.44E-09	3.82E-09	2.35E-09	1.17E-09	7.09E-10	4.80E-10	3.47E-10	2.64E-10	2.08E-10	1.69E-10
ENE	1.66E-08	5.62E-09	2.89E-09	1.77E-09	8.84E-10	5.36E-10	3.63E-10	2.63E-10	2.00E-10	1.57E-10	1.27E-10
E	7.45E-09	2.52E-09	1.29E-09	7.94E-10	3.96E-10	2.40E-10	1.62E-10	1.18E-10	8.95E-11	7.05E-11	5.71E-11
ESE	1.39E-08	4.70E-09	2.41E-09	1.48E-09	7.39E-10	4.48E-10	3.03E-10	2.20E-10	1.67E-10	1.32E-10	1.07E-10
SE	2.60E-08	8.80E-09	4.52E-09	2.77E-09	1.38E-09	8.39E-10	5.67E-10	4.11E-10	3.12E-10	2.46E-10	1.99E-10
SSE	3.47E-08	1.17E-08	6.02E-09	3.70E-09	1.84E-09	1.12E-09	7.56E-10	5.47E-10	4.16E-10	3.28E-10	2.66E-10

CP COL 2.3(3)

Table 2.3-345 (Sheet 2 of 3)
D/Q (M-2) at Each 22.5° Sector for Each Distance (Mi) Shown at the Top

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	2.69E-10	1.32E-10	8.28E-11	4.19E-11	2.53E-11	1.70E-11	1.22E-11	9.14E-12	7.11E-12	5.68E-12	4.63E-12
SSW	1.81E-10	8.86E-11	5.56E-11	2.81E-11	1.70E-11	1.14E-11	8.17E-12	6.14E-12	4.77E-12	3.81E-12	3.11E-12
SW	1.26E-10	6.18E-11	3.88E-11	1.96E-11	1.19E-11	7.95E-12	5.70E-12	4.28E-12	3.33E-12	2.66E-12	2.17E-12
WSW	1.07E-10	5.23E-11	3.28E-11	1.66E-11	1.00E-11	6.73E-12	4.82E-12	3.62E-12	2.81E-12	2.25E-12	1.84E-12
W	1.24E-10	6.05E-11	3.80E-11	1.92E-11	1.16E-11	7.79E-12	5.58E-12	4.19E-12	3.26E-12	2.60E-12	2.13E-12
WNW	1.96E-10	9.59E-11	6.01E-11	3.04E-11	1.84E-11	1.23E-11	8.84E-12	6.64E-12	5.16E-12	4.12E-12	3.37E-12
NW	4.46E-10	2.19E-10	1.37E-10	6.93E-11	4.20E-11	2.81E-11	2.02E-11	1.51E-11	1.18E-11	9.40E-12	7.67E-12
NNW	6.47E-10	3.17E-10	1.99E-10	1.01E-10	6.09E-11	4.08E-11	2.92E-11	2.20E-11	1.71E-11	1.36E-11	1.11E-11
N	5.70E-10	2.79E-10	1.75E-10	8.85E-11	5.36E-11	3.59E-11	2.57E-11	1.93E-11	1.50E-11	1.20E-11	9.80E-12
NNE	2.28E-10	1.12E-10	7.02E-11	3.55E-11	2.15E-11	1.44E-11	1.03E-11	7.75E-12	6.02E-12	4.81E-12	3.93E-12
NE	1.39E-10	6.83E-11	4.29E-11	2.17E-11	1.31E-11	8.79E-12	6.30E-12	4.73E-12	3.68E-12	2.94E-12	2.40E-12
ENE	1.05E-10	5.16E-11	3.24E-11	1.64E-11	9.91E-12	6.65E-12	4.76E-12	3.58E-12	2.78E-12	2.22E-12	1.81E-12
E	4.72E-11	2.31E-11	1.45E-11	7.34E-12	4.44E-12	2.98E-12	2.13E-12	1.60E-12	1.25E-12	9.95E-13	8.12E-13
ESE	8.81E-11	4.32E-11	2.71E-11	1.37E-11	8.29E-12	5.56E-12	3.98E-12	2.99E-12	2.32E-12	1.86E-12	1.52E-12
SE	1.65E-10	8.08E-11	5.07E-11	2.56E-11	1.55E-11	1.04E-11	7.45E-12	5.59E-12	4.35E-12	3.47E-12	2.84E-12
SSE	2.20E-10	1.08E-10	6.75E-11	3.41E-11	2.07E-11	1.39E-11	9.93E-12	7.45E-12	5.79E-12	4.63E-12	3.78E-12

CP COL 2.3(3)

Table 2.3-345 (Sheet 3 of 3)
D/Q (M-2) at Each 22.5° Sector for Each Distance (Mi) Shown at the Top

SECTOR	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	7.67E-09	2.37E-09	9.43E-10	5.15E-10	3.28E-10	1.41E-10	4.36E-11	1.73E-11	9.23E-12	5.71E-12
SSW	5.15E-09	1.59E-09	6.33E-10	3.46E-10	2.20E-10	9.44E-11	2.93E-11	1.16E-11	6.20E-12	3.84E-12
SW	3.59E-09	1.11E-09	4.41E-10	2.41E-10	1.53E-10	6.58E-11	2.04E-11	8.09E-12	4.32E-12	2.67E-12
WSW	3.04E-09	9.38E-10	3.73E-10	2.04E-10	1.30E-10	5.57E-11	1.73E-11	6.85E-12	3.66E-12	2.26E-12
W	3.52E-09	1.09E-09	4.32E-10	2.36E-10	1.50E-10	6.45E-11	2.00E-11	7.93E-12	4.23E-12	2.62E-12
WNW	5.57E-09	1.72E-09	6.85E-10	3.74E-10	2.38E-10	1.02E-10	3.17E-11	1.26E-11	6.70E-12	4.15E-12
NW	1.27E-08	3.92E-09	1.56E-09	8.53E-10	5.42E-10	2.33E-10	7.22E-11	2.86E-11	1.53E-11	9.46E-12
NNW	1.84E-08	5.69E-09	2.26E-09	1.24E-09	7.87E-10	3.38E-10	1.05E-10	4.15E-11	2.22E-11	1.37E-11
N	1.62E-08	5.01E-09	1.99E-09	1.09E-09	6.92E-10	2.97E-10	9.22E-11	3.66E-11	1.95E-11	1.21E-11
NNE	6.50E-09	2.01E-09	7.99E-10	4.37E-10	2.78E-10	1.19E-10	3.70E-11	1.47E-11	7.83E-12	4.84E-12
NE	3.97E-09	1.23E-09	4.88E-10	2.67E-10	1.70E-10	7.28E-11	2.26E-11	8.95E-12	4.78E-12	2.96E-12
ENE	3.00E-09	9.27E-10	3.69E-10	2.02E-10	1.28E-10	5.50E-11	1.71E-11	6.76E-12	3.61E-12	2.24E-12
E	1.34E-09	4.15E-10	1.65E-10	9.03E-11	5.74E-11	2.47E-11	7.65E-12	3.03E-12	1.62E-12	1.00E-12
ESE	2.51E-09	7.75E-10	3.08E-10	1.69E-10	1.07E-10	4.60E-11	1.43E-11	5.65E-12	3.02E-12	1.87E-12
SE	4.69E-09	1.45E-09	5.77E-10	3.15E-10	2.00E-10	8.61E-11	2.67E-11	1.06E-11	5.65E-12	3.50E-12
SSE	6.25E-09	1.93E-09	7.69E-10	4.20E-10	2.67E-10	1.15E-10	3.56E-11	1.41E-11	7.53E-12	4.66E-12

CP COL 2.3(3)

Table 2.3-346 (Sheet 1 of 2)
χ/Q and D/Q Values for 2.26 and 8 Day Decay Half-Lives

Type of Location	Sector	Distance (mi)	Distance (meters)	X/Q (m^3) 2.26 Day Decay Undepleted	X/Q (m^3) 8.00 Day Decay Depleted	D/Q (m^{-2})
EAB	S	0.37	600	1.70E-06	1.50E-06	2.30E-08
EAB	SSW	0.37	600	1.30E-06	1.20E-06	1.50E-08
EAB	SW	0.37	600	1.00E-06	9.40E-07	1.10E-08
EAB	WSW	0.37	600	9.80E-07	9.00E-07	9.10E-09
EAB	W	0.37	600	1.40E-06	1.30E-06	1.10E-08
EAB	WNW	0.37	600	2.20E-06	2.00E-06	1.70E-08
EAB	NW	0.37	600	4.40E-06	4.10E-06	3.80E-08
EAB	NNW	0.37	600	5.50E-06	5.10E-06	5.50E-08
EAB	N	0.37	600	3.90E-06	3.60E-06	4.90E-08
EAB	NNE	0.37	600	3.50E-06	3.20E-06	1.90E-08
EAB	NE	0.37	600	3.10E-06	2.80E-06	1.20E-08
EAB	ENE	0.37	600	2.40E-06	2.20E-06	9.00E-09
EAB	E	0.37	600	1.30E-06	1.20E-06	4.00E-09
EAB	ESE	0.37	600	1.70E-06	1.60E-06	7.50E-09
EAB	SE	0.37	600	2.20E-06	2.00E-06	1.40E-08
EAB	SSE	0.37	600	1.30E-06	1.20E-06	1.90E-08
Residence	S	1.09	1753	3.50E-07	3.10E-07	3.90E-09
Residence	SSW	0.79	1276	4.40E-07	3.90E-07	4.50E-09

CP COL 2.3(3)

Table 2.3-346 (Sheet 2 of 2)
χ/Q and D/Q Values for 2.26 and 8 Day Decay Half-Lives

Type of Location	Sector	Distance (mi)	Distance (meters)	X/Q (m^3) 2.26 Day Decay Undepleted	X/Q (m^3) 8.00 Day Decay Depleted	D/Q (m^{-2})
Residence	SW	0.79	1276	3.30E-07	3.00E-07	3.10E-09
Residence	WSW	1.16	1869	1.80E-07	1.60E-07	1.40E-09
Residence	W	1.16	1869	2.60E-07	2.20E-07	1.60E-09
Residence	WNW	2.26	3645	1.50E-07	1.30E-07	8.00E-10
Residence	NW	2.18	3515	3.30E-07	2.70E-07	1.90E-09
Residence	NNW	2.18	3515	4.10E-07	3.40E-07	2.80E-09
Residence	N	1.99	3202	3.20E-07	2.60E-07	2.90E-09
Residence	NNE	1.99	3202	2.70E-07	2.30E-07	1.20E-09
Residence	NE	2.39	3853	1.80E-07	1.50E-07	5.20E-10
Residence	ENE	2.4	3863	1.30E-07	1.10E-07	3.90E-10
Residence	E	2.76	4449	6.00E-08	4.90E-08	1.40E-10
Residence	ESE	2.43	3903	9.70E-08	8.00E-08	3.20E-10
Residence	SE	1.95	3146	1.80E-07	1.50E-07	8.70E-10
Residence	SSE	1.83	2942	1.20E-07	1.00E-07	1.30E-09
Garden	ENE	2.86	4609	1.10E-07	8.50E-08	2.90E-10
Garden	E	2.86	4609	5.70E-08	4.60E-08	1.30E-10

CP COL 2.3(1)

Table 2.3-347
Annual and Maximum 24-hr Rainfall

	Annual Average Rainfall (in)	Maximum 24-hr Rainfall (in) and date
Dallas Fort Worth	34.73	5.91 (1959)
Dallas Love Field	37.05	6.02 (1977)
Mineral Wells	31.79	6.65 (1981)
Glen Rose	34.82	8.48 (1995)

Reference: Climatography of the United States, No. 20, 1971 – 2000, U.S. Department of Commerce, National Oceanic & Atmospheric Administration, National Environmental Satellite, Data and Information Service.

CP COL 2.3(3)

Table 2.3-348 (Sheet 1 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for no decay, undepleted for each 22.5° sector at the distances (mi) shown at the top

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	5.60E-06	1.62E-06	7.94E-07	4.87E-07	2.49E-07	1.56E-07	1.10E-07	8.20E-08	6.44E-08	5.23E-08	4.35E-08
SSW	4.50E-06	1.30E-06	6.36E-07	3.89E-07	1.99E-07	1.25E-07	8.75E-08	6.56E-08	5.15E-08	4.18E-08	3.48E-08
SW	3.31E-06	9.40E-07	4.57E-07	2.79E-07	1.42E-07	8.84E-08	6.16E-08	4.61E-08	3.61E-08	2.92E-08	2.43E-08
WSW	3.20E-06	9.11E-07	4.44E-07	2.71E-07	1.38E-07	8.61E-08	6.00E-08	4.48E-08	3.51E-08	2.84E-08	2.36E-08
W	4.60E-06	1.33E-06	6.53E-07	4.00E-07	2.05E-07	1.29E-07	9.07E-08	6.82E-08	5.36E-08	4.36E-08	3.64E-08
WNW	7.44E-06	2.16E-06	1.06E-06	6.50E-07	3.35E-07	2.12E-07	1.50E-07	1.13E-07	8.92E-08	7.28E-08	6.10E-08
NW	1.52E-05	4.44E-06	2.19E-06	1.34E-06	6.97E-07	4.43E-07	3.14E-07	2.38E-07	1.89E-07	1.55E-07	1.30E-07
NNW	1.89E-05	5.52E-06	2.72E-06	1.67E-06	8.65E-07	5.50E-07	3.90E-07	2.95E-07	2.34E-07	1.92E-07	1.61E-07
N	1.32E-05	3.81E-06	1.86E-06	1.14E-06	5.91E-07	3.75E-07	2.66E-07	2.01E-07	1.59E-07	1.30E-07	1.09E-07
NNE	1.21E-05	3.51E-06	1.72E-06	1.06E-06	5.53E-07	3.55E-07	2.54E-07	1.94E-07	1.55E-07	1.28E-07	1.08E-07
NE	1.07E-05	3.13E-06	1.52E-06	9.39E-07	4.92E-07	3.17E-07	2.27E-07	1.74E-07	1.39E-07	1.15E-07	9.73E-08
ENE	8.34E-06	2.42E-06	1.17E-06	7.24E-07	3.82E-07	2.47E-07	1.78E-07	1.37E-07	1.10E-07	9.10E-08	7.72E-08
E	4.43E-06	1.29E-06	6.27E-07	3.87E-07	2.04E-07	1.32E-07	9.49E-08	7.28E-08	5.84E-08	4.83E-08	4.10E-08
ESE	5.92E-06	1.73E-06	8.41E-07	5.20E-07	2.73E-07	1.76E-07	1.27E-07	9.71E-08	7.78E-08	6.43E-08	5.45E-08
SE	7.68E-06	2.24E-06	1.09E-06	6.74E-07	3.53E-07	2.27E-07	1.63E-07	1.24E-07	9.94E-08	8.20E-08	6.93E-08
SSE	4.38E-06	1.26E-06	6.16E-07	3.77E-07	1.93E-07	1.22E-07	8.56E-08	6.44E-08	5.07E-08	4.13E-08	3.45E-08

CP COL 2.3(3)

Table 2.3-348 (Sheet 2 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for no decay, undepleted for each 22.5° sector
at the distances (mi) shown at the top

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.70E-08	1.99E-08	1.29E-08	7.01E-09	4.59E-09	3.31E-09	2.54E-09	2.03E-09	1.67E-09	1.41E-09	1.21E-09
SSW	2.96E-08	1.60E-08	1.04E-08	5.68E-09	3.73E-09	2.70E-09	2.08E-09	1.67E-09	1.38E-09	1.17E-09	1.00E-09
SW	2.06E-08	1.11E-08	7.15E-09	3.91E-09	2.57E-09	1.86E-09	1.44E-09	1.15E-09	9.54E-10	8.08E-10	6.97E-10
WSW	2.00E-08	1.07E-08	6.92E-09	3.77E-09	2.47E-09	1.79E-09	1.37E-09	1.10E-09	9.08E-10	7.67E-10	6.60E-10
W	3.10E-08	1.68E-08	1.09E-08	6.00E-09	3.95E-09	2.87E-09	2.21E-09	1.77E-09	1.47E-09	1.24E-09	1.07E-09
WNW	5.20E-08	2.85E-08	1.87E-08	1.04E-08	6.93E-09	5.06E-09	3.92E-09	3.16E-09	2.62E-09	2.23E-09	1.93E-09
NW	1.11E-07	6.16E-08	4.07E-08	2.29E-08	1.53E-08	1.12E-08	8.73E-09	7.06E-09	5.88E-09	5.01E-09	4.34E-09
NNW	1.38E-07	7.62E-08	5.04E-08	2.83E-08	1.89E-08	1.39E-08	1.08E-08	8.73E-09	7.27E-09	6.19E-09	5.36E-09
N	9.37E-08	5.19E-08	3.44E-08	1.94E-08	1.30E-08	9.59E-09	7.48E-09	6.07E-09	5.07E-09	4.32E-09	3.76E-09
NNE	9.28E-08	5.25E-08	3.53E-08	2.04E-08	1.39E-08	1.03E-08	8.12E-09	6.64E-09	5.58E-09	4.79E-09	4.18E-09
NE	8.39E-08	4.78E-08	3.23E-08	1.88E-08	1.28E-08	9.58E-09	7.56E-09	6.19E-09	5.21E-09	4.48E-09	3.92E-09
ENE	6.68E-08	3.84E-08	2.62E-08	1.54E-08	1.06E-08	7.95E-09	6.31E-09	5.19E-09	4.39E-09	3.79E-09	3.32E-09
E	3.54E-08	2.03E-08	1.38E-08	8.04E-09	5.52E-09	4.14E-09	3.27E-09	2.69E-09	2.27E-09	1.95E-09	1.71E-09
ESE	4.70E-08	2.68E-08	1.82E-08	1.06E-08	7.23E-09	5.40E-09	4.27E-09	3.50E-09	2.95E-09	2.54E-09	2.22E-09
SE	5.97E-08	3.39E-08	2.28E-08	1.32E-08	8.99E-09	6.70E-09	5.28E-09	4.32E-09	3.63E-09	3.12E-09	2.72E-09
SSE	2.94E-08	1.60E-08	1.05E-08	5.81E-09	3.85E-09	2.81E-09	2.17E-09	1.75E-09	1.46E-09	1.24E-09	1.07E-09

CP COL 2.3(3)

Table 2.3-348 (Sheet 3 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χQ (s/m^3) for no decay, undepleted for each 22.5° sector
for each segment (mi) shown at the top

DIRECTION	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	8.40E-07	2.61E-07	1.11E-07	6.48E-08	4.37E-08	2.06E-08	7.24E-09	3.34E-09	2.04E-09	1.42E-09
SSW	6.73E-07	2.08E-07	8.87E-08	5.18E-08	3.50E-08	1.65E-08	5.86E-09	2.73E-09	1.67E-09	1.17E-09
SW	4.85E-07	1.48E-07	6.25E-08	3.63E-08	2.44E-08	1.14E-08	4.03E-09	1.88E-09	1.16E-09	8.10E-10
WSW	4.71E-07	1.44E-07	6.09E-08	3.53E-08	2.37E-08	1.11E-08	3.89E-09	1.80E-09	1.10E-09	7.69E-10
W	6.91E-07	2.15E-07	9.20E-08	5.40E-08	3.65E-08	1.73E-08	6.18E-09	2.89E-09	1.78E-09	1.24E-09
WNW	1.12E-06	3.51E-07	1.52E-07	8.97E-08	6.12E-08	2.94E-08	1.07E-08	5.10E-09	3.17E-09	2.23E-09
NW	2.31E-06	7.28E-07	3.18E-07	1.90E-07	1.30E-07	6.34E-08	2.35E-08	1.13E-08	7.09E-09	5.02E-09
NNW	2.88E-06	9.05E-07	3.95E-07	2.35E-07	1.61E-07	7.84E-08	2.90E-08	1.40E-08	8.76E-09	6.20E-09
N	1.98E-06	6.18E-07	2.69E-07	1.60E-07	1.10E-07	5.34E-08	1.99E-08	9.67E-09	6.09E-09	4.33E-09
NNE	1.82E-06	5.77E-07	2.57E-07	1.56E-07	1.08E-07	5.38E-08	2.08E-08	1.04E-08	6.66E-09	4.80E-09
NE	1.62E-06	5.14E-07	2.30E-07	1.40E-07	9.76E-08	4.89E-08	1.91E-08	9.63E-09	6.21E-09	4.49E-09
ENE	1.25E-06	3.98E-07	1.80E-07	1.10E-07	7.74E-08	3.93E-08	1.56E-08	8.00E-09	5.20E-09	3.79E-09
E	6.68E-07	2.13E-07	9.59E-08	5.87E-08	4.11E-08	2.07E-08	8.19E-09	4.16E-09	2.70E-09	1.96E-09
ESE	8.95E-07	2.85E-07	1.28E-07	7.82E-08	5.46E-08	2.74E-08	1.08E-08	5.43E-09	3.51E-09	2.54E-09
SE	1.16E-06	3.68E-07	1.65E-07	9.99E-08	6.95E-08	3.47E-08	1.35E-08	6.74E-09	4.33E-09	3.13E-09
SSE	6.53E-07	2.02E-07	8.68E-08	5.10E-08	3.46E-08	1.65E-08	5.97E-09	2.83E-09	1.76E-09	1.24E-09

CP COL 2.3(3)

Table 2.3-348 (Sheet 4 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for a 2.26 day decay, undepleted for each 22.5° sector at the distances (mi) shown at the top

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	5.59E-06	1.61E-06	7.93E-07	4.86E-07	2.48E-07	1.56E-07	1.09E-07	8.15E-08	6.39E-08	5.18E-08	4.31E-08
SSW	4.50E-06	1.29E-06	6.34E-07	3.88E-07	1.98E-07	1.24E-07	8.69E-08	6.51E-08	5.11E-08	4.14E-08	3.45E-08
SW	3.31E-06	9.38E-07	4.56E-07	2.78E-07	1.41E-07	8.79E-08	6.12E-08	4.56E-08	3.57E-08	2.89E-08	2.39E-08
WSW	3.20E-06	9.09E-07	4.43E-07	2.70E-07	1.37E-07	8.55E-08	5.95E-08	4.44E-08	3.47E-08	2.80E-08	2.33E-08
W	4.59E-06	1.33E-06	6.52E-07	3.99E-07	2.04E-07	1.28E-07	9.00E-08	6.75E-08	5.30E-08	4.31E-08	3.59E-08
WNW	7.43E-06	2.15E-06	1.06E-06	6.48E-07	3.34E-07	2.11E-07	1.49E-07	1.12E-07	8.82E-08	7.19E-08	6.01E-08
NW	1.51E-05	4.43E-06	2.18E-06	1.34E-06	6.94E-07	4.41E-07	3.12E-07	2.36E-07	1.87E-07	1.53E-07	1.28E-07
NNW	1.89E-05	5.51E-06	2.71E-06	1.66E-06	8.59E-07	5.45E-07	3.85E-07	2.91E-07	2.30E-07	1.88E-07	1.58E-07
N	1.32E-05	3.81E-06	1.86E-06	1.14E-06	5.89E-07	3.73E-07	2.64E-07	2.00E-07	1.58E-07	1.29E-07	1.08E-07
NNE	1.20E-05	3.50E-06	1.71E-06	1.05E-06	5.47E-07	3.50E-07	2.50E-07	1.90E-07	1.51E-07	1.24E-07	1.05E-07
NE	1.07E-05	3.11E-06	1.51E-06	9.32E-07	4.87E-07	3.12E-07	2.23E-07	1.70E-07	1.36E-07	1.12E-07	9.40E-08
ENE	8.33E-06	2.42E-06	1.17E-06	7.21E-07	3.79E-07	2.45E-07	1.76E-07	1.35E-07	1.08E-07	8.96E-08	7.59E-08
E	4.42E-06	1.29E-06	6.24E-07	3.85E-07	2.02E-07	1.31E-07	9.37E-08	7.18E-08	5.75E-08	4.74E-08	4.01E-08
ESE	5.92E-06	1.72E-06	8.38E-07	5.17E-07	2.72E-07	1.75E-07	1.25E-07	9.59E-08	7.67E-08	6.32E-08	5.34E-08
SE	7.67E-06	2.24E-06	1.09E-06	6.72E-07	3.51E-07	2.26E-07	1.61E-07	1.23E-07	9.82E-08	8.09E-08	6.83E-08
SSE	4.38E-06	1.26E-06	6.15E-07	3.76E-07	1.93E-07	1.21E-07	8.51E-08	6.39E-08	5.03E-08	4.09E-08	3.41E-08

CP COL 2.3(3)

Table 2.3-348 (Sheet 5 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for a 2.26 day decay, undepleted for each 22.5° sector at the distances (mi) shown at the top

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	3.66E-08	1.96E-08	1.26E-08	6.79E-09	4.39E-09	3.13E-09	2.38E-09	1.88E-09	1.53E-09	1.28E-09	1.09E-09
SSW	2.93E-08	1.57E-08	1.01E-08	5.47E-09	3.55E-09	2.54E-09	1.93E-09	1.53E-09	1.25E-09	1.04E-09	8.87E-10
SW	2.03E-08	1.08E-08	6.94E-09	3.73E-09	2.42E-09	1.73E-09	1.31E-09	1.04E-09	8.47E-10	7.07E-10	6.01E-10
WSW	1.97E-08	1.05E-08	6.69E-09	3.58E-09	2.31E-09	1.64E-09	1.24E-09	9.80E-10	7.96E-10	6.63E-10	5.61E-10
W	3.05E-08	1.64E-08	1.06E-08	5.73E-09	3.71E-09	2.65E-09	2.01E-09	1.59E-09	1.29E-09	1.08E-09	9.15E-10
WNW	5.12E-08	2.79E-08	1.82E-08	9.95E-09	6.51E-09	4.67E-09	3.56E-09	2.83E-09	2.31E-09	1.93E-09	1.64E-09
NW	1.10E-07	6.04E-08	3.97E-08	2.21E-08	1.46E-08	1.05E-08	8.09E-09	6.46E-09	5.31E-09	4.46E-09	3.82E-09
NNW	1.35E-07	7.36E-08	4.81E-08	2.65E-08	1.73E-08	1.24E-08	9.48E-09	7.53E-09	6.16E-09	5.15E-09	4.39E-09
N	9.25E-08	5.09E-08	3.35E-08	1.86E-08	1.24E-08	8.97E-09	6.90E-09	5.52E-09	4.55E-09	3.83E-09	3.28E-09
NNE	8.97E-08	5.00E-08	3.31E-08	1.85E-08	1.23E-08	8.87E-09	6.80E-09	5.42E-09	4.44E-09	3.72E-09	3.18E-09
NE	8.07E-08	4.52E-08	3.00E-08	1.69E-08	1.12E-08	8.09E-09	6.20E-09	4.94E-09	4.05E-09	3.39E-09	2.89E-09
ENE	6.55E-08	3.73E-08	2.52E-08	1.45E-08	9.79E-09	7.22E-09	5.62E-09	4.54E-09	3.76E-09	3.19E-09	2.74E-09
E	3.46E-08	1.96E-08	1.31E-08	7.49E-09	5.03E-09	3.68E-09	2.84E-09	2.28E-09	1.88E-09	1.59E-09	1.36E-09
ESE	4.60E-08	2.60E-08	1.74E-08	9.89E-09	6.63E-09	4.85E-09	3.75E-09	3.01E-09	2.48E-09	2.09E-09	1.79E-09
SE	5.87E-08	3.30E-08	2.21E-08	1.25E-08	8.41E-09	6.17E-09	4.78E-09	3.85E-09	3.18E-09	2.69E-09	2.31E-09
SSE	2.90E-08	1.57E-08	1.02E-08	5.59E-09	3.66E-09	2.64E-09	2.02E-09	1.60E-09	1.32E-09	1.10E-09	9.42E-10

CP COL 2.3(3)

Table 2.3-348 (Sheet 6 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for a 2.26 day decay, undepleted at each 22.5° sector for each segment (mi) shown at the top

DIRECTION	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	8.39E-07	2.60E-07	1.10E-07	6.43E-08	4.33E-08	2.03E-08	7.01E-09	3.17E-09	1.89E-09	1.28E-09
SSW	6.72E-07	2.08E-07	8.81E-08	5.14E-08	3.46E-08	1.62E-08	5.65E-09	2.57E-09	1.54E-09	1.05E-09
SW	4.84E-07	1.48E-07	6.21E-08	3.59E-08	2.40E-08	1.12E-08	3.86E-09	1.75E-09	1.04E-09	7.09E-10
WSW	4.70E-07	1.44E-07	6.04E-08	3.49E-08	2.34E-08	1.08E-08	3.71E-09	1.66E-09	9.85E-10	6.65E-10
W	6.90E-07	2.14E-07	9.12E-08	5.34E-08	3.60E-08	1.69E-08	5.91E-09	2.68E-09	1.60E-09	1.08E-09
WNW	1.12E-06	3.49E-07	1.51E-07	8.88E-08	6.03E-08	2.87E-08	1.02E-08	4.72E-09	2.84E-09	1.94E-09
NW	2.31E-06	7.26E-07	3.16E-07	1.88E-07	1.29E-07	6.22E-08	2.27E-08	1.06E-08	6.49E-09	4.48E-09
NNW	2.87E-06	8.99E-07	3.90E-07	2.32E-07	1.58E-07	7.58E-08	2.72E-08	1.26E-08	7.56E-09	5.17E-09
N	1.97E-06	6.15E-07	2.67E-07	1.59E-07	1.09E-07	5.24E-08	1.91E-08	9.04E-09	5.54E-09	3.84E-09
NNE	1.81E-06	5.71E-07	2.53E-07	1.52E-07	1.05E-07	5.13E-08	1.90E-08	8.94E-09	5.44E-09	3.73E-09
NE	1.61E-06	5.08E-07	2.26E-07	1.36E-07	9.43E-08	4.64E-08	1.73E-08	8.16E-09	4.96E-09	3.40E-09
ENE	1.25E-06	3.96E-07	1.78E-07	1.09E-07	7.61E-08	3.82E-08	1.48E-08	7.26E-09	4.55E-09	3.19E-09
E	6.65E-07	2.11E-07	9.48E-08	5.77E-08	4.02E-08	2.00E-08	7.65E-09	3.70E-09	2.29E-09	1.59E-09
ESE	8.93E-07	2.83E-07	1.27E-07	7.70E-08	5.36E-08	2.66E-08	1.01E-08	4.88E-09	3.02E-09	2.09E-09
SE	1.16E-06	3.67E-07	1.63E-07	9.87E-08	6.85E-08	3.39E-08	1.28E-08	6.21E-09	3.86E-09	2.69E-09
SSE	6.52E-07	2.02E-07	8.63E-08	5.06E-08	3.42E-08	1.62E-08	5.76E-09	2.66E-09	1.61E-09	1.11E-09

CP COL 2.3(3)

Table 2.3-348 (Sheet 7 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for an 8.00 day decay, depleted for each 22.5° sector at the distances (mi) shown at the top

SECTOR	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	5.30E-06	1.48E-06	7.07E-07	4.26E-07	2.11E-07	1.29E-07	8.85E-08	6.51E-08	5.01E-08	4.00E-08	3.28E-08
SSW	4.26E-06	1.18E-06	5.66E-07	3.40E-07	1.69E-07	1.03E-07	7.07E-08	5.20E-08	4.01E-08	3.20E-08	2.63E-08
SW	3.14E-06	8.58E-07	4.07E-07	2.44E-07	1.20E-07	7.31E-08	4.98E-08	3.65E-08	2.81E-08	2.23E-08	1.83E-08
WSW	3.03E-06	8.31E-07	3.96E-07	2.37E-07	1.17E-07	7.11E-08	4.85E-08	3.55E-08	2.73E-08	2.17E-08	1.78E-08
W	4.35E-06	1.21E-06	5.82E-07	3.50E-07	1.74E-07	1.07E-07	7.33E-08	5.40E-08	4.17E-08	3.34E-08	2.74E-08
WNW	7.04E-06	1.97E-06	9.44E-07	5.69E-07	2.84E-07	1.75E-07	1.21E-07	8.95E-08	6.94E-08	5.57E-08	4.59E-08
NW	1.43E-05	4.05E-06	1.95E-06	1.18E-06	5.91E-07	3.67E-07	2.54E-07	1.89E-07	1.47E-07	1.18E-07	9.79E-08
NNW	1.79E-05	5.04E-06	2.42E-06	1.46E-06	7.34E-07	4.55E-07	3.15E-07	2.34E-07	1.82E-07	1.46E-07	1.21E-07
N	1.25E-05	3.48E-06	1.66E-06	1.00E-06	5.01E-07	3.10E-07	2.15E-07	1.59E-07	1.24E-07	9.98E-08	8.25E-08
NNE	1.14E-05	3.21E-06	1.53E-06	9.23E-07	4.68E-07	2.93E-07	2.05E-07	1.53E-07	1.20E-07	9.72E-08	8.08E-08
NE	1.01E-05	2.85E-06	1.36E-06	8.20E-07	4.17E-07	2.61E-07	1.83E-07	1.37E-07	1.08E-07	8.75E-08	7.28E-08
ENE	7.89E-06	2.21E-06	1.04E-06	6.33E-07	3.24E-07	2.04E-07	1.44E-07	1.08E-07	8.54E-08	6.96E-08	5.81E-08
E	4.19E-06	1.18E-06	5.58E-07	3.38E-07	1.73E-07	1.09E-07	7.66E-08	5.76E-08	4.54E-08	3.69E-08	3.08E-08
ESE	5.60E-06	1.58E-06	7.49E-07	4.54E-07	2.32E-07	1.46E-07	1.02E-07	7.69E-08	6.05E-08	4.91E-08	4.10E-08
SE	7.27E-06	2.05E-06	9.73E-07	5.90E-07	2.99E-07	1.88E-07	1.31E-07	9.85E-08	7.73E-08	6.27E-08	5.22E-08
SSE	4.14E-06	1.15E-06	5.49E-07	3.30E-07	1.64E-07	1.01E-07	6.92E-08	5.10E-08	3.95E-08	3.16E-08	2.60E-08

CP COL 2.3(3)

Table 2.3-348 (Sheet 8 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for an 8.00 day decay, depleted for each 22.5° sector at the distances (mi) shown at the top

SECTOR	5	7.5	10	15	20	25	30	35	40	45	50
S	2.75E-08	1.40E-08	8.59E-09	4.32E-09	2.65E-09	1.80E-09	1.31E-09	1.00E-09	7.91E-10	6.41E-10	5.30E-10
SSW	2.20E-08	1.12E-08	6.92E-09	3.49E-09	2.15E-09	1.47E-09	1.07E-09	8.21E-10	6.49E-10	5.27E-10	4.36E-10
SW	1.53E-08	7.75E-09	4.76E-09	2.40E-09	1.48E-09	1.01E-09	7.38E-10	5.65E-10	4.47E-10	3.63E-10	3.01E-10
WSW	1.49E-08	7.51E-09	4.60E-09	2.31E-09	1.42E-09	9.65E-10	7.03E-10	5.37E-10	4.24E-10	3.43E-10	2.84E-10
W	2.30E-08	1.18E-08	7.27E-09	3.68E-09	2.27E-09	1.55E-09	1.13E-09	8.67E-10	6.86E-10	5.56E-10	4.61E-10
WNW	3.86E-08	2.00E-08	1.25E-08	6.40E-09	3.98E-09	2.74E-09	2.01E-09	1.55E-09	1.23E-09	9.99E-10	8.29E-10
NW	8.26E-08	4.32E-08	2.72E-08	1.41E-08	8.82E-09	6.11E-09	4.51E-09	3.48E-09	2.77E-09	2.26E-09	1.88E-09
NNW	1.02E-07	5.32E-08	3.34E-08	1.73E-08	1.08E-08	7.43E-09	5.47E-09	4.21E-09	3.34E-09	2.72E-09	2.26E-09
N	6.96E-08	3.64E-08	2.29E-08	1.19E-08	7.50E-09	5.21E-09	3.86E-09	2.98E-09	2.38E-09	1.95E-09	1.63E-09
NNE	6.85E-08	3.65E-08	2.33E-08	1.23E-08	7.81E-09	5.46E-09	4.06E-09	3.15E-09	2.52E-09	2.06E-09	1.72E-09
NE	6.18E-08	3.32E-08	2.12E-08	1.13E-08	7.19E-09	5.04E-09	3.75E-09	2.92E-09	2.34E-09	1.92E-09	1.60E-09
ENE	4.95E-08	2.69E-08	1.74E-08	9.39E-09	6.05E-09	4.28E-09	3.22E-09	2.52E-09	2.04E-09	1.68E-09	1.42E-09
E	2.62E-08	1.42E-08	9.12E-09	4.90E-09	3.14E-09	2.21E-09	1.66E-09	1.30E-09	1.04E-09	8.59E-10	7.21E-10
ESE	3.48E-08	1.87E-08	1.20E-08	6.44E-09	4.12E-09	2.90E-09	2.17E-09	1.69E-09	1.36E-09	1.12E-09	9.39E-10
SE	4.43E-08	2.37E-08	1.52E-08	8.08E-09	5.15E-09	3.62E-09	2.71E-09	2.11E-09	1.70E-09	1.40E-09	1.17E-09
SSE	2.18E-08	1.12E-08	6.99E-09	3.57E-09	2.22E-09	1.53E-09	1.12E-09	8.63E-10	6.86E-10	5.59E-10	4.64E-10

CP COL 2.3(3)

Table 2.3-348 (Sheet 9 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average χ/Q (s/m^3) for an 8.00 day decay, depleted at each 22.5° sector for each segment (mi) shown at the top

DIRECTION	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	7.53E-07	2.23E-07	9.00E-08	5.06E-08	3.30E-08	1.46E-08	4.53E-09	1.83E-09	1.01E-09	6.44E-10
SSW	6.03E-07	1.78E-07	7.19E-08	4.04E-08	2.64E-08	1.17E-08	3.66E-09	1.49E-09	8.28E-10	5.30E-10
SW	4.35E-07	1.27E-07	5.07E-08	2.83E-08	1.84E-08	8.10E-09	2.51E-09	1.03E-09	5.70E-10	3.65E-10
WSW	4.22E-07	1.23E-07	4.93E-08	2.75E-08	1.79E-08	7.85E-09	2.42E-09	9.81E-10	5.41E-10	3.45E-10
W	6.19E-07	1.83E-07	7.45E-08	4.20E-08	2.75E-08	1.23E-08	3.85E-09	1.58E-09	8.74E-10	5.59E-10
WNW	1.01E-06	2.99E-07	1.23E-07	6.99E-08	4.61E-08	2.08E-08	6.67E-09	2.78E-09	1.56E-09	1.00E-09
NW	2.07E-06	6.21E-07	2.58E-07	1.48E-07	9.83E-08	4.48E-08	1.47E-08	6.19E-09	3.50E-09	2.27E-09
NNW	2.58E-06	7.71E-07	3.20E-07	1.83E-07	1.21E-07	5.52E-08	1.80E-08	7.53E-09	4.24E-09	2.74E-09
N	1.77E-06	5.27E-07	2.18E-07	1.25E-07	8.28E-08	3.78E-08	1.24E-08	5.28E-09	3.00E-09	1.96E-09
NNE	1.63E-06	4.91E-07	2.08E-07	1.21E-07	8.11E-08	3.77E-08	1.28E-08	5.52E-09	3.17E-09	2.07E-09
NE	1.45E-06	4.37E-07	1.86E-07	1.09E-07	7.31E-08	3.42E-08	1.17E-08	5.10E-09	2.93E-09	1.92E-09
ENE	1.12E-06	3.39E-07	1.46E-07	8.59E-08	5.83E-08	2.77E-08	9.68E-09	4.33E-09	2.54E-09	1.69E-09
E	5.98E-07	1.81E-07	7.76E-08	4.56E-08	3.09E-08	1.46E-08	5.06E-09	2.24E-09	1.30E-09	8.62E-10
ESE	8.02E-07	2.43E-07	1.04E-07	6.08E-08	4.11E-08	1.93E-08	6.65E-09	2.93E-09	1.70E-09	1.12E-09
SE	1.04E-06	3.14E-07	1.33E-07	7.78E-08	5.24E-08	2.45E-08	8.36E-09	3.66E-09	2.12E-09	1.40E-09
SSE	5.85E-07	1.73E-07	7.04E-08	3.98E-08	2.61E-08	1.17E-08	3.73E-09	1.55E-09	8.69E-10	5.61E-10

CP COL 2.3(3)

Table 2.3-348 (Sheet 10 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average D/Q (m^{-2}) at each 22.5° sector
for each distance (mi) shown at the top

DIRECTION	0.25	0.5	0.75	1	1.5	2	2.5	3	3.5	4	4.5
S	4.26E-08	1.44E-08	7.40E-09	4.54E-09	2.26E-09	1.37E-09	9.28E-10	6.73E-10	5.12E-10	4.03E-10	3.26E-10
SSW	2.87E-08	9.70E-09	4.98E-09	3.06E-09	1.52E-09	9.25E-10	6.25E-10	4.53E-10	3.44E-10	2.71E-10	2.20E-10
SW	1.98E-08	6.71E-09	3.44E-09	2.11E-09	1.05E-09	6.39E-10	4.32E-10	3.13E-10	2.38E-10	1.88E-10	1.52E-10
WSW	1.68E-08	5.69E-09	2.92E-09	1.80E-09	8.95E-10	5.43E-10	3.67E-10	2.66E-10	2.02E-10	1.59E-10	1.29E-10
W	1.96E-08	6.62E-09	3.40E-09	2.09E-09	1.04E-09	6.31E-10	4.26E-10	3.09E-10	2.35E-10	1.85E-10	1.50E-10
WNW	3.08E-08	1.04E-08	5.35E-09	3.29E-09	1.64E-09	9.93E-10	6.72E-10	4.87E-10	3.70E-10	2.92E-10	2.36E-10
NW	7.05E-08	2.39E-08	1.23E-08	7.52E-09	3.75E-09	2.27E-09	1.54E-09	1.11E-09	8.47E-10	6.67E-10	5.40E-10
NNW	1.02E-07	3.46E-08	1.77E-08	1.09E-08	5.43E-09	3.29E-09	2.23E-09	1.61E-09	1.23E-09	9.67E-10	7.83E-10
N	8.95E-08	3.03E-08	1.55E-08	9.54E-09	4.76E-09	2.89E-09	1.95E-09	1.41E-09	1.08E-09	8.47E-10	6.86E-10
NNE	3.61E-08	1.22E-08	6.27E-09	3.85E-09	1.92E-09	1.16E-09	7.87E-10	5.70E-10	4.34E-10	3.42E-10	2.77E-10
NE	2.21E-08	7.46E-09	3.83E-09	2.35E-09	1.17E-09	7.12E-10	4.81E-10	3.49E-10	2.65E-10	2.09E-10	1.69E-10
ENE	1.66E-08	5.60E-09	2.88E-09	1.77E-09	8.81E-10	5.34E-10	3.61E-10	2.62E-10	1.99E-10	1.57E-10	1.27E-10
E	7.39E-09	2.50E-09	1.28E-09	7.88E-10	3.93E-10	2.38E-10	1.61E-10	1.17E-10	8.87E-11	6.99E-11	5.66E-11
ESE	1.39E-08	4.70E-09	2.42E-09	1.48E-09	7.39E-10	4.48E-10	3.03E-10	2.20E-10	1.67E-10	1.32E-10	1.07E-10
SE	2.60E-08	8.78E-09	4.51E-09	2.77E-09	1.38E-09	8.37E-10	5.66E-10	4.10E-10	3.12E-10	2.46E-10	1.99E-10
SSE	3.47E-08	1.17E-08	6.02E-09	3.70E-09	1.84E-09	1.12E-09	7.56E-10	5.48E-10	4.16E-10	3.28E-10	2.66E-10

CP COL 2.3(3)

Table 2.3-348 (Sheet 11 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average D/Q (m^{-2}) at each 22.5° sector
for each distance (mi) shown at the top

DIRECTION	5	7.5	10	15	20	25	30	35	40	45	50
S	2.70E-10	1.32E-10	8.30E-11	4.19E-11	2.54E-11	1.70E-11	1.22E-11	9.16E-12	7.12E-12	5.69E-12	4.64E-12
SSW	1.82E-10	8.91E-11	5.59E-11	2.82E-11	1.71E-11	1.15E-11	8.21E-12	6.17E-12	4.80E-12	3.83E-12	3.13E-12
SW	1.26E-10	6.16E-11	3.86E-11	1.95E-11	1.18E-11	7.92E-12	5.68E-12	4.26E-12	3.32E-12	2.65E-12	2.16E-12
WSW	1.07E-10	5.23E-11	3.28E-11	1.66E-11	1.00E-11	6.73E-12	4.82E-12	3.62E-12	2.81E-12	2.25E-12	1.84E-12
W	1.24E-10	6.07E-11	3.81E-11	1.93E-11	1.17E-11	7.82E-12	5.60E-12	4.21E-12	3.27E-12	2.61E-12	2.13E-12
WNW	1.95E-10	9.57E-11	6.00E-11	3.03E-11	1.84E-11	1.23E-11	8.82E-12	6.62E-12	5.15E-12	4.11E-12	3.36E-12
NW	4.47E-10	2.19E-10	1.37E-10	6.95E-11	4.20E-11	2.82E-11	2.02E-11	1.52E-11	1.18E-11	9.42E-12	7.69E-12
NNW	6.48E-10	3.17E-10	1.99E-10	1.01E-10	6.09E-11	4.08E-11	2.93E-11	2.20E-11	1.71E-11	1.37E-11	1.11E-11
N	5.67E-10	2.78E-10	1.74E-10	8.81E-11	5.33E-11	3.58E-11	2.56E-11	1.92E-11	1.50E-11	1.20E-11	9.76E-12
NNE	2.29E-10	1.12E-10	7.04E-11	3.56E-11	2.15E-11	1.44E-11	1.03E-11	7.76E-12	6.04E-12	4.82E-12	3.94E-12
NE	1.40E-10	6.85E-11	4.30E-11	2.17E-11	1.32E-11	8.82E-12	6.32E-12	4.75E-12	3.69E-12	2.95E-12	2.41E-12
ENE	1.05E-10	5.15E-11	3.23E-11	1.63E-11	9.88E-12	6.62E-12	4.74E-12	3.56E-12	2.77E-12	2.21E-12	1.81E-12
E	4.68E-11	2.29E-11	1.44E-11	7.27E-12	4.40E-12	2.95E-12	2.12E-12	1.59E-12	1.24E-12	9.86E-13	8.05E-13
ESE	8.81E-11	4.32E-11	2.71E-11	1.37E-11	8.29E-12	5.56E-12	3.98E-12	2.99E-12	2.33E-12	1.86E-12	1.52E-12
SE	1.64E-10	8.06E-11	5.06E-11	2.56E-11	1.55E-11	1.04E-11	7.43E-12	5.58E-12	4.34E-12	3.47E-12	2.83E-12
SSE	2.20E-10	1.08E-10	6.75E-11	3.41E-11	2.07E-11	1.39E-11	9.93E-12	7.45E-12	5.80E-12	4.63E-12	3.78E-12

CP COL 2.3(3)

Table 2.3-348 (Sheet 12 of 15)
CPNPP Units 3 and 4 Evaporation Pond

Annual Average D/Q (m^{-2}) at each 22.5° sector
for each segment (mi) shown at the top

DIRECTION	.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
S	7.68E-09	2.37E-09	9.45E-10	5.16E-10	3.28E-10	1.41E-10	4.37E-11	1.73E-11	9.25E-12	5.72E-12
SSW	5.17E-09	1.60E-09	6.36E-10	3.48E-10	2.21E-10	9.49E-11	2.94E-11	1.17E-11	6.23E-12	3.86E-12
SW	3.58E-09	1.11E-09	4.40E-10	2.40E-10	1.53E-10	6.56E-11	2.04E-11	8.06E-12	4.31E-12	2.67E-12
WSW	3.04E-09	9.38E-10	3.73E-10	2.04E-10	1.30E-10	5.57E-11	1.73E-11	6.85E-12	3.66E-12	2.26E-12
W	3.53E-09	1.09E-09	4.34E-10	2.37E-10	1.51E-10	6.47E-11	2.01E-11	7.96E-12	4.25E-12	2.63E-12
WNW	5.56E-09	1.72E-09	6.83E-10	3.73E-10	2.37E-10	1.02E-10	3.16E-11	1.25E-11	6.69E-12	4.14E-12
NW	1.27E-08	3.93E-09	1.56E-09	8.55E-10	5.43E-10	2.33E-10	7.24E-11	2.87E-11	1.53E-11	9.48E-12
NNW	1.84E-08	5.70E-09	2.27E-09	1.24E-09	7.87E-10	3.38E-10	1.05E-10	4.16E-11	2.22E-11	1.37E-11
N	1.62E-08	4.99E-09	1.99E-09	1.09E-09	6.89E-10	2.96E-10	9.18E-11	3.64E-11	1.94E-11	1.20E-11
NNE	6.52E-09	2.01E-09	8.01E-10	4.38E-10	2.78E-10	1.20E-10	3.71E-11	1.47E-11	7.84E-12	4.85E-12
NE	3.98E-09	1.23E-09	4.90E-10	2.68E-10	1.70E-10	7.30E-11	2.27E-11	8.98E-12	4.79E-12	2.97E-12
ENE	2.99E-09	9.24E-10	3.68E-10	2.01E-10	1.28E-10	5.48E-11	1.70E-11	6.74E-12	3.60E-12	2.23E-12
E	1.33E-09	4.12E-10	1.64E-10	8.95E-11	5.69E-11	2.44E-11	7.58E-12	3.00E-12	1.60E-12	9.93E-13
ESE	2.51E-09	7.75E-10	3.09E-10	1.69E-10	1.07E-10	4.60E-11	1.43E-11	5.66E-12	3.02E-12	1.87E-12
SE	4.68E-09	1.45E-09	5.76E-10	3.15E-10	2.00E-10	8.59E-11	2.66E-11	1.06E-11	5.64E-12	3.49E-12
SSE	6.26E-09	1.93E-09	7.69E-10	4.20E-10	2.67E-10	1.15E-10	3.56E-11	1.41E-11	7.53E-12	4.66E-12

CP COL 2.3(3)

Table 2.3-348 (Sheet 13 of 15) CPNPP Units 3 and 4 Evaporation Pond

χ/Q and D/Q values at each receptor location

RELEASE	DIRECTION	DIST. (MI)	χ/Q (SEC/M^3) NO DECAY, UNDEPLETED	χ/Q (SEC/M^3) 2.26 DAY DECAY, UNDEPLETED	χ/Q (SEC/M^3) 8 DAY DECAY, DEPLETED	D/Q (M^{-2})
EAB	S	0.08	5.20E-05	5.10E-05	5.00E-05	2.30E-07
EAB	SSW	0.08	4.10E-05	4.10E-05	4.10E-05	1.60E-07
EAB	SW	0.09	2.20E-05	2.20E-05	2.20E-05	8.50E-08
EAB	WSW	0.1	1.90E-05	1.90E-05	1.80E-05	6.60E-08
EAB	W	0.13	1.60E-05	1.60E-05	1.60E-05	5.30E-08
EAB	WNW	0.18	1.30E-05	1.30E-05	1.30E-05	4.90E-08
EAB	NW	0.3	1.10E-05	1.10E-05	1.00E-05	5.30E-08
EAB	NNW	0.51	5.30E-06	5.30E-06	4.80E-06	3.30E-08
EAB	N	0.75	1.90E-06	1.90E-06	1.70E-06	1.60E-08
EAB	NNE	0.89	1.30E-06	1.30E-06	1.10E-06	4.70E-09
EAB	NE	1.05	8.60E-07	8.50E-07	7.50E-07	2.10E-09
EAB	ENE	0.88	9.00E-07	8.90E-07	7.90E-07	2.20E-09
EAB	E	0.54	1.10E-06	1.10E-06	1.00E-06	2.20E-09
EAB	ESE	0.27	5.20E-06	5.20E-06	4.90E-06	1.20E-08
EAB	SE	0.16	1.70E-05	1.70E-05	1.70E-05	5.10E-08
EAB	SSE	0.11	1.80E-05	1.80E-05	1.80E-05	1.10E-07
Residence	S	0.67	9.70E-07	9.70E-07	8.70E-07	9.00E-09
Residence	SSW	0.31	3.10E-06	3.10E-06	2.90E-06	2.10E-08
Residence	SW	0.31	2.30E-06	2.30E-06	2.10E-06	1.50E-08

CP COL 2.3(3)

Table 2.3-348 (Sheet 14 of 15)
CPNPP Units 3 and 4 Evaporation Pond

χ/Q and D/Q values at each receptor location

RELEASE	DIRECTION	DIST. (MI)	χ/Q (SEC/M^3) NO DECAY, UNDEPLETED	χ/Q (SEC/M^3) 2.26 DAY DECAY, UNDEPLETED	χ/Q (SEC/M^3) 8 DAY DECAY, DEPLETED	D/Q (M^{-2})
Residence	WSW	0.31	2.20E-06	2.20E-06	2.10E-06	1.20E-08
Residence	W	0.83	5.50E-07	5.50E-07	4.90E-07	2.90E-09
Residence	WNW	0.83	9.00E-07	9.00E-07	8.00E-07	4.60E-09
Residence	NW	2.16	3.90E-07	3.90E-07	3.20E-07	2.00E-09
Residence	NNW	2.31	4.40E-07	4.30E-07	3.60E-07	2.60E-09
Residence	N	2.44	2.80E-07	2.70E-07	2.20E-07	2.00E-09
Residence	NNE	2.44	2.60E-07	2.60E-07	2.10E-07	8.20E-10
Residence	NE	2.87	1.90E-07	1.80E-07	1.50E-07	3.80E-10
Residence	ENE	2.87	1.50E-07	1.40E-07	1.20E-07	2.80E-10
Residence	E	2.91	7.60E-08	7.50E-08	6.00E-08	1.20E-10
Residence	ESE	1.86	2.00E-07	2.00E-07	1.60E-07	5.10E-10
Residence	SE	1.59	3.20E-07	3.20E-07	2.70E-07	1.20E-09
Residence	SSE	0.67	7.60E-07	7.60E-07	6.80E-07	7.30E-09
Garden	ENE	3.27	1.20E-07	1.20E-07	9.50E-08	2.20E-10
Garden	E	3.27	6.40E-08	6.30E-08	5.00E-08	1.00E-10
SCR	NNW	0.41	7.90E-06	7.90E-06	7.30E-06	4.80E-08
SCR	N	0.41	5.50E-06	5.50E-06	5.00E-06	4.20E-08
SCR	NNE	0.41	5.10E-06	5.00E-06	4.60E-06	1.70E-08
SCR	NE	0.41	4.50E-06	4.50E-06	4.10E-06	1.00E-08

CP COL 2.3(3)

Table 2.3-348 (Sheet 15 of 15)
CPNPP Units 3 and 4 Evaporation Pond

χ/Q and D/Q values at each receptor location

RELEASE	DIRECTION	DIST. (MI)	χ/Q (SEC/M^3) NO DECAY, UNDEPLETED	χ/Q (SEC/M^3) 2.26 DAY DECAY, UNDEPLETED	χ/Q (SEC/M^3) 8 DAY DECAY, DEPLETED	D/Q (M^{-2})
SCR	ENE	0.41	3.50E-06	3.50E-06	3.20E-06	7.80E-09
SCR	E	0.41	1.90E-06	1.90E-06	1.70E-06	3.50E-09
SCR	ESE	0.41	2.50E-06	2.50E-06	2.30E-06	6.50E-09

Note:

SCR refers to Squaw Creek Reservoir.

CP COL 2.3(3)

Table 2.3-349
CPNPP Units 3 and 4 Evaporation Pond
Distances, in Meters, from the Center Point of the Evaporation Pond to the Nearest Boundary of the EAB in each Sector

Sector	EAB Distance
S	122
SSW	122
SW	145
WSW	156
W	203
WNW	295
NW	486
NNW	822
N	1205
NNE	1436
NE	1697
ENE	1413
E	874
ESE	434
SE	255
SSE	185

CP COL 2.3(3)

Table 2.3-350
CPNPP Units 3 and 4 Evaporation Pond
Distances, in Meters, from the Center Point of the Evaporation Pond to the Nearest Receptor in each Sector

Sector	Nearest Residence	Nearest Garden	SCR
S	1073		
SSW	493		
SW	493		
WSW	493		
W	1328		
WNW	1328		
NW	3472		
NNW	3723		655
N	3927		655
NNE	3927		655
NE	4621		655
ENE	4621	5265	655
E	4680	5265	655
ESE	2995		655
SE	2565		
SSE	1073		

Note:

SCR refers to Squaw Creek Reservoir.

Table 2.3-351
Monthly Average Humidity Comparison (6/12/2008 - 9/23/2008)

Month	CPNPP	DFW	MWL
June	52.9	52.1	54.7
July	50.4	46.7	51.7
August	59.6	56.3	64.0
September	64.7	59.3	67.6
Average	56.7	53.3	59.4
Std. Dev from CPNPP		7.80	8.07

DFW - Dallas Fort Worth

MWL - Mineral Wells Airport

2.4 HYDROLOGIC ENGINEERING

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

CP SUP 2.4(1) Add the following content after the third paragraph of DCD Section 2.4.

Section 2.4 describes the hydrological characteristics of the CPNPP Unit 3 and 4 Site. The site location and description are provided in Section 2.1 of this report in sufficient detail to support the safety analysis. This section discusses characteristics and natural phenomena that have the potential to affect the design basis for the US-APWR units. The section is divided into the following 14 subsections:

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

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2.4.1 Hydrologic Description

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.1.

This subsection describes regional and site hydrological conditions, specifically surface water and groundwater characteristics. Information provided in this subsection includes descriptions of the site and features, hydrosphere, hydrologic characteristics, drainage, dams and reservoirs, proposed water management changes, and surface water users.

2.4.1.1 Site and Facilities

Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 are located on the western end of a peninsula formed by the southern shore of Squaw Creek Reservoir (SCR) and the CPNPP Units 1 and 2 Safe Shutdown Impoundment, approximately 0.49 mi west-northwest of CPNPP Units 1 and 2 in Somervell County, Texas. The CPNPP site is located in Somervell and Hood Counties, Texas approximately 5.2 mi north-northwest of the town of Glen Rose, Texas (Figure 2.1-202).

CPNPP Units 3 and 4 are located approximately 0.49 mi west-northwest of CPNPP Units 1 and 2 as shown in Figure 2.1-201 and utilize mechanical draft cooling towers for circulating and service water system cooling. Cooling water comes from Lake Granbury located approximately 7.13 mi north-northeast of the CPNPP site.

Maximum relief in the CPNPP site area is approximately 220 ft, with elevations ranging from 640 ft to 860 ft above sea level, with slopes that are typically steep, ranging from 15 to 30 degrees or more, and generally exhibiting a stair-stepped appearance. Rock outcrops of limestone and claystone comprise approximately 40 to 60 percent of these slopes. The remaining areas, including the higher flat-topped plateau remnants, are mantled by a thin cover of soil, which at the surface generally consists of silt and sand (Reference 2.4-201). The standard plant floor elevation of the safety-related facilities is established at 822 ft above msl. The center of the nonsafety-related mechanical draft cooling towers is located about 1,800 ft to the northwest of the CPNPP Unit 3 and 4 center point at a grade elevation of 850 ft msl (Figure 2.1-201). Locations and topographic profiles showing the relationship between the CPNPP site, SCR, and Lake Granbury are illustrated on Figures 2.4.1-201 and 2.4.1-202. Grading and drainage improvements are illustrated on Figure 2.4.2-202.

Lake Granbury, the source of cooling water for the cooling tower system, is discussed in detail in Subsection 2.4.1.2. Cooling water is expected to be withdrawn by an intake structure located approximately 1.31 mi upstream from the DeCordova Bend dam. The cooling water is pumped to the CPNPP Units 3 and 4 cooling system through two pipelines, and the blowdown water from the cooling water system is discharged through two separate pipelines back to Lake Granbury about 1.14 mi downstream from the intake structure. Figure 2.4.1-203 deplicts the

location of the intake and discharge structures on Lake Granbury. Emergency safe shutdown of the reactor does not rely on an external source of cooling water.

The individual plant arrangement is comprised of five principal building structures; the reactor building, auxiliary building, emergency power source building, access building, and turbine building. The two unit configuration employs a single radwaste building located between the two units. The reactor building, power source buildings, power source fuel storage vaults, essential service water pipe tunnel and ultimate heat sink related structures are designed to seismic category I requirements and contain safety-related equipment for accident mitigation. The nuclear island consists of the reactor building including pre-stressed concrete containment vessel and containment internal structure, auxiliary building, access building, and power source buildings. The foundation for the nuclear island is an independent base mat which supports each building. Floor elevation of the nuclear island is set 1 ft above the plant grade of 822 ft msl with the embedded depth of the nuclear island base mat at approximately 784 ft msl. The locations of these safety-related components are shown on Figure 2.1-201. The elevation for all facilities and accesses are listed in Table 2.4.1-201.

Flooding at the CPNPP Units 3 and 4 site, of SCR, and on the Brazos River are considered for potential impacts to the site and safety-related facilities. The causal mechanism considered for flooding in the immediate vicinity is local intense precipitation. Temporal distribution of the precipitation is selected to maximize the effects of local intense precipitation. The local intense precipitation is point precipitation and assumed to apply to the entire site. Therefore, spatial distribution of the precipitation is not applicable. The effects of local intense precipitation are discussed in Subsection 2.4.2. Flooding of SCR is considered as discussed in Subsection 2.4.1.2.2. Flooding on the Brazos River is considered as discussed in Subsection 2.4.1.2.

The majority of the natural surface runoff surrounding the CPNPP Unit 3 and 4 site area flows in a northerly direction into SCR. At the location of the power plant facilities, the surface drainage is directed to the yard holding pond and Probable Maximum Precipitation (PMP) ditch. Runoff collected in the yard holding pond and PMP ditch is expected to drain by overflow weirs or sheet flow into SCR. A small amount of surface runoff on the northwest side of the power plant facilities is anticipated to flow along the natural gap and piping grade towards SCR. A description of the site grading and earthwork is presented in Subsection 2.4.2.3.

A bathymetric survey was conducted in April, 2007 in the vicinity of the intake and discharge structures on Lake Granbury (Reference 2.4-202). Figure 2.4.1-204 shows the locations of waypoints used for temperature measurements, and Table 2.4.1-202 provides measurement data. Figure 2.4.1-205 depicts water depth obtained from the bathymetric survey within the portions of Lake Granbury adjacent to the intake and discharge structures. Water temperatures were taken at the surface then at 10 feet increments to a depth of 50 feet where allowable due to total depth at that location. The data reveal an approximate 8°F difference in water temperature between surface and bottom measurements.

Soil characteristics are discussed in Subsection 2.5.4. Site vicinity maps are provided in Section 2.1.

2.4.1.2 Hydrosphere

The Brazos River Basin has the largest drainage area of all basins between the Rio Grande and the Red River in Texas. Total basin drainage area is approximately 45,700 sq mi, of which approximately 43,000 sq mi are in Texas, the remainder, in New Mexico. (Reference 2.4-203) The Brazos River Basin crosses through three distinct physiographic provinces: the Great Plains, Central Lowland, and Coastal Plain (Reference 2.4-204). Watershed elevations range from about 4700 ft near the headwaters in eastern New Mexico to sea level near Freeport (Reference 2.4-201).

Rainfall runoff in the Brazos River watershed generally flows southeast from the upper reaches of Brazos River tributaries in northwest Texas and portions of New Mexico to the Gulf of Mexico. According to the Brazos River Authority (BRA) Clean River Program (Reference 2.4-273), tributaries of the Salt and Double Mountain Forks of the Brazos River are located in the Caprock watershed.

The Caprock watershed is a non-contributing watershed to the Brazos River Basin due to lack of rainfall and high evaporative rates in northwest Texas. Precipitation in this area is either absorbed by area soils or is contained in the hundreds of playa lakes in this part of the state. Playa lakes are shallow, round depressions that fill after storms then rapidly dry due to evaporation. Due to their ephemeral natures, these lakes are not monitored or assessed as part of the BRA Clean River Program. The Caprock watershed contains the ephemeral headwaters of the Brazos River identified in Figure 2.4.1-208, Yellow House Draw (Hydrologic Unit 12050001), Blackwater Draw (Hydrologic Unit 12050002), Running Water Draw (Hydrologic Unit 12050005), and White River (Hydrologic Unit 12050006).

The watershed of the Salt Fork and Double Mountain Fork of the Brazos River begins with the formation of the Double Mountain Fork of the Brazos River near the city of Tahoka in Lynn County. The Salt Fork of the Brazos River is formed in southeastern Crosby County and flows approximately 175 miles before joining with the Double Mountain Fork in Stonewall County to form the main stem of the Brazos River. Both the Double Mountain Fork and Salt Fork are shallow meandering streams. Figure 2.4.1-208 identifies the watershed containing the Salt Fork (Hydrologic Unit 12050007), Double Mountain Fork (Hydrologic Unit 12050003), and North Double Mountain Fork (Hydrologic Unit 12050004). The main stem of the Brazos is also identified as Middle Brazos-Miller(Hydrologic Unit 12060101).

The Clear Fork watershed begins in Fisher County and flows 284 miles east through Jones, Shackelford, Throckmorton, Stephens, and Young Counties, to its mouth on the Brazos River near South Bend in southern Young County. Figure 2.4.1-208 identifies the watershed containing Paint (Hydrologic Unit 12060103),

Upper Clear Fork Brazos (Hydrologic Unit 12060102), Lower Clear Fork Brazos (Hydrologic Unit 12060104), and Hubbard (Hydrologic Unit 12060105).

The Upper watershed of the Brazos River drains approximately 4725 square miles stretching from the Salt and Double Mountain Fork confluence to the impoundment at the Lake Whitney Dam. The river is generally wide with banks heavily vegetated with elm, willow, oak, and juniper trees. Figure 2.4.1-208 identifies the watershed containing Middle Brazos-Palo Pinto (Hydrologic Unit 12060201) and Middle Brazos-Lake Whitney (Hydrologic Unit 12060202).

The Aquilla Creek watershed covers about 466 square miles, begins in Johnson County flows through Hill County then discharges into the Brazos River in McLennan County. The Aquilla Creek watershed is contained in the Middle Brazos-Lake Whitney (Hydrologic Unit 12060202) and joins the Brazos below Lake Whitney.

The Bosque River begins in Erath County and drains 1652 square miles before emptying into Lake Waco in McLennan County. Figure 2.4.1-208 identifies the watershed containing North Bosque (Hydrologic Unit 12060204) and Bosque (Hydrologic Unit 12060203).

The Leon River watershed drains approximately 3750 square miles through Bell, Hamilton, Coryell, Comanche, and Eastland Counties. Figure 2.4.1-208 identifies the watershed containing Leon (Hydrologic Unit 12070201) and Cowhouse (Hydrologic Unit 12070202).

The Lampasas River watershed drains approximately 1502 square miles through Lampasas and portions of Mills, Burnet, Williamson and Bell Counties. The majority of the Lampasas River watershed drains into Stillhouse Hollow Lake. Salado Creek drains into the Lampasas River below Stillhouse Hollow Lake before the confluence with the Leon River. Much of the Lampasas River has heavily vegetated banks and is characterized by low flow conditions much of the time. Lampasas (Hydrologic Unit 12070203) is identified in Figure 2.4.1-208.

The Little River watershed drains approximately 2349 square miles through Williamson, Bell, Milam and portions of Burnet Counties. This watershed includes Lake Georgetown and Lake Granger. Figure 2.4.1-208 identifies the watershed containing San Gabriel (Hydrologic Unit 12070205) and Little (Hydrologic Unit 12070204).

The Central Brazos River watershed drains approximately 2710 square miles from Lake Brazos Dam in Waco to the mouth of the Navasota River southeast of College Station through Falls, Burleson, Robertson, and portions of McLennan and Brazos Counties. The Central Brazos is identified as Lower Brazos-Little Brazos (Hydrologic Unit 12070101) in Figure 2.4.1-208.

The Navasota River watershed drains approximately 2235 square miles through Limestone, Robertson, Brazos, Grimes and portions of Madison, Leon, and

Freestone Counties. The main stem of the river is impounded in three places in Limestone County creating Lake Mexia, Lake Springfield, and Lake Limestone. Navasota (Hydrologic Unit 12070103) is identified in Figure 2.4.1-208.

The Yegua Creek watershed drains approximately 1316 square miles through Milam, Lee, Burleson, and Washington Counties. Yegua (Hydrologic Unit 12070102) is identified in Figure 2.4.1-208.

The Upper Oyster Creek watershed drains approximately 127 square miles in Fort Bend County. This segment varies from a natural stream course to a highly modified system of canals and dams. The Upper Oyster Creek watershed is a small portion of the area of the Lower Brazos (Hydrologic Unit 12070104) identified in Figure 2.4.1-208.

The Lower Brazos watershed drains approximately 2077 square miles through Washington, Grimes, Waller, Austin, Fort Bend, and Brazoria counties before discharging into the Gulf of Mexico. Lower Brazos (Hydrologic Unit 12070104) is identified in Figure 2.4.1-208.

Within the Brazos River Basin, the CPNPP site is located in the Middle-Brazos Lake Whitney watershed, USGS hydrologic unit code 12060202, and Lake Granbury is located in the Middle-Brazos Palo Pinto watershed, USGS hydrologic unit code 12060201 (Reference 2.4-205). These watersheds incorporate portions of Archer, Young, Jack, Stephens, Palo Pinto, Parker, Eastland, Erath, Hood, Somervell, Johnson, Bosque, Hill, McClennan, Limestone, and Falls counties.

Near the site, the Brazos River Channel is located in incised meanders formed by the river. These meanders may be the result of uplift of the area and sea level fluctuations after a mature meandering drainage pattern is attained. The meanders eroded through and are flanked by rock slopes confining the river within a relatively narrow channel. Immediately adjacent to the channel within the meanders is a narrow flood plain. Although accretion and erosion occur within the channel, as is typical of a meandering river, the well-defined meanders indicate that the channel location is closely confined. The geometry of the banks is governed closely by their location with respect to the meander pattern. The bank on the outside of a bend generally is steep; whereas, the bank on the inside of the bend usually has a gentler slope (Reference 2.5-201).

Because of the proximity to the site, flooding on the Brazos River is considered for potential impacts to the site and safety-related facilities. The causal mechanisms considered for flooding are precipitation, dam failures, ice effects, and flooding generated from the Gulf of Mexico. Precipitation flooding for the watershed above site is evaluated in Subsection 2.4.4. Dam failure flooding is also considered in the evaluation for existing and future conditions. Dam failures are considered coincident with probable maximum flooding to maximize the effects of precipitation and dam failures combined. In addition, coincident wind wave activity is included to maximize resulting flood levels. Flooding from ice effects are

considered in Subsection 2.4.7. Ocean surge and tsunami from the Gulf of Mexico are considered in Subsections 2.4.5 and 2.4.6.

The Texas Water Development Board (TWDB) lists 44 major reservoirs within the watershed of the Brazos River Basin (Reference 2.4-206). These reservoirs and their associated dams (Figure 2.4.1-206) are utilized for water supply, recreation, flood control, cooling, and power generation. For the safety analyses, the most significant portions of the Brazos River basin are those between Possum Kingdom Lake and Lake Whitney, including Lake Granbury, as this area exhibits closely confined basin geometry and includes the highest concentration of major main stem reserviors. As shown on Figure 2.4.1-207 there are seven large manmade impoundments located within 150 stream mi of the DeCordova Bend Dam on Lake Granbury that could affect or be affected by plant operations. These impoundments include:

- Possum Kingdom Lake, on-channel, upstream reservoir located approximately 145 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201 (Figure 2.4.1-208).
- Lake Palo Pinto, off-channel, upstream reservoir located approximately 80 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201.
- Lake Mineral Wells, off-channel, upstream reservoir located approximately 70 stream mi northwest of DeCordova Bend Dam, in Hydrologic Unit 12060201.
- Lake Granbury, the primary cooling water source for CPNPP Units 3 and 4, on-channel reservoir located approximately 7 mi northeast of the CPNPP site, in Hydrologic Unit 12060201.
- SCR, off-channel reservoir located adjacent north and east of CPNPP Units 3 and 4, in Hydrologic Unit 12060202.
- Wheeler Branch Reservoir, off-channel reservoir located approximately 2 mi south of CPNPP Units 3 and 4, in Hydrologic Unit 12060202.
- Lake Whitney, on-channel, downstream reservoir located approximately 70 stream mi south of DeCordova Bend Dam in Hydrologic Unit 12060202.

Possum Kingdom Lake and Lake Granbury are operated by the Brazos River Authority (BRA), Lake Whitney by the USACE, Lake Palo Pinto by the Palo Pinto Water District No. 1, Lake Mineral Wells by the City of Mineral Wells, SCR by Luminant, and Wheeler Branch Reservoir by the Somervell County Water District. Table 2.4.1-203 provides information on dam and reservoir specifications for these impoundments.

The U.S. Army Corps of Engineers (USACE) maintains water flow rates on its website (Reference 2.4-207) for each day of the year for the major impoundments on the Brazos River, including Possum Kingdom Lake, Lake Granbury, and Lake Whitney.

Reservoir yields for the years 2000 and 2060 were obtained from the 2006 Brazos G Regional Water Plan (Reference 2.4-208). The firm yield is the greatest amount a reservoir could have supplied without shortage during a repeat of historical hydrologic conditions. Safe yield is defined as the amount of water that can be diverted from a reservoir during a repeat of the worst drought of record while still maintaining a reserve capacity equal to a 1-yr supply. Utilization of safe yield versus firm yield is a common practice in west Texas. Safe yield provides additional assurance of supply in an area where water resource alternatives are limited. Reservoir yields were limited to authorized diversions, and the period of record for the firm yield analyses was for the years 1940 through 1997.

For the dam failure analysis discussed in Subsection 2.4.4, the peak flow of the PMF coincident with assumed hydrologic domino-type dam failure of Fort Phantom Hill Dam, the proposed Cedar Ridge Reservoir Dam, Stamford Dam, Morris Sheppard Dam, and De Cordova Bend Dam at the Brazos River and the Paluxy River confluence were analyzed. Using a qualitative analysis approach based on a comparison of distance from the Brazos River and the Paluxy confluence, storage capacity, dam height, and drainage area along with the assumption of transposition of resulting dam failure effects without attenuation, it was determined that the controlling dam failure scenario would include domino-failure of Morris Sheppard Dam and De Cordova Bend Dam. Upstream of Morris Sheppard Dam the significant dams are located on individual tributaries. Using the qualitative approach, Hubbard Creek Dam was determined to be the controlling dam failure scenario upstream of Morris Sheppard Dam. However, considering future conditions, the domino-failure of Fort Phantom Hill Dam and the proposed Cedar Ridge Reservoir Dam along with the simultaneous failure of Lake Stamford Dam was determined to be the controlling dam failure scenario upstream of Morris Sheppard Dam. By quantitative analysis, it was determined that future conditions provide the controlling dam failure scenario. A complete description of the qualitative and quantitative analyses is provided in Subsection 2.4.4.

According to the 2006 Brazos Region G Water Plan, most of the sites in the state that are readily amenable to reservoir development have already been utilized. Many other sites that are amenable to reservoir development have not been thoroughly developed as potential water supplies, even though they have been studied for many years. These projects have been mentioned in previous state water plans, but have not been developed due to permitting problems, environmental impacts, water quality, or cost considerations. Over the last 10 to 20 years, the development of major reservoirs has slowed considerably due to stringent permitting requirements and increased environmental awareness. For these reasons, any major reservoir should be considered only as a long-term solution for the development of the project. If the project is taken to fruition, it would most likely take more than 10 years.

Seven potential upstream reservoir sites were evaluated in the 2006 Brazos Region G Water Plan (Reference 2.4-208). For the dam failure analysis, the development potential of each potential reservoir site was considered. All but one of these potential sites, the South Bend Reservoir, were found to contain less storage than Possum Kingdom Lake.

The site known as South Bend, located approximately 251 miles upstream of the Brazos River and Paluxy River confluence, could store up to 771,604 ac-ft. This site was not recommended as a water management strategy in the 2011 Region G Water Plan, which indicates that development of the site known as South Bend Reservoir would encounter difficult permitting constraints and would likely require significant treatment due to water quality concerns. Although this site would be one of the closest to the Brazos River and Paluxy River confluence and would impound a greater volume of water than any reservoir upstream of the confluence, the potential site has not been recommended as a water management strategy for Region G, is not a proposed reservoir, and was not included in the dam failure analysis.

Similarly, the two Double Mountain Fork Reservoir alternatives, the Lake Palo Pinto Off-Channel Reservoir and the Throckmorton Reservoir are not identified in the 2011 Region G Water Plan as recommended water management strategies. Therefore, these sites are not proposed reservoirs and are not included in the dam failure analysis. The sites recommend as water management strategies are included for consideration in the dam failure analysis, as described in Subsection 2.4.4.

Three potential reservoir sites are identified in the 2011 Llano Estacado (Region O) Water Plan (Reference 2.4-269) as recommended water management strategies. Therefore, these sites are included for consideration in the dam failure analysis as described in Subsection 2.4.4.

Based on information from the 2006 Brazos Region G (Reference 2.4-208) and 2006 Region H (Reference 2.4-270) water plans, there are no proposed main stem reservoirs downstream of Lake Whitney. Failure of downstream structures would reduce the effects of upstream dam failure and were not considered in the dam failure analysis. Similarly, failures of downstream off-channel structures were not considered.

2.4.1.2.1 Brazos River and Lake Granbury

Principal streams that enter the 145-mi segment of the Brazos River between Morris-Sheppard Dam on Possum Kingdom Lake and DeCordova Bend Dam include Palo Pinto and Rock Creeks. Along this segment, the Brazos River has a slope of 0.04 percent, and a gradient of 2.117 ft/mi. The additional catchment area between the two dams is about 2140 sq mi, all of which contribute to flow in the Brazos River (Reference 2.4-201). Approximate lengths and slopes of these streams are presented in Table 2.4.1-204.

The principal tributaries of the Brazos River above the Morris-Sheppard Dam that impounds Possum Kingdom Lake are the Salt, Double Mountain, and Clear forks of the Brazos River (Figure 2.4.1-209). The catchment area above Morris-Sheppard Dam (Figure 2.4.1-207) is about 22,550 sq mi, of which about 9240 sq mi are probably non-contributing. Of the contributing area, nearly half is in the Clear Fork Basin (Reference 2.4-201).

There are six intermittent streams that flow into Lake Granbury within a 6-mi radius of the Units 3 and 4 intake and discharge structures upstream of the DeCordova Bend Dam (Figure 2.4.1-210). These streams include Lusk Branch, Walnut Creek, Contrary Creek, Rough Creek, Lambert Branch, and Rucker Creek. Approximate lengths and slopes of these streams are presented in Table 2.4.1-205.

Water Rights Permit No. 2111, issued July 24, 1964, authorized the BRA to construct and maintain a dam and reservoir (Lake Granbury) on the Brazos River, to impound and not exceed 155,000 ac-ft of water. The BRA was permitted to divert and use not to exceed 10,000 ac-ft/yr of water for municipal purposes, 70,000 ac-ft/yr for industrial purposes, 20,000 ac-ft/yr for irrigation and 350,000 ac-ft/yr for hydroelectric power generation. Several amendments were made to Permit 2111 in the following years. On September 28, 1966, the authorization to divert 350,000 ac-ft/yr of water for hydroelectric power generation was deleted and on September 13, 1979 the impounded waters of Lake Granbury was approved for recreational purposes. A change in water use resulted in another amendment to the Permit that was approved on November 25, 1980. It allowed the permittee to use 500 ac-ft of the 20,000 ac-ft of water designated for irrigation to be used for mining purposes.

The Certificate of Adjudication, No. 12-5156, was issued to the BRA on December 14, 1987. It grants the BRA the right to impound and use the waters of Lake Granbury as previously described along with several "Special Conditions" concerning the "Systems Operations Order." The priority rights of Lake Granbury also fall under the order of Certificate of Adjudication 5167 for the purpose of system operation as authorized by Commission Order of July 23, 1964, as amended and as modified, by the Commission's final determination of all claims of water rights in the Brazos River Basin and the San Jacinto-Brazos Coastal Basin maintained by the BRA, the Fort Bend County W.C.I.D. No. 1 and the Galveston County Water Authority on June 26,1985 (Reference 2.4-209).

A review of USGS reservoir gauge data indicates the surface water elevation at Lake Granbury is kept at approximately 692.5 ft above msl (Reference 2.4-210). Graphs of daily reservoir elevation and storage from October 2002 to September 2007 for Lake Granbury are shown on Figure 2.4.1-211. Constant water level at Lake Granbury is maintained by an open spillway and retention time has been estimated at 260 days (Reference 2.4-211). Yield analysis for Lake Granbury indicates a firm yield of 64,712 ac-ft in 2000 and 63,212 ac-ft in 2060 (Reference 2.4-208).

The results of the 2003 TWDB Volumetric Survey indicate Lake Granbury has a volume of 129,011 ac-ft, and extends across 7945 surface ac at the conservation pool elevation of 693.0 ft above msl. (Reference 2.4-209) The 693.0 conservation pool elevation equals the elevation of the top of the gates for DeCordova Bend Dam and represents maximum storage capacity. (Reference 2.4-263) The revised TWDB 1994 survey report (1993 field survey) found 7949 surface ac and a total volume of 131,593 ac-ft. (Reference 2.4-209)

Comparison of the revised 1993 field survey to the current 2003 TWDB Volumetric Survey of Lake Granbury show little or no change in surface area and a 2 percent reduction in total volume at the top of the conservation pool. Most of this reduction appears to be in the area of continued deltaic accretion in the upper reaches of Lake Granbury where the Brazos River enters the main body of the reservoir. (Reference 2.4-209)

2.4.1.2.2 Squaw Creek and Squaw Creek Reservoir

SCR, the cooling water source for CPNPP Units 1 and 2 is located on Squaw Creek in Hood and Somervell Counties, approximately 4.3 mi north of the creek's confluence with the Paluxy River (Reference 2.4-201). At the conservation pool elevation (775.0 ft above msl), the lake has approximately 36 mi of shoreline and is 5 mi long. At the dam site the reservoir has a drainage area of 64 sq mi. Squaw Creek Dam and Reservoir are owned and operated by Luminant.

There are six intermittent streams that flow into the SCR within a 6-mi radius of CPNPP Units 3 and 4 upstream of the Squaw Creek Dam (Figure 2.4.1-210). These streams include Squaw Creek, Panter Branch, Lollar Branch, Panther Branch, Million Branch, and an unnamed stream branch. Approximate lengths and slopes of these streams are presented in Table 2.4.1-205.

The results of the 1997 TWDB Volumetric Survey indicate SCR has a volume of 151,418 ac-ft, and extends across 3297 surface ac at the conservation pool elevation of 775.0 ft above msl. Within the lake, the survey determined that the Squaw Creek safe shutdown impoundment (SSI) held 701 ac-ft, spread over a surface area of 53 ac. (Reference 2.4-212)

Yield analysis for SCR indicates a firm yield of 8830 ac-ft/yr in 2000 and 8710 ac-ft/yr in 2060 (Reference 2.4-208).

Because SCR is adjacent to the site, flooding of SCR is considered for potential impacts to the site and safety-related facilities. The causal mechanisms considered for flooding are precipitation over the Squaw Creek watershed, backwater analysis due to downstream flooding, wind induced surge, seismic induced surge or seiche, and landslide induced seiche. Precipitation flooding for the Squaw Creek watershed is evaluated in Subsection 2.4.3. Temporal and spatial distributions of the precipitation are examined to determine the maximum runoff and resulting water surface elevation. In addition, coincident wind wave activity is included to maximize resulting flood levels. The backwater analysis

considers downstream flooding from adjacent watersheds and the Brazos River including dam failures. Surges are considered in Subsection 2.4.5. Seiches are considered in Subsection 2.4.6.

2.4.1.2.3 Water Control Structures

2.4.1.2.3.1 New Water Control Structures

Lake Granbury is bounded by two existing dams; DeCordova Bend Dam is located approximately 1.31 mi downstream of the CPNPP intake structures and Morris Sheppard Dam is located approximately 145 river miles upstream from the DeCordova Bend Dam. Both of these dams are owned and operated by the BRA and are primarily used for water supply, with secondary uses that include recreation, flood control, cooling, and power generation. No additional water control structures are planned or required for the facility.

2.4.1.2.3.2 Makeup Water Intake Structure

The Makeup Water Intake Structure is a reinforced concrete box-type structure housing the makeup water pumps, makeup water jockey pump, strainers, valves and associated piping. There is no safety-related equipment in the Circulating Water System, nor does loss of its normal operating capability adversely affect any safety-related components.

The intake structure is located approximately 1.31 mi upstream from the DeCordova Bend Dam. The blowdown water from the Circulating Water System is discharged through a separate pipeline back to Lake Granbury about 1.14 mi downstream from the intake structure

The bottom of the intake structure is at elevation 666.0 ft msl. Under agreement with Luminant, the BRA maintains a minimum pool elevation of 675 ft msl. The operating deck is at elevation 700.0 ft msl, which is below the DeCordova Bend Dam maximum elevation of 706 ft msl. The structure houses 5 pumps sized to adequately supply the required Circulating Water System flow of 32,700 gpm per unit. Service water is bled off the Circulating Water System flow and is expected to provide one hundred percent of the required make-up to the Service Water System under normal operating conditions and during periods of peak demand.

Screens provide course screening of floating and suspended debris, and prevent aquatic life from entering the structure. All screens are the single flow through automatic cleaning type. Two screens are provided for each of the two supply loops at the inlet to the intake structure. Each of the two screens on each loop has sufficient capacity to screen the total water required for one loop. The intake screens are sized so that the thru-screen flow velocity is less than 0.15 mps (0.5 fps). If fouling occurs, the screens are cleaned by air burst backwash.

Due to the depth and location of the intake structures on Lake Granbury, it is not anticipated that maintenance de-silting to remove sediment is necessary.

2.4.1.2.3.3 DeCordova Bend Dam

DeCordova Bend Dam impounds Lake Granbury on the Brazos River approximately 145 stream mi southeast of Morris Sheppard Dam and approximately 7.5 mi southeast of Granbury, at mile BRM 542.5. The lake was built by the BRA for the conservation of water for irrigation, municipal, and industrial uses and was completed in 1969. Lake Granbury and associated DeCordova Bend Dam are owned by the BRA. (Reference 2.4-267) Lake Granbury inundates approximately 33 mi of the original Brazos river bed (Reference 2.4-213) and has a drainage area of 25,679 sq mi (Reference 2.4-267).

Ambursen Engineering Corp. of Houston designed the dam and the H. B. Zachry Company was the contractor. Construction began in December, 1966 and deliberate impoundment commenced September 15, 1969. The earth-rolled embankment is 2200 ft long, with a maximum height of 84 ft at elevation 706.5 ft msl. The service spillway is a gate-controlled ogee crest. There are 16 tainter gates, each 36 ft long by 35 ft high that have a crest elevation of 658.0 ft above msl. Outlet works consist of two 84 in by 96 in openings, motor-controlled by sluice gates with invert elevations at 652.0 and 640.0 ft above msl. (Reference 2.4-213)

DeCordova Bend Dam is expected to overtop during the Probable Maximum Flood event (described in further detail in Subsection 2.4.2). No seismic rating criteria have been published.

2.4.1.2.3.4 Squaw Creek Dam

Records indicate the construction for Squaw Creek Dam began on November 17, 1974, and was completed on June 16, 1977. Freese and Nichols Consulting Engineers of Fort Worth designed the facility, and Brown and Root Inc. managed the construction project. Squaw Creek Dam and appurtenant structures consist of an earthfill embankment 4360 ft in length, with a maximum height of 159 ft and a crest elevation of 796.0 ft. The service spillway is an uncontrolled concrete ogee type located between the right (southwest) end of the embankment and abutment. The crest of the spillway is 100 ft in width at elevation 775.0 ft. The emergency spillway is an earthcut channel through bedrock located at the left abutment, northeast of the embankment. The width of the channel is 2200 ft, with a crest elevation of 783.0 ft. The service outlet structure consists of a concrete tower housing three gate-controlled outlets with invert elevations of 764.0 ft, 715.0 ft and 666.5 ft. The 30-in diameter low-flow outlet has an invert elevation of 653.0 ft. All discharges from the outlet tower pass through a six foot diameter concrete encased conduit and are released downstream of the embankment (Reference 2.4-212).

Contained within SCR, is a smaller reservoir known as the Safe Shutdown Impoundment (SSI). The smaller reservoir is designed to provide cooling water during an emergency situation to safely shutdown CPNPP Units 1 and 2. The SSI

Dam is located on Panther Branch, a tributary of Squaw Creek. The dam is composed of an earthfill embankment, approximately 1520 ft long. The maximum height of the embankment is 70 ft above the natural streambed. The 40 ft wide crest is at elevation 796.0 ft. The service/emergency spillway is a 40 ft wide by 400 ft long earthcut channel connecting the SSI facility to the main reservoir. This ingress/egress channel, located to the right (south) of the SSI Dam, is also referred to as the equalization channel for the two reservoirs. The flow of water between the two reservoirs is controlled by a 3 ft tall by 3 ft wide concrete weir that extends the width of the channel with a flowline elevation of 769.5 ft. (Reference 2.4-212)

Squaw Creek Dam is designed to withstand the Probable Maximum Flood event (described in further detail in Subsection 2.4.2). No seismic rating criteria have been published for the Squaw Creek Dam; however, the SSI Dam is a seismic Category I structure (Reference 2.4-214).

2.4.1.2.4 Surface Water Use

Surface water quality in SCR is slightly saline and is currently used for CPNPP Units 1 and 2 cooling with reservoir make up water coming from Lake Granbury. Surface water quality in Lake Granbury is slightly saline and five municipal water systems obtain water from the Brazos River Authority’s (BRA) Lake Granbury Surface Water and Treatment System (SWATS) as their sole or primary water supply (Table 2.4.1-206) (Reference 2.4-215). The closest municipal user to the CPNPP Lake Granbury discharge is the Lake Granbury SWATS, located approximately 3.45 mi upstream. There are no downstream municipal users between the CPNPP Lake Granbury discharge and the City of Waco, Texas, approximately 65 mi south-southwest. The closest industrial user is the Wolf Hollow electric power plant, with an intake located approximately 150 ft downstream from the CPNPP intake structures on Lake Granbury. The closest upstream industrial user is the DeCordova Bend electrical power plant located approximately 1.56 mi from the CPNPP Lake Granbury intake.

Non-consumptive water uses, such as navigation, hydroelectric generation, environmental flows, and recreation, are not reported by the TWDB. The water use reported by the TWDB annual survey covers consumptive withdrawals only and does not include net use by category or water return information. Additionally, the TWDB reports water use by category on an annual basis and monthly use rates are not provided in the data.

The TWDB estimates total water use within the Brazos River Basin in 2004 was 3,544,885 ac-ft (Reference 2.4-216). Approximately 75 percent of this annual use was for irrigation, 11 percent for municipal use, 6 percent for steam electric use, 5 percent for manufacturing use, 2 percent for livestock use, and 1 percent for mining use. Table 2.4.1-207 provides 2004 water use estimates by category for the Brazos River Basin.

The portion of the Brazos River catchment between Possum Kingdom Lake and Lake Whitney encompasses portions of Palo Pinto, Parker, Hood, Somervell, Bosque, and Hill counties. Surface water use estimates for users with allocated water rights of 500 ac-ft or more in these counties were obtained from the Texas Commission on Environmental Quality (TCEQ). The 2006 monthly withdrawal data for users in this area are provided in Table 2.4.1-208, and the locations of major water rights in the Brazos River Basin are shown on Figure 2.4.1-212.

In Palo Pinto County in 2006, the BRA reported diversions from the Brazos River, Possum Kingdom Lake area, of 160,311 ac-ft for municipal, hydroelectric, mining, irrigation, industrial, and other uses. Also in Palo Pinto County, the Palo Pinto Municipal Water District reported a diversion from Palo Pinto Creek, Lake Palo Pinto area, of 4800 ac-ft for municipal use, and the Rocking W. Ranch reported a diversion of 647 ac-ft from the Brazos River for irrigation use.

In Parker County, the City of Mineral Wells reported a diversion of 54 ac-ft from Rock Creek, Lake Mineral Wells area, for municipal use. No diversion amount was reported in 2006 by the TXI Operations company for industrial and irrigation use.

In Hood County, the BRA reported diversions of 56,815 ac-ft from the Brazos River, Lake Granbury area, for municipal, industrial, irrigation, and mining uses.

In Somervell County, a diversion of 3,367,805 ac-ft was reported from SCR, Panther Branch, and Lake Granbury. This total includes diversion from Lake Granbury as well as circulation water estimates through the once through cooling system for CPNPP Units 1 and 2. In 2006, no diversion amount from the Paluxy River was reported by the Somervell County Water District for municipal use.

In Bosque County in 2006, Chisholm Trails Adventures reported a diversion of 3621 ac-ft from the Brazos River, downstream of Lake Granbury, for irrigation use. The cities of Meridian and Clifton were identified as having significant water rights in Bosque County; however, diversions for these municipalities are on the North Bosque River and cannot affect or be impacted by CPNPP operations.

In Hill County, the BRA reported diversions of 7302 ac-ft from the Brazos River, Lake Whitney area, for municipal and industrial uses.

2.4.1.2.5 Groundwater Use

Twelve existing water wells were identified on the CPNPP site. The wells include seven active potable water wells that support CPNPP Units 1 and 2 operations, one inactive potable water well associated with Squaw Creek Park, and four observation wells. Information regarding these wells is provided in Table 2.4.1-209, and the well locations are shown on Figure 2.4.1-213. On-site groundwater withdrawal information for 2006 was obtained from an annual report provided by Luminant. The report indicated on-site withdrawals of 27.90 ac-ft (9,092,700 gal) from five active wells in 2006, which is a use rate of 24,911.5 gpd or approximately 17.3 gpm (Reference 2.4-217). Monthly use data for 2006 is provided in Table

2.4.1-210. Luminant is not anticipating using groundwater as an operational or safety-related source of water for CPNPP Units 3 and 4, and has implemented a conservation plan for future groundwater withdrawals at the CPNPP site. During construction of CPNPP Units 3 and 4, and during operation of CPNPP Units 1 through 4, potable water is planned to be supplied by the Somervell County Water District's water supply system. Water for temporary fire protection, concrete batching, and other construction uses is expected to be supplied by the Somervell County Water District. Ground water conservation at CPNPP has voluntarily been an environmental commitment with the TCEQ, Clean Texas Program, since 2003 and with the EPA Performance Track Program since 2005. CPNPP has reduced groundwater use from ~ 50 gpm in mid-90's to ~ 16 gpm during 2007.

Groundwater is not expected to be used for construction or operation at the CPNPP Units 3 and 4 site. Groundwater is fully discussed in Subsection 2.4.12.

2.4.2 Floods

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.2.

2.4.2.1 Flood History

Floods in Texas typically are associated with thunderstorms during the summer and hurricanes and tropical storms in the late summer through early fall. (Reference 2.4-228) Historical flooding in the Brazos River watershed above the site has been a result of precipitation runoff. There are no known historical floods due to dam failures, surges, seiches, tsunamis, ice jams, or landslides. Dam failures are discussed in Subsection 2.4.4. Surge and seiches are discussed in Subsection 2.4.5. Tsunamis are discussed in Subsection 2.4.6. Ice effects are discussed in Subsection 2.4.7. Landslides are discussed in Subsection 2.4.9. The maximum recorded water surface elevation associated with floods of record for all rivers and streams in the vicinity are significantly lower than the Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 site grade as discussed below.

The greatest known flood of the Brazos River occurred in 1876 prior to any monitoring. Therefore, quantitative data for this event do not exist (Reference 2.4-214). The USGS gage (08091000) on the Brazos River nearest to the site is located near Glen Rose, Texas just upstream of the confluence with the Paluxy River. Although there are no flood control dams upstream of the gage on the Brazos River, the gage is subject to regulation by Morris Sheppard Dam, completed in 1941 and impounding Possum Kingdom Lake, and DeCordova Bend Dam, completed in 1969 and impounding Lake Granbury. (Reference 2.4-222) The gage drainage area is 25,818 sq mi. The contributing drainage area of the gage is 16,252 sq mi (Reference 2.4-224) and the gage location is shown in Figure 2.4.2-201.

The peak flow measurement period of record for the gage 08091000 is from 1923 to the present. The maximum recorded water surface elevation of 603.58 ft msl occurred on April 28, 1990 and corresponded to a discharge of 79,800 cfs. The discharge was exceeded in 1991, 1981, 1957, and 1935. However, the recorded water surface elevations were less than the flood elevation occurring in 1990. The maximum recorded discharge of 97,600 cfs occurred on May 18, 1935 (Reference 2.4-224). The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-201. The datum for USGS gage (08091000) is reported in North American Datum 1927 (NAD27) and National Geodetic Vertical Datum of 1929 (NGVD29).

The Paluxy River is a tributary of the Brazos River. A USGS gage (08091500) is located upstream of the confluence with the Squaw Creek tributary near Glen Rose, Texas. The gage drainage area is 410 sq mi (Reference 2.4-225) and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage contains periodic measurements in 1908, 1918, and 1922 and is continuous from 1948 to the present. (Reference 2.4-220) The maximum recorded water surface elevation of 636.86 ft msl occurred on April 17, 1908 and

corresponded to the maximum recorded discharge of 59,000 cfs (Reference 2.4-225). The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-202. The datum for USGS gage (08091500) is reported in NAD27 and NGDV29.

The USGS gage (08091750) closest to the site is located on Squaw Creek just below the SCR. The gage drainage area is 70.3 sq mi (Reference 2.4-226) and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage is from 1973 to 2006. (Reference 2.4-220) The maximum recorded water surface elevation of 610.90 ft msl occurred on April 8, 1975 and corresponded to the maximum recorded discharge of 9030 cfs. (Reference 2.4-226) Squaw Creek Dam, impounding SCR, was completed in 1977. (Reference 2.4-222) Since completion of the Squaw Creek Dam, the maximum recorded water surface elevation of 610.85 ft msl occurred on June 13, 1989 and corresponded to the maximum recorded discharge of 8940 cfs. (Reference 2.4-220) The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-203. The datum for USGS gage (08091500) is reported in NAD27 and NGDV29.

Prior to completion of the Squaw Creek Dam, a USGS gage (08091700) was located upstream of the site on the Panter Branch, a tributary of Squaw Creek. The gage drainage area is 7.82 sq mi and the gage location is shown in Figure 2.4.2-201. The peak flow measurement period of record for the gage is from 1966 to 1973. The maximum recorded water surface elevation of 904.88 ft msl occurred on September 16, 1972 and corresponded to the maximum recorded discharge of 3750 cfs. (Reference 2.4-220) The annual peak stage and discharge measurements for the period of record are provided in Table 2.4.2-204. The datum for USGS gage (08091700) is reported in NAD27 and NAVD88.

2.4.2.2 Flood Design Considerations

By examination of the vicinity of CPNPP Units 3 and 4 site and area topography, it was determined that the flooding potential at the site would originate from local intense precipitation, the adjacent SCR, or the Brazos River and the Squaw Creek or the Paluxy River tributaries. Squaw Creek joins the Paluxy River just below SCR. The Paluxy River joins the Brazos River just below the junction with Squaw Creek. In addition, coincident wind wave activity is considered.

The local intense precipitation analysis is approached conservatively. The precipitation selected is the point PMP at the most critical temporal distribution and assumed to apply to the entire site. No losses are assumed. All rainfall is converted to runoff. Conservative estimates for roughness coefficients are utilized in the determination of peak flows. Downstream boundary conditions are based on the maximum water surface elevation for SCR and account for datum conversion.

The SCR flooding analysis is approached conservatively. The PMP is maximized for SCR watershed using the critical storm center, orientation, and temporal

distribution. No losses are assumed. All rainfall is converted to runoff. Baseflow is determined based on the maximum average monthly flow for a nearby stream gage. The most recent storage elevation relationship for SCR is utilized. The spillway rating curves are derived to be more conservative than the published elevation discharge curves. The service spillway is evaluated assuming a flooded tailwater condition. The emergency spillway discharge is based on downstream channel flow depth at 90 to 100 percent of the headwater elevation.

Snyder's unit hydrographs are derived based on maximizing the peaking coefficient and minimizing the lag time coefficient. The peak of the unit hydrographs is increased by 20 percent to account for the effects of nonlinear basin response. A backwater analysis downstream of Squaw Creek Dam to determine tailwater effects is performed by maximizing the flow from adjacent watersheds in conjunction with the maximum downstream elevation on the Brazos River. Conservative estimates for roughness coefficients are utilized.

The Brazos River flooding analysis is approached conservatively and considers failure of upstream dams under existing and proposed conditions. Upstream tributary dams are assumed to fail under the probable maximum flood (PMF) for the tributary dam's watershed. Dams are assumed to fail in a domino-type manner or simultaneous as applicable to determine maximum downstream effects. No attenuation is assumed and dam failure results are transposed downstream instantaneously. When considering failure of the Brazos River dams, the dam failure results that include the PMF for the tributary dams are combined with the PMF for the Brazos River, which also includes the drainage area for the tributary dams.

Antecedent reservoir elevations are based on maximum recorded elevations or higher crest elevations. Wind setup is included to maximize water surface elevations. Conservative breach parameters are utilized. Breach wave heights and breach flows are evaluated to determine the maximum downstream effects. Although tailwater is considered, conservative roughness coefficients are used to minimize the tailwater effect on the breach wave heights and breach flows, which are dependent on the difference between the headwater elevation and the tailwater elevation. In the vicinity of the site, the Brazos River has been incorporated into the stream course model utilized for the backwater analysis. Conservative roughness coefficients are utilized to maximize the resulting water surface elevation. Datum conversion is accounted for in the comparison to the site grade.

The coincident wind wave activity analysis is approached conservatively. A straight line fetch is assumed instead of using an effective fetch. The maximum PMF elevation of SCR is used to determine the maximum fetch length. The maximum appropriate wind speed for the area is used. Wind setup is included in the analysis. Runup is evaluated for slopes from 10:1 to vertical. Datum conversion is accounted for in the comparison to the site grade.

The summary results of the events evaluated to determine the worst potential flood are provided as follows:

- Probable maximum precipitation (PMP) on the total watershed and critical sub-watersheds, including seasonal variations and potential consequent dam failures, with a corresponding water surface elevation of 793.66 ft msl (discussed in Subsection 2.4.3).
- Dam failures, including a postulated domino-type failures of three upstream dams coincident with the Probable Maximum Flood (PMF), with a corresponding water surface level of 760.68 ft msl (discussed in Subsection 2.4.4).
- Two year coincident wind waves with a corresponding water surface level of 810.64 ft msl (discussed in Subsection 2.4.3).

Specific analysis of Brazos River flood levels resulting from ocean front surges, seiches, and tsunamis is not required because of the inland location and elevation characteristics of the CPNPP site. Additional details are provided in Subsections 2.4.5 and 2.4.6. Snowmelt and ice effect considerations are unnecessary because of the temperate zone location of CPNPP. Additional details are provided in Subsection 2.4.3 and Subsection 2.4.7. Flood waves from landslides into reservoirs required no specific analysis, in part because of the absence of major elevation relief. In addition, elevation characteristics of the vicinity relative to the associated water features, combined with limited slide volumes prohibit significant landslide induced flood waves. Additional details are provided in Subsection 2.4.9.

The maximum flood level at CPNPP Units 3 and 4 is elevation 793.66 ft msl. This elevation would result from a PMP on the Squaw Creek watershed, as described in Subsection 2.4.3. Coincident wind waves would create maximum waves of 16.98 ft resulting in a design basis flood elevation of 810.87 ft msl. CPNPP Units 3 and 4 safety-related plant elevation is 822 ft msl, providing more than 11 ft of freeboard under the worst potential flood considerations.

2.4.2.3 Effects of Local Intense Precipitation

CPNPP Units 3 and 4 drainage system was evaluated for the PMP on the local area. The site is graded such that overall runoff will drain away from safety-related structures directly to the SCR. The PMP flood analysis assumes that storm drainage structures within the local area are non-functioning. Computed water surface elevations in the vicinity of safety-related structures are below site grade elevation of 822 ft msl. The site grading and drainage plan is shown in Figure 2.4.2- 202.

The local intense PMP is defined by Hydrometeorological Report No. 51 (HMR 51) and No. 52 (HMR 52). PMP values for durations from 6-hr. to 72-hr. are determined using the procedures as described in HMR No. 51 for areas of 10-sq mi (Reference 2.4-218). Using the CPNPP location, the rainfall depth is read from

the HMR 51 PMP charts for each duration. The 1-sq mi PMP values for durations of 1-hour and less are determined using the procedures as described in HMR 52. (Reference 2.4-219) Using the CPNPP location, the rainfall depth for each duration is read from the HMR 52 1-sq mi PMP charts. A smooth curve is fitted to the points. The derived PMP curve is detailed in Table 2.4.2-205. The corresponding PMP depth duration curve is shown in Figure 2.4.2-203.

HMR 52 guidance indicates that PMP rates for 10-sq mi areas are the same as point rainfall. Also indicated in HMR 52, the 1-sq mi PMP rates may also be considered the point rainfall for areas less than 1-sq mi. Therefore, intensities for any drainage areas with durations longer than 1-hr. are derived from the PMP rates for 10-sq mi areas. Intensities for drainage areas with durations equal to or less than 1-hr. are derived from the PMP rates for 1-sq mi areas. The corresponding local intense PMP depth duration curve is shown in Figure 2.4.2-204. The US-APWR plant design is based on a PMP of 19.4 in/hr and 6.3 in/5 min. The derived local intense PMP and Intensity duration curve is detailed in Table 2.4.2-206. The derived Intensity Duration Curve corresponding to the local intense PMP is shown in Figure 2.4.2-205. CPNPP Units 3 and 4 site is within the plant design limits for PMP.

The areas adjoining the power block on the north and east side are open to the downward slopes leading into the SCR. This feature does not provide a barrier and allows drainage to pass freely across to the SCR under local intense precipitation.

The local site analysis for CPNPP Units 3 and 4 was based on the Grading and Drainage Plan. CPNPP Units 3 and 4 site was divided into 11 sub basins for analyzing the effects of local intense precipitation as shown in Figure 2.4.2-202. The PMP for the sub basins used to determine the peak runoff flows is based on the time of concentration. The time of concentration is calculated using the Soil Conservation Service (SCS) segmental approach as described in Technical Release 55. (Reference 2.4-221) The time of concentration (T_c) is the sum of the time for the runoff to flow from the upper part of the sub basin to the point of concentration. A combination of sheet flow, shallow flow, and channel flow conditions for the sub basins is considered in determining the total T_c.

T_c = Sheet flow T_t + Shallow concentrated flow T_t + Channel flow T_t

T_t is calculated using the following equation for Sheet Flow:

Sheet flow $T_t = \dfrac{0.007 \cdot (n \cdot L)^{0.8}}{P_2^{0.5} \cdot S^{0.4}}$ (Reference 2.4-221) (Equation 1)

where:

T_t= Sheet flow travel time (hr)

n = Mannings Friction Factor

L = Flow Length of the Runoff which is not greater than 300 (ft)

P_2 = Rainfall Depth of the 2 year 24 hour rainfall event (in)

S = Slope of the Runoff Travel Path (ft/ft)

T_t is calculated using the following equation for shallow concentrated flow and channel flow:

The Technical Release 55 minimum Manning's Friction Factors are 0.011 for smooth concrete surfaces and 0.15 for grass. Sheet flow was evaluated using Manning's Friction Factors of 0.01 smooth concrete surfaces and 0.075 for grass. Using smaller values is conservative because minimizing the friction factor also minimizes the travel time. The shorter travel times result in a greater intensity and peak runoff.

Shallow concentrated flow and channel flow $T_t = \frac{L}{3600V}$ (Reference 2.4-221)

(Equation 2)

where:

T_t= Shallow concentrated and channel flow travel time (hr)

L = Flow Length (ft)

V= Velocity of flow (fps)

For shallow concentrated flow over paved areas:

$$V = 20.328 \cdot S^{0.5}$$ (Reference 2.4-221)

S = Slope of the Runoff Travel Path (ft/ft)

For open channel flow, velocity is determined using the Manning's formula:

$$V = \frac{1}{n} K \cdot r^{\frac{2}{3}} \cdot s^{\frac{1}{2}}$$ (Reference 2.4-221)

where:

V = Velocity of open channel flow (fps)

K = constant = 1.49 for English units

r = hydraulic radius = a/ p_w (ft)

a = cross sectional flow area in (sq ft)

p_w = wetted perimeter (ft)

s = slope of hydraulic grade line (ft/ft)

n = Manning's roughness coefficient for open channel flow

For open channel flow, according to Chow (Reference 2.4-223), the minimum Manning's roughness coefficient for excavated or natural channels is 0.016. Open channel flow was evaluated using a Manning's roughness coefficient of 0.015. Using smaller values is conservative because minimizing the roughness coefficient increases the velocity. Increased velocity minimizes the travel time. The shorter travel times result in a greater intensity and peak runoff.

The rational method was used to determine peak runoff rates for the drainage sub basins. The rational method is given by the equation:

$Q = C \cdot i \cdot A$ (Reference 2.4-227) (Equation 3)

where:

Q = Runoff (cfs)

C = Unitless coefficient of runoff

i = Intensity (in/hr)

A = Drainage area (ac)

No runoff losses were assumed. Therefore, the runoff coefficient was assumed equal to one. The weir equation is used to determine the PMF elevation for the peak runoff rate from the sub basins. A tail water elevation at 793.66 ft msl from a PMF at the SCR was considered for the local site analysis.

The equation for weir is given by the equation:

$Q = C_d \cdot L \cdot HW_r^{1.5}$ (Reference 2.4-223) (Equation 4)

where:

Q = runoff (cfs)

C_d= Overtopping discharge coefficient (Reference 2.4-223)

L = Crest length of overflow section (ft)

HW_r= Head water elevation for the weir (ft)

Site drainage area details are tabulated in Table 2.4.2-207. Resulting PMP water surface elevation at the points of discharge from the local site analysis are shown in Table 2.4.2-208. Drainage areas 1, 2, and 3 result in a maximum water surface elevation of 820.90 ft msl at the point of discharge W1. CPNPP Units 3 and 4 safety-related structures are located above the effects of local intense precipitation at plant elevation 822 ft msl.

Due to the temperate climate and relatively light snowfall, significant icing is not expected. Based on the site layout and grading, any potential ice accumulation on site facilities is not expected to affect flooding conditions or damage safety-related facilities. Ice effects are discussed in Subsection 2.4.7.

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.3.

The guidance in Appendix A of the U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.59 was followed in determining the probable maximum flood (PMF) by applying the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229). ANSI/ANS-2.8-1992 was issued to supersede ANSI N170-1976, which is referred to by Regulatory Guide 1.59. ANSI/ANS-2.8-1992 is the latest available standard.

The PMF was determined for the Squaw Creek watershed and routed through the SCR to determine a water surface elevation of 793.66 ft msl. The PMF for the Paluxy River watershed at the confluence with the Brazos River was also examined. The PMF for the Paluxy River and the Squaw Creek watersheds was combined with the Brazos River dam failure flood flow to determine any backwater effects that may affect the site. The Brazos River dam failure flood flow is described in Subsection 2.4.4 and includes the PMF for the Brazos River. The resulting water surface elevation downstream of the Squaw Creek Dam is 761.11 ft msl.

The CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. Therefore, PMF on rivers and streams does not present any potential hazards for CPNPP Units 3 and 4 safety-related facilities.

2.4.3.1 Probable Maximum Precipitation

The PMP is defined by HMR 51 (Reference 2.4-218) and HMR 52 (Reference 2.4-219). HMR 53 (Reference 2.4-230) may be used to derive seasonal estimates of the PMP. The PMP was determined for the Squaw Creek watershed and the combined Squaw Creek and Paluxy River watersheds to maximize the effects of flooding downstream of the SCR. Using the location of the watersheds, HMR 51 PMP charts are used to determine generalized estimates of the all-season PMP for drainage areas from 10 to 20,000 sq mi for durations from 6 to 72 hr. The resulting depth-area-duration (DAD) values are shown in Table 2.4.3-201.

HMR 52 is used to determine the aerial distribution of PMP estimates derived from HMR 51. The recommended elliptical isohyetal pattern from HMR 52, shown in Figure 2.4.3-201, is used for the watersheds. The watershed model, combining both watersheds, contains 4 subbasins and is shown in Figure 2.4.3-202. The watershed model is discussed in detail in Subsection 2.4.3.3.

HMR 52 computer software (Reference 2.4-231), developed by USACE, is used to determine the optimum storm size and orientation to produce the greatest PMP over the watersheds using the HMR 51 derived DAD table. Several storm centers were examined for each watershed to determine the critical storm center.

In accordance with Appendix A of Regulatory Guide 1.59, the 72-hr PMP storm is combined with an antecedent storm equal to 40 percent of the PMP. Therefore,

the complete sequential storm considered includes a 3-day, 40 percent PMP event followed by a 3-day dry period, which is followed by the 3-day full PMP event. Critical temporal distribution was determined by runoff analysis. Multiple temporal distributions were examined, including one-third, center, two-thirds, and end peaking arrangements.

Considering only the SCR watershed, Basin 1, the critical storm center for the SCR watershed was found to be near the Squaw Creek watershed centroid, identified as point SC X in Figure 2.4.3-202. A storm center at SC2 results in the maximum PMP for the SCR watershed. However, the storm center SC X results in a higher runoff and hence SC X is considered to be the critical storm center for the SCR watershed. The critical storm area was found to be 100 sq mi, corresponding to isohyet D in Figure 2.4.3-201. The critical storm orientation was found to be 181 degrees.

The critical 72-hr storm PMP rainfall total is 42.53 in for the SCR watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-208. The critical temporal distribution was determined by the runoff analyses to be a two-thirds peaking arrangement for the SCR watershed. The hourly temporal distribution of the 72-hr PMP rainfall for the SCR watershed, Basin 1, is provided in Table 2.4.3-209. The corresponding hyetograph is shown in Figure 2.4.3-211.

For the remaining portion of the Squaw Creek watershed and the Paluxy River watershed, the critical PMP for each basin was determined considering the combined areas for both watersheds.

For the remaining portion of the Squaw Creek watershed, Basin 2, the critical storm center was found to be near the watershed centroid, identified as point SC X in Figure 2.4.3-202. A storm center at SC2 results in the maximum PMP for the Squaw Creek watershed. The storm center SC X results in a higher runoff and hence SC X is considered to be the critical storm center for the Squaw Creek watershed. The critical storm area was found to be 700 sq mi, corresponding to isohyet H in Figure 2.4.3-201. The critical storm orientation was found to be 145 degrees.

The critical 72-hr storm PMP rainfall total is 38.46 in for the Squaw Creek watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-202. The critical temporal distribution was determined by runoff analysis to be an two-thirds peaking arrangement for the Squaw Creek watershed. The hourly two-thirds temporal distribution of the 72-hr PMP rainfall for Basin 2 is provided in Table 2.4.3-203. The corresponding hyetograph is shown in Figure 2.4.3-203.

For the Paluxy River watershed, Basins 3 and 4 are the critical storm center was found to be near the watershed centroid, identified as point PR Y in Figure 2.4.3-202. The critical storm area was found to be 450 sq mi, corresponding to isohyet G in Figure 2.4.3-201. The critical storm orientation was found to be 172 degrees.

The critical 72-hr storm PMP rainfall total is 35.08 in for the Paluxy River watershed. The standard HMR 52 temporal arrangement of 6-hr precipitation increments is provided in Table 2.4.3-204. The critical temporal distribution was determined by runoff analysis to be a two-thirds peaking arrangement for the Paluxy River watershed. The hourly temporal distributions of the 72-hr PMP rainfall for Basins 3 and 4 are provided in Table 2.4.3-205. The corresponding hyetographs are shown in Figure 2.4.3-204 and 2.4.3-212.

The watersheds do not occur in the orographic regions identified by HMR 51 and HMR 52. Additionally, the area does not contain significant changes in elevation that would require modification to the PMP. Therefore, orographic effects are not considered.

According to HMR 53, the all-season PMP estimates are associated with the warmer summer months. HMR 53 winter precipitation estimates are greatly reduced compared to the all-season PMP estimates. Additionally, snowmelt does not contribute significantly to river floods anywhere in the state (Reference 2.4-214). Therefore, snowmelt is not considered to be a factor in modeling the PMF event.

The potential dam failures consider coincident PMF flows for the Brazos River watershed. The PMP for the Brazos River was not determined. The approach detailed in Appendix B of Regulatory Guide 1.59 was used to derive the peak PMF flow directly. Potential dam failures are discussed in Subsection 2.4.4.

2.4.3.2 Precipitation Losses

For evaluation of CPNPP Units 3 and 4, no initial losses were assumed, indicating saturated antecedent moisture conditions at the onset of the antecedent storm. This assumption is more conservative than the guidance provided in ANSI/ANS-2.8-1992. Additionally, no loss rate was assumed for the duration of the modeled events. All rainfall is transformed to runoff. The runoff model is described in Subsection 2.4.3.3.

2.4.3.3 Runoff and Stream Course Models

The runoff and stream course models are based on an existing study for the SCR. The watershed and subbasins are shown in Figure 2.4.3-202. Basin 1 was further subdivided into three subbasins – 1a, 1b, and 1c. Basin 1a represents the drainage area above the SCR, Basin 1b represents the contributing area adjacent to the SCR, and Basin 1c represents the SCR. Drainage areas for each subbasin are provided in Table 2.4.3-207.

Based on USGS quadrangles, the topography of the Squaw Creek watershed generally slopes to the stream course running through the middle of the watershed. The stream course slopes to the southeast from about 1100 ft msl to a low point of 650 ft msl. However, the SCR has inundated elevations below 775 ft

msl. The highest point in the basin is the plateau peak of the geographic feature Comanche Peak at elevation 1230 ft msl (Reference 2.4-237).

The Paluxy River basin generally slopes to the river course running through the middle of the watershed. The river course slopes to the southeast from about 1450 ft msl to a low point of 570 ft msl at the confluence with the Brazos River. The highest point in the basin is elevation 1490 ft msl (Reference 2.4-237).

The USACE HEC-HMS, Version 3.4 (Reference 2.4-232), modeling software was used for rainfall runoff and routing calculations. The HEC-HMS model watershed routing layout is shown in Figure 2.4.3-205. The unit hydrographs for each basin were based on the existing study using the synthetic Snyder's Unit Hydrograph. Snyder's method was used for the CPNPP Units 1 and 2 unit hydrograph development (Reference 2.4-214), and is applicable under PMF conditions. The Snyder's method provided reasonable estimates for peak direct runoff rate at the CPNPP location and is acceptable in determining the peak direct runoff rate for the CPNPP Units 3 and 4. To represent a conservative approach, the basin characteristics resulting in higher runoff at the CPNPP Units 3 and 4 were used in the runoff model. The basin characteristics are provided in Table 2.4.3-207.

Basin area, length of stream, and length of stream to the basin centroid are measureable parameters. The basin areas from the existing study were confirmed based on USGS topography. The length of stream and the length of stream to the basin centroid were calculated and compared with the existing study results. The more conservative smaller values were used to determine unit hydrograph characteristics.

Base flow was determined using the average monthly flow of the 46 cfs from USGS Gage 08091750. The highest of these monthly flows was used as the base flow. Because the basin areas are different from gage area (70.3 sq mi), the base flow was adjusted on the basis of ratio of basin drainage area to the gage area. The adjusted baseflow was applied to the model as a constant rate and is provided in Table 2.4.3-207.

The Snyder's lag time coefficient and peaking coefficient were selected to maximize runoff. Lag time coefficients range from 1.8 to 2.2. However, lag time coefficients have been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico. Lower lag time coefficients are more conservative. Therefore, a 0.4 lag time coefficient has been selected. Peaking coefficients range from 0.4 to 0.8. Higher peaking coefficients are more conservative. Therefore, a 0.8 peaking coefficient has been selected.

Using the watershed subbasin characteristics provided in Table 2.4.3-207, the Snyder's unit hydrograph method was applied to derive unit hydrographs for each subbasin. The resulting Snyder's unit hydrograph characteristics and equations utilized are provided in Table 2.4.3-210. To account for nonlinear basin response at high rainfall rates, the peak of the unit hydrograph for each subbasin has been increased by 20 percent. The unit hydrograph was then adjusted to maintain the

unit hydrograph characteristic of 1 in of runoff. The derived and modified to account for nonlinear basin response unit hydrographs are provided for each subbasin. The Basin 1a and 1c unit hydrographs are shown in Figure 2.4.3-213. The Basin 1b unit hydrographs are shown in Figure 2.4.3-214. The Basin 2 unit hydrographs are shown in Figure 2.4.3-215. The Basin 3 unit hydrographs are shown in Figure 2.4.3-216. The Basin 4 unit hydrographs are shown in Figure 2.4.3-217.

The Muskingum-Cunge 8-point cross section method was used for the river routing reaches within the HEC-HMS model. Channel slope, length, and cross section data were developed using USGS quadrangles. Manning's roughness coefficients were based on the existing study and compared with accepted published tables by Chow (Reference 2.4-233). Squaw Creek Manning's roughness coefficients range from 0.06 for the channel to 0.09 for the overbanks. The Paluxy River Manning's roughness coefficients range from 0.045 for the channel to 0.07 for the overbanks. To account for variability and uncertainty, the Manning's roughness coefficient of 0.15 has been used within HEC-HMS and HEC-RAS.

SCR is the only significant reservoir within the Paluxy River and Squaw Creek watersheds. The storage-elevation rating curve for the SCR is provided in Figure 2.4.3-206 and was obtained from the following two sources:

- The storage-elevation data for elevation 775 ft msl and below have been obtained from the TWDB Volumetric Survey for SCR conducted in 2007. (Reference 2.4-212)
- The storage-elevation data for elevations above 775 ft msl have been obtained from the Operation and Maintenance Procedures for Squaw Creek Dam prepared by Freese and Nichols in 1997.

In order to project flows beyond those provided in the Operation and Maintenance Procedures for Squaw Creek Dam, the spillway rating curves have been reconstituted using the methods of the U.S. Bureau of Reclamation Design of Small Dams for the service spillway with an ogee crest and the methods of the Federal Highway Administration Hydraulic Design Series Number 5 for the emergency spillway. It is assumed that the ogee crest is submerged 1 ft by tailwater flooding up to elevation 776 ft. The ogee crest discharge coefficient was determined to range from 0 to 3.71 for an overtopping depth of 1 ft to 20 ft. Submergence effects cease as the depth of overtopping flow approaches 4 ft.

Although the emergency spillway crest is not affected by tailwater, submergence is accounted for based on the effects of flow in the channel immediately downstream from the spillway. The rating curve in the Operation and Maintenance Procedures accounts for downstream channel depth of flow from 100 percent to 90 percent of the overtopping headwater depth. Based on the effects of downstream flow, discharge coefficients were derived to range from 1.46 to 2.55 for an overtopping depth of 1 ft to 12 ft.

The combined service spillway and emergency spillway rating curve is provided in Figure 2.4.3-218.

Because of large magnitude flows and potential backwater effects from flooding of the Paluxy River and the Brazos River, a standard step method, unsteady-flow hydraulic analysis was also performed to assess the resulting water surface elevation downstream of Squaw Creek Dam. The USACE HEC-RAS, Version 3.1.3 (Reference 2.4-234), modeling software was used to route the flood hydrographs obtained from the HEC-HMS model.

The Paluxy River reach through Basin 3 and the Squaw Creek reach through Basin 2 were included in the HEC-RAS model. Cross sections were estimated using the existing study and USGS quadrangles. Cross section interpolations were performed as necessary to provide a stabilized HEC-RAS model.

The Basin 1 hydrograph routed through the SCR and the Paluxy River Basin 3 hydrograph from the HEC-HMS analysis were used as upstream boundary input. The Basin 2 and Basin 4 hydrographs from the HEC-HMS analysis were included as lateral inflows. A constant stage hydrograph, due to the peak dam failure flow described in Subsection 2.4.4, was used as the boundary condition at the downstream end of the Paluxy River. This is a bounding condition including the conservative assumptions that multiple PMF scenarios occur coincidentally and that the peak domino-type dam failure effects are maintained at the confluence throughout the duration of the PMF. A computation interval of 5 min was used in the HEC-RAS model.

2.4.3.4 Probable Maximum Flood Flow

Applying the precipitation, described in Subsection 2.4.3.1, with the precipitation losses, described in Subsection 2.4.3.2, to the runoff model, described in Subsection 2.4.3.3, the SCR peak PMF inflow was determined to be 319,000 cfs. The routed peak discharge from the SCR is 206,000 cfs. The resulting inflow and outflow hydrographs are shown in Figure 2.4.3-207. Position of the storm and temporal distribution of the PMP is discussed in Subsection 2.4.3.1. Discussion of dam failure is provided in Subsection 2.4.4. There are no significant current or planned upstream structures. No credit is taken for the lowering of flood levels at the site due to downstream dam failure.

Based on the individual basin controlling PMP, the peak flow for Squaw Creek Basin 2 was determined to be 31,300 cfs, using the two-thirds temporal distribution at the storm center SC X. The peak flow for Paluxy River Basin 3 was determined to be 85,000 cfs, using the two-thirds temporal distribution at the storm center PR Y. The peak flow for Paluxy River Basin 4 was determined to be 945,000 cfs, using the two-thirds temporal distribution at the storm center PR Y.

The individual basin PMP distributions provide maximum peak flows and the temporal distributions are aligned for all basins. Therefore, the maximum backwater flow is determined using the two-thirds temporal distribution at the

storm center SC X for Basin 1 and 2, and PR Y for Basin 3 and 4. The maximum backwater flow on the downstream end of the Squaw Creek Dam is 181,880 cfs. The associated backwater analysis does not provide the controlling PMF water surface elevation at the site.

2.4.3.5 Water Level Determinations

The PMF runoff, routed through the SCR, results in a peak water surface elevation of 793.0 ft msl at CPNPP Units 3 and 4. The water surface elevation is determined using the HEC-HMS runoff and routing model as described in Subsection 2.4.3.3. The hydrograph for the SCR is provided in Figure 2.4.3-208.

Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey (Reference 2.4-290), the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 ft for the site. Therefore, it is conservative to account for a maximum conversion of +0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the SCR maximum water surface elevation of 793.66 ft NAVD 88 is well below the CPNPP Units 3 and 4 safety-related structures elevation of 822 ft NAVD 88.

The standard step, unsteady-flow analysis for the Squaw Creek and the Paluxy River watersheds, resulted in a water surface elevation of 760.45 ft msl on the downstream side of the SCR. The HEC-RAS model described in Subsection 2.4.3.3 was used to translate runoff to the water surface elevation. Considering datum conversion, the resulting elevation of 761.11 ft msl is below the elevation of CPNPP Units 3 and 4 safety-related facilities and presents no hazard. In an unlikely event of achieving the water surface elevation described above, possible headcutting on the downstream slope of Squaw Creek could result in failure of the Squaw Creek Dam. However, failure would lower the water surface elevation of the SCR.

2.4.3.6 Coincident Wind Wave Activity

Fetch length was estimated based on USGS Quadrangles and the PMF maximum water surface elevation of SCR. The critical fetch length was found to be 2.7 mi originating from the east as shown in Figure 2.4.3-209. CPNPP is protected from wind wave activity from the west and south by the local topography. Wave height, setup, and runup are estimated using USACE “Coastal Engineering Manual, EM 1110-2-1100” guidance (Reference 2.4-235).

A two-year annual extreme mile wind speed of 50 mph was estimated based on ANSI/ANS-2.8-1992 as shown in Figure 2.4.3-210. The two-year annual extreme mile wind speed was adjusted for duration, based on the fetch length, level, over land or over water, and stability.The critical duration was found to be about 53 min. This corresponds to an adjusted wind speed of 49.91 mph.

Significant wave height (average height of the maximum 33-1/3 percent of waves) is estimated to be 2.76 ft, crest to trough. The maximum wave height (average height of the maximum 1 percent of waves) is estimated to be 4.59 ft., crest to trough. The corresponding wave period is 2.6 sec.

Slopes of 10:1 and 3:1, horizontal to vertical, in the vicinity of the CPNPP were used to determine the wave setup and runup. Additionally, wind wave activity at the vertical retaining wall was also examined. The runup includes wave setup. Runup for the 10:1 slopes was estimated to be 2.85 ft. Runup for the 3:1 slopes was estimated to be 6.99 ft. Runup at the vertical retaining wall on the north side of CPNPP Units 3 and 4 was estimated to be 16.90 ft.

Wind setup was estimated using additional USACE Hydrologic Engineering Requirements for Reservoirs, EM 1110-2-1420 guidance (Reference 2.4-236). The maximum wind setup was estimated to be 0.08 ft. The maximum total wind wave activity is estimated to be 16.98 ft and occurs at the vertical retaining wall. The PMF and maximum coincident wind wave activity results in a flood elevation of 810.64 ft msl. Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey, the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 in for the site. Therefore, it is conservative to account for a maximum conversion of +0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the coincident wind wave activity water surface elevation is 810.64 ft NAVD 88. The top elevation of the retaining wall is 795 ft msl. Although the coincident wind wave activity water surface elevation exceeds the top elevation of the retaining wall, the water surface elevation is maximized by assuming a vertical surface continues above elevation 805 ft msl. The CPNPP Units 3 and 4 safety-related structures are located at elevation 822 ft msl and are unaffected by flood conditions and coincident wind wave activity. In the event of Squaw Creek Dam failure, the determined fetch length would not be increased.

2.4.4 Potential Dam Failures

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.4.

There are no surface water impoundments other than small farm ponds that could impact the SCR. The small farm ponds have negligible storage capacity and a breach would have no measurable effect. Failure of downstream dams, including Squaw Creek Dam, would not affect the CPNPP Units 3 and 4.

There are currently three reservoirs located on the main stem of the Brazos River: Possum Kingdom Lake, Lake Granbury, and Lake Whitney. Each of these reservoirs is within 150 river miles of the CPNPP site and most of the main stem Brazos River reservoir storage is concentrated along this reach. Because the site is located off-channel on a tributary of the Brazos River, the most conservative approach for the critical dam failure event would be for this reach of the Brazos River to flood by way of domino-type dam failure of upstream dams, and for flood waters to back up from the Brazos River and Paluxy River confluence onto the site by way of the Squaw Creek catchment. For the dam failure analysis, the peak flow of the probable maximum flood (PMF) coincident with assumed hydrologic domino-type dam failure of upstream dams were analyzed at the Brazos River and the Paluxy River confluence. Morris Sheppard Dam and De Cordova Bend Dam are located within the portion of the Brazos River Basin identified as most significant for the dam failure analysis.Dam failures are included coincident with PMF flows and transposed downstream without any attenuation. Thus, the closely confined basin geometry of this reach and the concentration of major reservoirs were used as the basis for determining this portion of the basin as the most significant for the dam failure analysis.

Upstream dams are evaluated qualitatively to determine inclusion or exclusion from the critical dam failure scenario. The qualitative analysis considers both existing and future conditions, and is performed based on a comparison of distance from the confluence of the Paluxy River with the Brazos River, reservoir storage, dam height, and drainage area. Domino-type failures and simultaneous failures are postulated when applicable.

For existing conditions the qualitative analysis identifies the potential controlling domino-type dam failure scenario including Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam. For future conditions the qualitative analysis identifies the potential controlling domino-type dam failure scenario including Fort Phantom Hill Dam, proposed Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition Lake Stamford Dam is assumed to fail simultaneously with the Cedar Ridge Reservoir Dam. The two potential controlling scenarios are evaluated quantitatively to determine that future conditions provide the critical dam failure scenario.

The guidance in Appendix B of NRC Regulatory Guide 1.59 is used as an alternative approach to determine the coincident PMF. The Brazos River

watershed for existing conditions is identified in Figure 2.4.4-201. There are no safety-related structures that could be affected by flooding due to dam failures.

2.4.4.1 Dam Failure Permutations

SCR is located immediately downstream of the site. Squaw Creek is a tributary of the Paluxy River, which is a tributary of the Brazos River. Hubbard Creek Dam is located upstream of the site on a tributary of the Brazos River. Morris Sheppard Dam and DeCordova Bend Dam are located upstream of the site on the Brazos River. Lake Whitney Dam is located downstream of the site on the Brazos River.

Structural analysis of each structure has not been performed as part of this analysis. The potential backwater effects of dam failures on the Brazos River are examined assuming hydrologic failure of dams coincident with the PMF. The PMF is a more extreme event than the safe shutdown earthquake coincident with the peak of the 25-year flood, and the operating basis earthquake coincident with the peak of the one-half PMF or the 500-year flood. Seismic dam failure coincident with lesser flooding would result in lower flood elevations and has not been examined.

Qualitative Assessment for Dam Failure Analysis

Potential dam failures have been considered for dams located in the Lake Whitney watershed. Lake Whitney Dam is located on the Brazos River approximately 56 river miles downstream from the confluence with the Paluxy River. The site is located on SCR approximately 5 river miles upstream from the confluence of the Brazos River and the Paluxy River.

The distance from the confluence, reservoir storage, dam height, and drainage area are used as the basis for a qualitative assessment of dams to determine dam failure permutations that would warrant a quantitative assessment. Considering existing conditions, information for dams located in the Lake Whitney watershed has been obtained from the National Atlas (Reference 2.4-274), supplemented with information obtained from the U.S. Army Corps of Engineers National Inventory of Dams database (Reference 2.4-222), and is provided in Table 2.4.4-201. Wheeler Branch Dam and the associated Paluxy River Channel Dam are recently completed structures and have not been included in the National Atlas. Data for these structures have been obtained from the Somervell County Water District (Reference 2.4-275) and the 2011 Brazos G Regional Water Plan (Reference 2.4-276). The locations of the dams are shown on Figure 2.4.4-204.

Existing Conditions

Downstream Dams

There are three dams (Lake Pat Cleburne Dam, Cleburne State Park Lake Dam, and Lake Virginia Dam) located upstream from Lake Whitney but downstream from the confluence. The total maximum storage capacity of the three dams is

approximately 71,000 ac.-ft. Failure effects of these structures would continue downstream to Lake Whitney. Failure effects at the confluence from any combination of these structures would not exceed more critical dam failure permutations discussed below.

There are a number of dams located upstream of the confluence in the Paluxy River watershed. Including the recently completed Wheeler Branch Dam and associated Paluxy River Channel Dam, the total maximum storage capacity is approximately 42,000 ac.-ft. Failure effects at the confluence from any combination of these structures would not exceed more critical dam failure permutations discussed below.

Brazos River Upstream Dams to Morris Sheppard Dam

Lake Granbury, formed by De Cordova Bend Dam, is the largest reservoir (136,823 ac.-ft normal storage capacity and 240,640 ac.-ft maximum storage capacity) in the immediate vicinity of the confluence and is located approximately 33 river miles upstream on the Brazos River. There are no other dams located on the Brazos River between Lake Granbury and the confluence.

Possum Kingdom Reservoir, formed by Morris Sheppard Dam, is the largest reservoir (the normal and maximum storage capacity is listed as 556,220 ac.-ft) immediately upstream from Lake Granbury. Morris Sheppard Dam is located on the Brazos River approximately 129 river miles upstream of De Cordova Bend Dam. Failure of Morris Sheppard Dam would enhance the postulated failure at De Cordova Bend Dam.

Upstream of Lake Granbury, Lake Palo Pinto Dam was also considered as a candidate that would enhance the postulated failure at De Cordova Bend Dam and the effects at the confluence. Although Lake Palo Pinto Dam is closer to Lake Granbury than Morris Sheppard Dam, Lake Palo Pinto (44,100 ac.-ft normal storage capacity and 170,735 ac.-ft maximum storage capacity) is significantly smaller. The quantitative assessment is based on breach flow and breach wave height and is dependent on the headwater and dam height. Additionally, the failure effects are transposed downstream without attenuation. The dam height of Morris Sheppard Dam is higher than Lake Palo Pinto Dam. Therefore, it would be more conservative to consider the added effects from Morris Sheppard Dam failure in the quantitative analysis. The other dams in the Brazos watershed between Morris Sheppard Dam and De Cordova Bend Dam do not exceed 20,000 ac.-ft and were not considered further.

Upstream Dams above Morris Sheppard Dam

Upstream from Morris Sheppard Dam, there are seven dams (Graham Dam, Hubbard Creek Dam, Millers Creek Dam, Fort Phantom Hill Dam, Lake Stamford, John T. Montford Dam, and White River Dam) with reservoirs greater than 50,000 ac.-ft. Each of the seven dams is located on a separate tributary or multiple tributaries that precludes domino-type failure with dams other than Morris

Sheppard Dam. Hubbard Creek Dam forms the reservoir with the greatest storage capacity (317,750 ac.-ft normal storage capacity and 720,000 ac.-ft maximum storage capacity), has the largest drainage area, and is located approximately 99 river miles upstream of Morris Sheppard Dam.

Only Graham Dam is located closer to Morris Sheppard Dam. However, even when considering the storage capacity of the reservoir formed by Eddleman Dam, which is connected to the reservoir formed by Graham Dam, the combined storage capacity is much less than the reservoir formed by Hubbard Creek Dam. Additionally, Hubbard Creek Dam has a greater dam height. Furthermore, the quantitative assessment failure effects are transposed downstream without attenuation. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis.

Only John T. Montford Dam has a dam height greater than Hubbard Creek Dam. However, John T. Montford Dam is approximately 351 river miles upstream from Morris Sheppard Dam, whereas Hubbard Creek Dam is 99 river miles upstream. Although the quantitative assessment does not consider attenuation, there would be significant attenuation over 351 river miles compared to 99 river miles if more rigorous methods were introduced. The Hubbard Creek Dam also has a greater drainage area of 1107 sq. mi, whereas the John T. Montford Dam drainage area is only 394 sq. mi The quantitative assessment includes the PMF flow for the local watershed, which is greater for the larger drainage area. The quantitative assessment does not attenuate the combined PMF and failure effects from the Hubbard Creek Dam. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis.

Hubbard Creek Dam is closer to Morris Sheppard Dam, has a greater dam height, has a larger drainage area, and has a greater storage capacity than Millers Creek Dam, Fort Phantom Hill Dam, Lake Stamford Dam, and White River Dam. Therefore, it would be more conservative to consider the added effects from Hubbard Creek Dam failure in the quantitative analysis. Considering existing conditions, the limiting dam failure permutation for additional quantitative analysis is the domino-type failure of Hubbard Creek Dam, Morris Sheppard Dam, and De Cordova Bend Dam.

Future Conditions

Future conditions have been considered based on the information provided in the 2011 Brazos G Regional Water Plan (Reference 2.4-276) and the Llano Estacado Regional Water Plan (Reference 2.4-277). There are nine alternatives in the Brazos G Regional Water Plan and available details are provided in Table 2.4.4-202. There are three alternatives in the Llano Estacado Regional Water Plan and available details are provided in Table 2.4.4-203. The locations of the potential sites for each alternative are shown on Figure 2.4.4-204. Although potential sites are identified in the regional water plans, not all alternative potential sites are considered proposed dams as discussed below.

The Brazos G Regional Water Plan identifies sites to assess the potential for development in the Brazos River watershed. Some of the potential sites have not been identified as recommended water management strategies and are not considered to be proposed reservoirs because there are no intentions or actions to develop the potential sites. There have been no efforts to perform design work, identify budgets, procure necessary property, or execute any type of construction activity for the South Bend Reservoir, the two Double Mountain Fork reservoir alternatives, the Lake Palo Pinto Off-Channel Reservoir, or the Throckmorton Reservoir. Therefore, these sites are not considered proposed reservoirs. Additionally, the two Double Mountain Fork reservoirs are not concurrent alternatives. The plan identifies either the east or west alternative as a potential site, but not both.

Proposed Dams

The Turkey Peak Reservoir is a recommended water management strategy and is considered a proposed reservoir. The Turkey Peak Reservoir (22,577 ac.-ft storage capacity) would be located approximately 3 river miles downstream from Lake Palo Pinto Dam. Turkey Peak Reservoir has been proposed to recover lost storage capacity of the reservoir formed by Lake Palo Pinto Dam due to sedimentation. A recent volume survey determined the reservoir storage capacity to be 63 percent of the normal capacity.

Turkey Peak Reservoir would have the same water surface elevation as the reservoir formed by Lake Palo Pinto Dam. Portions of the Lake Palo Pinto Dam would be removed to allow the two reservoirs to be connected at an upper elevation. Additionally, a pipe will connect the two reservoirs at a lower elevation. This configuration would reduce the failure effects of Lake Palo Pinto Dam compared to existing conditions because of the normal high tailwater on the downstream face of Lake Palo Pinto Dam. Although, the Turkey Peak Reservoir Dam would be higher than the Lake Palo Pinto Dam, the height would not be expected to exceed the height of Morris Sheppard Dam. Additionally, the combined storage capacity is much less than the storage capacity at Morris Sheppard Dam. Therefore, as previously discussed for the existing Lake Palo Pinto Dam, the failure effects from a combined Lake Palo Pinto Dam and Turkey Peak Reservoir Dam failure would not exceed the existing limiting dam failure permutation.

The Millers Creek augmentation is a recommended water management strategy and is considered a proposed alternative. The Millers Creek augmentation consists of a proposed diversion dam on Lake Creek and a proposed dam on Millers Creek approximately 4 river miles downstream of the existing Millers Creek Dam. Both structures are to be located upstream of Morris Sheppard Dam. The diversion dam is a low head structure only 8 ft high and anticipated to maintain a small storage capacity. There are no downstream structures between the diversion dam and Morris Sheppard Dam. Therefore, dam failure of the diversion dam would not exceed the existing limiting dam failure permutation that includes Hubbard Creek Dam.

The new Millers Creek Dam would have a water surface elevation just 18 ft below the existing Millers Creek Dam. Therefore, the new reservoir would back up to the existing dam, causing a normal high tailwater on the downstream face of the existing dam. This configuration would reduce the failure effects of the existing Millers Creek Dam compared to current conditions. The height of the new Millers Creek Dam would not be expected to exceed the height of Hubbard Creek Dam. Additionally, the combined storage capacity of the existing and new Millers Creek Dams is much less than the storage capacity at Hubbard Creek Dam. There are no downstream structures between the new Millers Creek Dam and Morris Sheppard Dam. Therefore, the failure effects from the combined existing and new Millers Creek Dam failures would not exceed the existing limiting dam failure permutation as previously determined.

The Cedar Ridge Reservoir is a recommended water management strategy and is considered a proposed reservoir. The Cedar Ridge Reservoir (227,127 ac.-ft storage capacity) would be located on the Clear Fork of the Brazos River approximately 172 river miles upstream from Morris Sheppard Dam. Fort Phantom Hill Dam (70,036 ac.-ft normal storage capacity and 127,000 ac.-ft maximum storage capacity) is located approximately 41 river miles upstream from the proposed Cedar Ridge Reservoir on a tributary of the Clear Fork of the Brazos River. Domino-type failure of Fort Phantom Hill Dam and Cedar Ridge Reservoir Dam would enhance the postulated dam failure effects at Morris Sheppard Dam.

Furthermore, Lake Stamford Dam (57,927 ac.-ft normal storage capacity and 150,000 ac.-ft maximum storage capacity) is located about 10 miles to the northwest of Cedar Ridge Reservoir on Paint Creek, a tributary of the Clear Fork of the Brazos River. Although it is not located upstream from Cedar Ridge Reservoir, Lake Stamford Dam is also located approximately 170 river miles upstream from Morris Sheppard Dam. Simultaneous failure of Lake Stamford Dam and Cedar Ridge Reservoir Dam would also enhance the postulated dam failure effects at Morris Sheppard Dam.

The three alternatives from the Llano Estacado Regional Water Plan are all proposed to be developed in series on the North Fork Double Mountain Fork of the Brazos River. Lake 7 (20,700 ac.-ft storage capacity) is proposed to be developed immediately upstream from McMillan Dam (4200 ac.-ft normal storage capacity and 8280 ac.-ft maximum storage capacity). Post Reservoir (56,000 ac.-ft storage capacity) is proposed to be developed approximately 41 river miles downstream from McMillan Dam. Diversion Reservoir (1000 ac.-ft storage capacity) is proposed to be developed approximately 21 river miles downstream of Post Reservoir and just upstream of the confluence with the South Fork Double Mountain Fork of the Brazos River.

The three proposed reservoirs in conjunction with the existing reservoir formed by McMillan Dam contain relatively small storage capacities compared to the reservoir formed by John T. Montford Dam (115,937 ac.-ft normal storage capacity and 354,500 ac.-ft maximum storage capacity) on the South Fork and Double Mountain Fork of the Brazos River. Considering domino-type failure of the three

proposed structures and the existing McMillan Dam, there would be some attenuation between each successive failure. Because John T. Montford Dam contains a much greater storage capacity and is considered as previously discussed, the three proposed structures have not been considered further.

Considering future conditions, the limiting dam failure permutation for additional quantitative analysis is the domino-type failure of Fort Phantom Hill Dam, Cedar Creek Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam along with the simultaneous failure of Lake Stamford Dam.

Pertinent Information for Upstream Dams

The considered upstream structures are described below. Reservoirs are assumed to be at their maximum historical water surface elevation or higher prior to the onset of the PMF. Outlet, gated spillway, and turbine discharges are assumed to be unavailable to accomodate PMF flows. The gates at Morris Sheppard Dam and DeCordova Bend Dam are assumed to be closed. Wind setup for each reservoir is added to the maximum water surface elevation determined from the PMF combined with effects of upstream dam failures and transposed to the dam without attenuation. Failure of downstream structures would reduce the effects of dam failure and are not considered to fail.

The elevations provided below are referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29), unless noted otherwise. The plant site grading plan is referenced to the North American Vertical Datum of 1988 (NAVD 88). Datum conversion is discussed in Subsection 2.4.4.3.

Hubbard Creek Dam is an earthfilled embankment 15,150 ft in length with a maximum height of 112 ft or elevation 1208.0 ft. The service spillway is a circular concrete drop inlet structure that is gate controlled. The crest elevation of the drop inlet is 1176.5 ft and the top of the gates is at elevation 1185.0 ft. All water that enters the drop inlet is discharged through the embankment and exits downstream via a 22 ft diameter conduit. The normal pool elevation is 1183.0 ft. The emergency spillway is an excavated broad crested weir located near the left end of the dam. The 2000 ft long weir is at elevation 1194.0 ft. Also, incorporated in the emergency spillway is a 4000 ft long fuse plug with a crest elevation of 1197.0 ft (Reference 2.4-278).

According to the USGS gauge 08086400 Water-Data Report 2009 (Reference 2.4-279), the maximum recorded elevation for the reservoir is 1190.22 ft.

Lake Stamford Dam is an earthfilled embankment 3600 ft in length with a maximum height of 78 ft or crest elevation 1436.8 ft. The service spillway is an excavated channel at the left end of the dam with an uncontrolled spillway crest 100 ft in length at elevation 1416.8 ft. The normal pool elevation is 1416.8 ft. The emergency spillway is a natural channel located at the right end of the embankment with a spillway crest elevation of 1425.8 ft (Reference 2.4-280).

According to the USGS gauge 08084500 Water-Data Report 2009 (Reference 2.4-281), the maximum recorded elevation for the reservoir is 1426.18 ft.

Fort Phantom Hill Dam is an earthfilled embankment 3740 ft in length with a maximum height of 84 ft. The spillway is a natural ground channel with an uncontrolled ogee crest 800 ft in length at elevation 1635.9 ft. The normal pool elevation is 1635.9 ft (Reference 2.4-282). Based on the USGS quadrangle for Hamby, TX (Reference 2.4-283) that encompasses Lake Fort Phantom Hill, there is a levee along the west side of the lake at elevation 1643 ft and approximately 6765 ft long.

According to the USGS gauge 08083500 Water-Data Report 2009 (Reference 2.4-284), the crest of the dam is 1650.0 ft and the maximum recorded elevation for the reservoir is 1639.50 ft.

According to the Brazos G Regional Water Plan (Reference 2.4-276), Cedar Ridge Reservoir will inundate approximately 6635 ac at the normal full pool elevation of 1489.0 ft. No other specific details for the proposed dam have been developed. Spillway details have not been developed and it is unknown how high above the full pool elevation the dam may be constructed. Therefore, it is assumed that the Cedar Ridge Reservoir Dam crest is at elevation 1510.0 ft, which is 21 ft above the normal full pool elevation. This is consistent with other dams in the region such as Lake Stamford Dam (20 ft above normal full pool elevation), Fort Phantom Hill Dam (14.1 ft above normal full pool elevation), and Hubbard Creek Dam (25 ft above normal full pool elevation). Based on the approximated location the crest length is estimated to be 4965 ft.

Morris Sheppard Dam is a concrete buttress dam with earthen dikes and has a maximum height of 189 ft or elevation 1024.0 ft. The service spillway is gate controlled with an ogee crest elevation of 987.0 ft and the top of gates elevation of 1000.0 ft. The dam impounds Possum Kingdom Lake at a normal pool elevation of 1000.0 ft (Reference 2.4-285). According to the Brazos River Authority Morris Sheppard Dam Breach Analysis Report (Reference 2.4-286), the total length of the concrete buttress section is 1640 ft. At the right abutment, the dam continues with a 1107 ft long earthen dike with a concrete core wall. In 1991 a 1400 ft long emergency spillway at elevation 1000.0 ft was completed at the south end of the concrete core wall. The top elevation of the concrete core wall is 1028.0 ft. Based on the Federal Energy Regulatory Commission Environmental Use and Inspection Report (Reference 2.4-287), the spillway length is 707 ft with nine 73.6 ft wide gates.

According to the USGS gauge 08088500 Water-Data Report 2008 (Reference 2.4-288), the maximum recorded elevation for the reservoir is 1003.60 ft and occurred prior to completion of the emergency spillway.

De Cordova Bend Dam is a concrete buttress dam with earth-filled sections and has a maximum height of 84 ft. The total length of the dam is 2200 ft. The spillway section is gate controlled with an ogee crest elevation of 658.0 ft. There are 16

tainter gates, each 36 ft wide and 35 ft high. Therefore, the top of gates elevation is 693.0 ft. The dam impounds Lake Granbury at a normal pool elevation of 693.0 ft (Reference 2.4-289). The top of the dam is elevation 706.5 ft (Reference 2.4-209). According to the NID database (Reference 2.4-222), the spillway section is 656 ft long.

According to the USGS gauge 08090900 Water-Data Report 2008 (Reference 2.4-290), the maximum recorded elevation for the reservoir is 693.60 ft.

Quantitative Assessment for Dam Failure Analysis

The coincident PMF flows are determined using the approach detailed in Appendix B of the NRC Regulatory Guide 1.59 (RG 1.59). Overtopping depth at each structure is determined using the standard broad crested weir flow equation.

$$Q = C \cdot L \cdot H^{1.5}$$

where

Q = flow (cfs)
C = weir flow coefficient (C = 2.6)
L = weir length (ft)
H = weir energy head (ft)

Wind Setup Analysis

Wind setup is determined using the mathematical expression provided in the U.S. Army Corps of Engineers EM 1110-2-1420 (Reference 2.4-236).

S = U2 * F / (1,400 * D)

where

S = wind setup (ft)
U = average wind velocity over fetch distance (mph)
F = fetch distance (mi)
D = average depth of water generally along the fetch line (ft)

Wind speed is estimated based on the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229). As shown on Figure 2.4.3-210, the wind speed for CPNPP is 50 mph. However, the Brazos River watershed extends to areas of 60 mph. A two-year annual extreme mile wind speed of 60 mph is estimated for all upstream reservoirs. This is conservative and bounding for the expected range of values for the region.

The fetch distance is estimated to be the longest straight line fetch for the reservoir surface area at the maximum water surface elevation. The average depth of water is determined from the hydraulic depth using U.S. Geological

Survey contours and supplemented with bathymetry maps from individual reservoir volumetric survey reports developed by the Texas Water Development Board.

Tailwater depth is determined for the overtopping flow at a downstream cross section using FlowMaster (Reference 2.4-241) and the Manning friction formula. A Manning coefficient of 0.025 is applied to the channel and overbank areas. Based on Chow (Reference 2.4-233), this is the minimum coefficient for main stream and flood plain areas. For the purpose of dam failure evaluation, it is conservative to use a lower coefficient because it results in a lower tailwater elevation. A lower tailwater elevation will maximize the water height component of the dam failure equation and the resulting dam failure flow or breach wave height. When it is determined that overtopping discharge is not independent of tailwater, the weir flow coefficient is reduced based on the guidance provided in the Federal Highway Administration Hydraulic Design Series Number 5 (Reference 2.4-223). A reduction of the weir flow coefficient is conservative and will increase the overtopping headwater elevation.

The resulting overtopping dam failure flows are based on the St. Venant mathematical expression provided in the U.S. Army Corps of Engineers EM 1110-2-1420. (Reference 2.4-239).

$$Q = \frac{8}{27} \cdot W_b \cdot g^{0.5} \cdot Y_0^{1.5}$$

where

Q = flow (cfs)
W_b = width of breach (ft)
g = gravity coefficient (32.2 ft/sec2)
Y_o = initial depth (ft)

The expression assumes a rectangular cross section and is applied to concrete structures. A modified version of the expression, accounting for side slopes of a breach, is used for embankment sections. The following modified mathematical expression is provided in U.S. Army Corps of Engineers HEC-HMS documentation. (Reference 2.4-238).

$$Q = 1.7 \cdot W_b \cdot h^{1.5} + 1.35 \cdot S \cdot h^{2.5}$$

where

Q = outflow through the breach (cfs)
W_b = width of breach (ft)
h = smaller of the quantities: head difference between the reservoir interior water surface elevation and the tail water surface elevation, or head difference between reservoir interior water surface elevation and the breach bottom invert

elevation (ft)
S = side slope of breach

Alternatively, a breach wave heigh is computed using the method described in ANSI/ANS-2.8-1992 (Reference 2.4-229).

h = 4 * (headwater - tailwater) / 9

where

h = breach wave height (ft)

Breach characteristics are estimated based on the guidance included in the U.S. Army Corps of Engineers RD-13 (Reference 2.4-240). Estimated breach flows or breach wave heights combined with additional spillway flows and overtopping flows are transposed to the next downstream structure without any attenuation. The transposed flow is combined with coincident PMF flow and a resulting overtopping depth and breach flow or breach wave height is then determined.

Hubbard Creek Dam

A coincident PMF of 600,000 cfs is estimated for the 1107 sq. mi drainage area of Hubbard Creek Dam. The antecedent reservoir elevation is assumed to be at the emergency spillway elevation of 1194.0 ft. This exceeds the maximum recorded water surface elevation. The emergency spillway and fuse plug overtopping elevation is determined to be 1207.4 ft, which does not exceed the dam crest elevation.

Because the service spillway consists of a drop inlet structure interior to the reservoir, it is assumed the full capacity of the service spillway, 30,000 cfs, contributes to downstream flooding in addition to the PMF flow. The tailwater elevation is determined to be 1128.7 ft using the combined flow of 630,000 cfs. The tailwater is well below the spillway elevation.

The wind setup fetch distance is determined to be 11.4 mi using the USGS 1210 ft contour as the basis for the overtopping elevation. The average depth is determined to be 30.0 ft. The wind setup is determined to be 1.0 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1208.4 ft.

The following overtopping failures of Hubbard Creek Dam are considered:

- Overtopping failure of the main embankment dam
- Overtopping failure of the embankment fuse plug

A breach width of three times the dam height and 1:1 side slopes are assumed for the main dam. The breach flow is 490,000 cfs, accounting for tailwater. Breach flow is added to the combined PMF and service spillway flow for a total of 1,120,000 cfs. Alternatively, the breach wave height is 35.5 ft, accounting for tailwater.

The bottom of the fuse plug is determined to be at an elevation of 1170 ft, which is above the tailwater elevation. Therefore, no tailwater effects are considered for the fuse plug failure. The entire 4000 foot long fuse plug is assumed for the breach width along with 1:1 side slopes. The resulting breach flow is 1,640,000 cfs, which is added to the combined PMF and service spillway flow for a total of 2,270,000 cfs. Alternatively, the breach wave height is 17.1 ft.

The potential Hubbard Creek Dam failure effects to be considered (transposed downstream without attenuation to Morris Sheppard Dam) are a breach flow of 2,270,000 cfs from the fuse plug or a breach wave height of 35.5 ft from the main dam.

<u>Lake Stamford Dam</u>

A coincident PMF of 350,000 cfs is estimated for the 360 sq. mi drainage area of Lake Stamford Dam. The antecedent reservoir elevation is assumed to be at the dam crest elevation of 1436.8 ft, whichexceeds the maximum recorded water surface elevation. It is assumed the service and emergency spillway capacities are not available to accommodate any portion of the PMF. The overtopping elevation is determined to be 1448.0 ft. The tailwater elevation is determined to be 1409.1 ft for the PMF flow. The tailwater is well below the dam crest elevation.

The wind setup fetch distance is determined to be 10.7 mi using the USGS 1450 ft contour as the basis for the overtopping elevation. The average depth is determined to be 27.7 ft.The wind setup is determined to be 1.0 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1449.0 ft.

Overtopping failure of Lake Stamford Dam is considered. A breach width of three times the dam height and 1:1 side slopes are assumed. Accounting for tailwater, the breach flow is 120,000 cfs. Breach flow is added to the PMF for a total of 470,000 cfs. Alternatively, the breach wave height is 17.8 ft, accounting for tailwater. The potential Lake Stamford Dam failure effects are to be considered for combination with the proposed Cedar Ridge Reservoir Dam failure effects and transposed downstream without attenuation to Morris Sheppard Dam.

<u>Fort Phantom Hill Dam</u>

A coincident PMF of 410,000 cfs is estimated for the 478 sq mi drainage area of Fort Phantom Hill Dam. The antecedent reservoir elevation is assumed to be at the levee crest elevation of 1643.0 ft. This exceeds the maximum recorded water surface elevation. It is assumed spillway capacity is not available to accommodate

any portion of the PMF. The overtopping elevation is determined to be 1651.1 ft. The tailwater elevation is determined to be 1576.9 ft for the PMF flow. The tailwater is well below the levee and dam crest elevations.

The wind setup fetch distance is determined to be 7.9 mi using midway between the USGS 1650 ft and 1660 ft contours as the basis for the overtopping elevation. The average depth is determined to be 24.0 ft. The wind setup is determined to be 0.9 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1652.0 ft.

Because the levee is not as high, only overtopping failure of Fort Phantom Hill Dam is considered. A breach width of three times the dam height and 1:1 side slopes are assumed. The breach flow is 350,000 cfs, accounting for tailwater. Breach flow is added to the PMF for a total of 760,000 cfs. Alternatively, the breach wave height is 33.4 ft, accounting for tailwater. The potential Fort Phantom Hill Dam failure effects are transposed downstream without attenuation to the proposed Cedar Ridge Reservoir Dam.

Cedar Ridge Reservoir Dam

A coincident PMF of 810,000 cfs is estimated for the 2748 sq. mi drainage area of the proposed Cedar Ridge Reservoir Dam. Because the upstream dam failure effects include the Fort Phantom Hill Dam PMF of 410,000 cfs, only 400,000 cfs is added to the upstream dam failure effects to represent the contribution from the proposed Cedar Ridge Reservoir PMF. The antecedent reservoir elevation is assumed to be at the dam crest elevation of 1510.0 ft.

The overtopping elevation is determined to be 1530.1 ft for the combined PMF and upstream dam failure effects flow of 1,160,000 cfs. The corresponding tailwater elevation is determined to be 1441.7 ft, which is well below the dam crest elevation.

Alternatively, the upstream dam failure breach wave height is added to the antecedent reservoir elevation to determine the corresponding flow. The flow is 2,500,000 cfs at an overtopping elevation of 1543.4 ft. The contributing portion of the proposed Cedar Ridge Reservoir coincident PMF is added for the combined PMF and upstream dam failure breach wave height of 2,900,000 cfs. The resulting overtopping elevation is determined to be 1547.0 ft. The corresponding tailwater elevation is determined to be 1471.3 ft, which is well below the dam crest elevation.

The wind setup fetch distance is determined to be 6.9 mi using the USGS 1550 ft contour as the basis for the overtopping elevation. The average depth is determined to be 68.2 ft. The wind setup is determined to be 0.3 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1530.4 ft for an overtopping flow of 1,160,000 cfs or 1547.3 ft for an overtopping flow of 2,900,000 cfs.

The following overtopping failure conditions of the proposed Cedar Ridge Reservoir Dam are considered:

- Overtopping flow of 1,160,000 cfs with a headwater elevation 1530.4 ft and a tailwater elevation 1441.7 ft
- Overtopping flow of 2,900,000 cfs with a headwater elevation 1547.3 ft and a tailwater elevation 1471.3 ft

A breach width of three times the dam height and 1:1 side slopes are assumed. Based on an overtopping flow of 1,160,000 cfs and accounting for tailwater, the breach flow is 710,000 cfs. Breach flow is added to the PMF and overtopping flow for a total of 1,870,000 cfs. Alternatively, the breach wave height is 39.5 ft, accounting for tailwater. Based on an overtopping flow of 2,900,000 cfs and accounting for tailwater, the breach flow is 560,000 cfs. Breach flow is added to the PMF and overtopping flow for a total of 3,460,000 cfs. Alternatively, the breach wave height is 33.8 ft, accounting for tailwater.

The potential Cedar Ridge Reservoir Dam failure effects to be considered (transposed downstream without attenuation to Morris Sheppard Dam) are a breach flow of 3,460,000 cfs or a breach wave height of 39.5 ft. When combined with the Lake Stamford Dam failure effects, the total upstream dam failure effects are 3,930,000 cfs or a wave height of 57.3 ft. The combined upstream dam failure effects exceed the potential failure effects from Hubbard Creek Dam. Therefore, the controlling dam failure scenario includes the domino-type failures Fort Phantom Hill Dam, proposed Cedar Ridge Reservoir Dam, Morris Sheppard Dam, and De Cordova Bend Dam. In addition Lake Stamford Dam is assumed to fail simultaneous with the Cedar Ridge Reservoir Dam.

<u>Morris Sheppard Dam</u>

For the 13,310 sq. mi contributing drainage area of Morris Sheppard Dam, the greater 16,113 sq. mi contributing drainage area of De Cordova Bend Dam is used to determine the coincident PMF of 1,450,000 cfs is estimated. Although, the maximum historical elevation was recorded prior to construction of the emergency spillway, it is assumed the antecedent reservoir elevation is the maximum historical elevation of 1003.6 ft. Assuming the spillway gates are closed and overtopped by the antecedent reservoir elevation, the combined emergency spillway and gate overtopping flow is 40,000 cfs.

The upstream dam failure effects are added to the coincident PMF and antecedent reservoir elevation flow for a total overtopping flow of 5,420,000 cfs. The overtopping elevation is determined to be 1075.7 ft. The corresponding tailwater elevation is determined to be 973.0 ft, which is well below the spillway crest and top of gates elevations.

Alternatively, the upstream dam failure breach wave height is added to the antecedent reservoir elevation and combined with the coincident PMF to

determine the corresponding flow. At an overtopping elevation of 1060.9 ft the flow is 3,670,000 cfs. The combined PMF and upstream dam failure breach wave height flow is 5,120,000 cfs. The resulting overtopping elevation is determined to be 1073.3 ft. The corresponding tailwater elevation is determined to be 970.3 ft, which is well below the spillway crest and top of gates elevations.

The wind setup fetch distance is determined to be 2.3 mi using the USGS 1080 ft contour as the basis for the overtopping elevation. The average depth is determined to be 120.5 ft. Thewind setup is determined to be 0.1 ft using a wind speed of 60 mph. Therefore, dam failure is evaluated using a headwater elevation of 1075.8 ft for an overtopping flow of 5,420,000 cfs or 1073.4 ft for an overtopping flow of 5,120,000 cfs.

The following overtopping failures of Morris Sheppard Dam are considered:

- Overtopping failure of the spillway section.
- Overtopping failure of the embankment section.
- Overtopping failure of the buttress section at the left abutment.
- Overtopping failure of the buttress section between the spillway and embankment sections.

The overtopping failures of the buttress sections are eliminated without calculation. The left abutment buttress section has a much shorter crest length than the spillway section. Therefore, failure of the spillway section would result in a greater breach flow. The buttress section between the spillway and embankment sections is approximately the same length as the spillway, but the section depth is about half that of the spillway section. Therefore, failure of the spillway section would result in a greater breach flow.

The following overtopping failure conditions of Morris Sheppard Dam are considered:

- Overtopping flow of 5,420,000 cfs with a headwater elevation 1075.8 ft and a tailwater elevation 973.0 ft.
- Overtopping flow of 5,120,000 cfs with a headwater elevation 1073.4 ft and a tailwater elevation 970.3 ft.

A breach width of the entire spillway section and vertical side slopes are assumed. Based on an overtopping flow of 5,420,000 cfs and accounting for tailwater, the breach flow is 1,240,000 cfs. Breach flow is added to the overtopping flow for a total of 6,660,000 cfs. Alternatively, the breach wave height is 45.7 ft, accounting for tailwater. Based on an overtopping flow of 5,120,000 cfs and accounting for tailwater, the breach flow is 1,250,000 cfs. Breach flow is added to the PMF and

overtopping flow for a total of 6,370,000 cfs. Alternatively, the breach wave height is 45.9 ft, accounting for tailwater.

The bottom of the embankment section is determined to be at an elevation of 990 ft. This is above the tailwater elevation. Therefore, no tailwater effects are considered for the embankment section failure. A breach width of three times the dam height and 1:1 side slopes are assumed. Based on an overtopping flow of 5,420,000 cfs the resulting breach flow is 230,000 cfs. Breach flow is added to the overtopping flow for a total of 5,650,000 cfs. Alternatively, the breach wave height is 38.2 ft. Based on an overtopping flow of 5,120,000 cfs the resulting breach flow is 220,000 cfs. Breach flow is added to the overtopping flow for a total of 5,340,000 cfs. Alternatively, the breach wave height is 37.1 ft.

The potential Morris Sheppard Dam failure effects, transposed downstream without attenuation to De Cordova Bend Dam, to be considered are a spillway section breach flow of 6,660,000 cfs or a breach wave height of 45.9 ft.

De Cordova Bend Dam

The Morris Sheppard Dam failure effects include the PMF for the Brazos River at De Cordova Bend Dam. Therefore, no additional flow is combined with the upstream failure effects. For the overtopping flow, the antecedent reservoir elevation is assumed to be at the dam crest elevation of 706.5 ft. Because of topography conditions around the reservoir, above elevation 700 ft. the reservoir is capable of spilling over low lying elevations along the south rim of the reservoir into the Brazos River well downstream from the dam. Based on the overtopping flow of 6,660,000 cfs and a reduced weir flow coefficient of 1.54, the headwater is determined to be 766.4 ft. The corresponding tailwater is determined to be 751.1 ft. Tailwater is determined for only the 4,670,000 cfs portion of total flow that overtops the dam and adjacent abutment areas. The remaining flow overtops the south rim of the reservoir.

Alternatively, for the breach wave height, it is assumed the antecedent reservoir elevation is the maximum historical elevation of 693.6 ft. The upstream dam failure breach wave height is added to the antecedent reservoir elevation to determine the corresponding flow. At an overtopping elevation of 739.5 ft the flow is 3,270,000 cfs. The corresponding tailwater elevation is determined to be 734.2 ft. Tailwater is determined for only the 2,750,000 cfs portion of total flow that overtops the dam and adjacent abutment areas. The remaining flow overtops the south rim of the reservoir. Although, the tailwater exceeds the dam crest elevation, it is determined that at the overtopping elevation the weir flow coefficient does not require reduction.

The wind setup fetch distance is determined to be 5.3 mi using the USGS 770 ft contour as the basis for the overtopping elevation. The average depth is determined to be 67.9 ft. Using a wind speed of 60 mph, the wind setup is determined to be 0.3 ft. Therefore, dam failure is evaluated using a headwater

elevation of 766.7 ft for a total overtopping flow of 6,660,000 cfs or 739.8 ft for a total overtopping flow of 3,270,000 cfs.

The following overtopping failures of DeCordova Bend Dam are considered:

- Overtopping failure of the spillway section.
- Overtopping failure of the embankment section.

The following overtopping failure conditions of De Cordova Bend Dam are considered:

- Overtopping flow of 6,660,000 cfs with a headwater elevation 766.7 ft and a tailwater elevation 751.1 ft.
- Overtopping flow of 3,270,000 cfs with a headwater elevation 739.8 ft and a tailwater elevation 734.2 ft.

A breach width of the entire spillway section and vertical side slopes are assumed. Based on an overtopping flow of 6,660,000 cfs and accounting for tailwater, the breach flow is 70,000 cfs. Breach flow is added to the overtopping flow for a total of 6,730,000 cfs. Alternatively, the breach wave height is 7.0 ft, accounting for tailwater. Based on an overtopping flow of 3,270,000 cfs and accounting for tailwater, the breach flow is 20,000 cfs. Breach flow is added to the overtopping flow for a total of 3,290,000 cfs. Alternatively, the breach wave height is 2.5 ft, accounting for tailwater.

A breach width of three times the dam height and 1:1 side slopes are assumed for the embankment section. Based on an overtopping flow of 6,660,000 cfs and accounting for tailwater, the breach flow is 30,000 cfs. Breach flow is added to the overtopping flow for a total of 6,690,000 cfs. Based on an overtopping flow of 3,270,000 cfs and accounting for tailwater, the breach flow is 10,000 cfs. Breach flow is added to the overtopping flow for a total of 3,280,000 cfs. Alternatively, because of the tailwater effects, the embankment section breach wave heights are identical to those determined for the spillway section.

The overtopping failure of the entire spillway section results in the greatest breach flow.Because of the tailwater effects, the breach wave height was added to the downstream tailwater elevation to determine a corresponding flow. However, the result did not exceed the breach flow. Considering the breach flow and overtopping flow, including overtopping flow spreading out beyond the abutments and spilling over the south rim of the reservoir, the total outflow is determined to be 6,730,000. This flow is transposed downstream without any attenuation to the confluence of the Paluxy River near its confluence with Squaw Creek to determine the relevant water surface elevation.

There are no safety-related facilities that could be affected by loss of water supply due to dam failure or water supply blockages due to sediment deposition or

erosion during dam failure induced flooding. See Subsection 2.4.11. Landslide potential is addressed in Subsection 2.4.9. There are no safety-related structures that could be affected by waterborne objects. There are no on-site water control or storage structures located above site grade that may induce flooding.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The methods identified are standard industry methods applied to artificially large floods. The approach described above is conservative and utilizes conservative coefficients resulting in a bounding estimate for dam failure considerations. Therefore, a full unsteady flow analysis to determine dam breach flows and resulting water surface elevations with greater certainty is determined to be unnecessary. Downstream reservoirs have no affect on the results of this analysis. Domino-type failures are included coincident with PMF flows and transposed downstream without any attenuation as discussed above. As discussed below the resulting dam failure flood wave has no effect at the site.

2.4.4.3 Water Level at Plant Site

The potential backwater effect from flooding on the Brazos River is examined based on the assumed hydrologic domino-type dam failures coincident with the PMF. As described above, the assumed hydrologic domino-type dam failures of Fort Phantom Hill Dam, the proposed Cedar Ridge Dam, the Lake Stamford Dam, the Morris Sheppard Dam, and the DeCordova Bend Dam coincident with the PMF, is transposed to the confluence of the Paluxy River and the Brazos River without any attenuation. Squaw Creek is a tributary of the Paluxy River. Utilizing HEC-RAS computer software (Reference 2.4-234), the stream course model described in Subsection 2.4.3.3 is used as a basis to determine the water surface elevation at the confluence.

The HEC-RAS stream course model is appended to include cross sections for the Brazos River. The selected cross sections are identified in Figure 2.4.4-202. As discussed in Subsection 2.4.4.3, a Manning's Roughness coefficient of 0.15 is also used for the Brazos River. The peak flows from the HEC-HMS model described in Subsection 2.4.3 for the Paluxy River and Squaw Creek were included as inputs for the Brazos River tributaries. The transposed 6,730,000 cfs from the dam failure scenario is included as the Brazos River input. The HEC-RAS model was run using steady state conditions to determine the water surface elevation at the confluence.

The resulting maximum water surface elevation at the confluence of Brazos River and Paluxy River cross section is 760.02 ft msl for the total transposed flow combined with the peak tributary flows as shown in Figure 2.4.4-203. The resulting water surface elevation is below the Squaw Creek Dam crest elevation of 796 ft. Therefore, coincident wind wave activity results would be equivalent to the wind wave activity for SCR (See Subsection 2.4.3.6). In the unlikely event of achieving the water surface elevation described above, possible headcutting on the downstream slope of Squaw Creek Dam could result in failure of the Squaw

Creek Dam. However, failure would lower the water surface elevation of SCR. In the event of Squaw Creek Dam failure the fetch length determined by the wind wave activity in Subsection 2.4.3.6 would not be increased.

Elevations are provided with reference to the National Geodetic Vertical Datum of 1929 (NGVD 29). The plant site elevation is referenced to the North American Vertical Datum of 1988 (NAVD 88). According to the National Geodetic Survey, the datum shift of NAVD 88 minus NGVD 29 is equal to between 0 and +0.66 in for the site. Therefore, it is conservative to account for a maximum conversion of +0.66 ft when comparing water surface elevations determined using NGVD 29 to elevations at the site in NAVD 88. Considering conversion, the confluence water surface elevation of 760.68 ft NAVD 88 is well below the CPNPP Units 3 and 4 safety-related structures elevation of 822 ft NAVD 88.

2.4.5 Probable Maximum Surge and Seiche Flooding

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.5.

According to the NRC Regulatory Guide 1.59, “Design Basis Floods for Nuclear Power Plants,” probable maximum surge and seiche flooding is considered based on a probable maximum hurricane (PMH), probable maximum windstorm (PMWS), or moving squall line. (Reference 2.4-229) The region of occurrence for a PMH is along U.S. coastline areas. For a PMWS, the region of occurrence is along coastline areas and large bodies of water such as the Great Lakes. A moving squall is considered for the Great Lakes region.

According to USACE EM 1110-2-1100 (Reference 2.4-235) guidelines, meteorological wind systems generated by thunderstorms and frontal squall lines can generate waves up to 16.4 ft high for inland waters. Additionally, mesoscale convective complex wind systems affecting inland waters are fetch-limited and based on wind speeds of up to about 66 fps or 45 mph. Similar wind speeds are used to determine the coincident wind-generated wave activity discussed in Subsection 2.4.3. The coincident wind wave activity, including wave setup, results in maximum runup of 16.9 ft. The maximum wind setup is estimated to be 0.08 ft. Therefore, the total water surface elevation increase due to wind wave activity is estimated to be 16.98 ft. The resulting PMF coincident with wind wave activity elevation is 810.64 ft msl.

Seismic seiches mainly depend on factors such as frequency and magnitude of the excitation, depth and geometry of the water body, and the sediment properties surrounding the water body (References 2.4-292 and 2.4-293). The risk of the occurrence of seismic seiches greater than about 5 ft in height at the SCR site is considered very low because a comprehensive study by Barberopoulou (References 2.4-293 and 2.4-294) found that Lake Union, Washington, a site with geometry, geology, and seismicity conditions that are much more favorable for seismic seiche development, indicated a maximum seismic seiche height of about 5 ft. Lake Union is therefore considered to be a conservative bounding case for SCR, and maximum seismic seiche heights at the SCR location are not expected to exceed those for Lake Union. The CPNPP Units 3 and 4 site finish grade elevation of 822 ft provides an approximately 28-ft margin over the maximum SCR water level during the PMF event, which is significantly larger than the expected maximum seismic seiche height of 5 ft.

According to the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229), the region of occurrence for a PMH shall be considered for U.S. coastline areas and areas within 100 to 200 miles bordering the Gulf of Mexico. CPNPP Units 3 and 4 are located approximately 275 mi inland from the Gulf of Mexico and outside the region of occurrence for a PMH. Therefore, a PMH was not considered. CPNPP Units 3 and 4 safety-related facilities are located at the plant grade level elevation of 822 ft msl. A surge due to a PMH event would not cause flooding at the site.

According to the guidance of ANSI/ANS-2.8-1992 (Reference 2.4-229), the region of occurrence for a PMWS should be considered for locations along the Pacific Coast and North Atlantic Coast of the U.S. and large bodies of water such as the Great Lakes. Likewise, the region of occurrence for a moving squall line should be considered for locations along Lake Michigan and the other Great Lakes. CPNPP Units 3 and 4 are located outside of the region of occurrence for a PMWS and a moving squall line. Therefore, a PMWS and a moving squall line have not been considered.

SCR does not connect directly with any of the water bodies considered for such meteorological events associated with surge and seiche flooding. Because of the inland location and elevation characteristics, CPNPP Units 3 and 4 safety-related facilities are not at risk from surge and seiche flooding. Resonance wave phenomena including oscillations of waves at natural periodicity, lake reflection, and harbor resonance are traditionally characteristics of harbors, estuaries, and large lakes and not associated with river settings. Any effects on SCR produced by similar phenomena would not affect CPNPP Units 3 and 4.

Seismic-induced waves are not plausible for the SCR. Subsection 2.5.3 indicates there are no capable faults, and there is no potential for non-tectonic fault rapture within the 25 mi radius of the CPNPP Units 3 and 4. Additionally, there is no potential for tectonic or non-tectonic deformation within the 5 mi radius of the CPNPP Units 3 and 4. The geologic and seismic characteristics for the CPNPP Units 3 and 4 are described in Section 2.5.

Details of the geology and characteristics of the subsurface materials of the SCR general area are discussed in Section 2.5. The subsurface materials that are relevant to the stability of the SCR shoreline slopes include residual soils and the Glen Rose Formation bedrock. Within the depth of interest for stability evaluation, the Glen Rose Formation, consists of interbedded limestone and shale. The thickness of the residual soils over the Glen Rose Formation bedrock may range from a few feet to a few tens of feet.

The available past aerial photographs and images of the site do not show any features suggesting the presence of past landslides or slope failures along the shoreline slopes of the SCR. The engineering evaluation of the SCR shoreline slopes were studied using available USGS topographic data for both pre- and post-construction, and 10 critical sections were selected to represent the overall condition of the SCR shoreline slopes. SCR topographic data and the locations of the 10 selected sections are shown on Figure 2.4.5-201. A summary of the geometry of the selected sections is provided in Table 2.4.5-201 and shown on Figure 2.4.5-202. The selection of the sample sections along the SCR shoreline slopes was based on the slope height, slope gradient, and location. In general, the selected sections have heights ranging between 80 ft and 150 ft, and gradients ranging between 30H:1V (~2˚) and 3H:1V (~18˚), indicating fairly flat slopes.

Because of the uniform geology of the site and relatively flat-lying stratigraphy of the Glen Rose Formation, the subsurface materials for the selected sections are assumed to be similar to those of the CPNPP Unit 3 and 4 site, as described in Subsection 2.5.5. The thickness of the residual soils, within the sections, was selected based on an El. 833 ft for the top of the Glen Rose Formation engineering Layer A, which is an average value obtained from all the field exploration data for the CPNPP Units 3 and 4 site. As a conservative subsurface model, all material strength properties were assumed based on lower-bound values similar to the CPNPP site as discussed in FSAR Subsection 2.5.5 and shown in Table 2.5.5-202. The selected SCR shoreline slopes were analyzed using the conventional two dimensional limit-equilibrium for both the static and seismic loading conditions. A PGA value of 0.10g was used for both the horizontal and vertical components of seismic loading. Both the positive and negative directions were utilized for the vertical component, and the lowest resulting factor of safety was selected as the controlling condition. The slope stability analyses of the SCR shoreline slopes indicate acceptable static long-term and pseudo-static factors of safety, with values greater than 1.5 and 1.1, respectively, as summarized in Table 2.4.5-202. In order to consider the potential effect of the SCR water level fluctuations (from its maximum El. 783 ft at the spillway level to the minimum level of El. 770 ft), the sections were conservatively modeled as a rapid drawdown condition and stability analyses were performed assuming the very conservative condition that the SCR water level is instantaneously lowered from its highest level of El. 783 ft to the lowest level of El. 770 ft. Results of the slope stability analyses for the conservative condition of rapid drawdown, as shown on Table 2.4.5-202, are all acceptable.

In order to investigate the potential for variability of the subsurface material layering, an additional extremely conservative, worst case scenario model was also considered with the assumption that all subsurface materials consist of soil materials with lower-bound drained strength properties (C=200 psf and φ=25˚). This model is overly conservative and is not considered to be realistic, but it was considered as a parameter study to check the sensitivity of potential variation of the subsurface material layering. As shown in Table 2.4.5-203, the results of stability analyses using this worst case scenario model also indicate acceptable factors of safety.

The stability analyses described above demonstrate acceptable factors of safety for the cases of static, seismic, and rapid drawdown conditions for the SCR shoreline slopes and that no landslides or instability is expected. Therefore, landslide-induced seiches or waves are not considered plausible for the SCR.

2.4.6 Probable Maximum Tsunami Hazards

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.6.

Tsunami risk in the Gulf Coast region, primarily the Caribbean, has been studied to some degree, but no specific hazard maps have been developed for the Gulf Coast at this time. The USACE has developed a general tsunami risk map (Reference 2.4-242), as shown in Figure 2.4.6-201. The Gulf Coast is located in Zone 1, which corresponds to a wave height of 5 ft.

According to the National Oceanic and Atmospheric Administration's tsunami database (Reference 2.4-243), the maximum recorded tsunami wave height along the Gulf Coast or East Coast is about 20 ft. This height was recorded at Daytona Beach, Florida, on July 3, 1992. The database notes that the wave was probably meteorologically induced.

According to a recent USGS study (Reference 2.4-244), very little is known about a landslide-generated tsunami threat from the Mexican coast. Tsunamis generated by earthquakes do not appear to impact the Gulf of Mexico coast. CPNPP Units 3 and 4 are located approximately 275 mi inland from the Gulf Coast. CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. Because of their inland location and elevation, CPNPP Units 3 and 4 safety-related facilities would not be at risk from tsunami flooding.

Landslide-induced waves are not plausible for SCR. As discussed in Subsection 2.5.5, the slope stability analysis indicates stable permanent slopes, and therefore hill slope failure-induced waves are not plausible for SCR.

Seismic-induced waves are not plausible for SCR. Subsection 2.5.3 states there are no capable faults and there is no potential for non-tectonic fault rapture within the 25 mi radius of the CPNPP Units 3 and 4. Additionally, there is no potential for tectonic or non-tectonic deformation within the 5 mi radius of the CPNPP Units 3 and 4. The geologic and seismic characteristics for the CPNPP Units 3 and 4 are described in Section 2.5.

2.4.7 Ice Effects

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.7.

According to the EPA STOrage and RETrieval (STORET) database, two gaging stations located on the SCR and its tributaries recorded water temperatures for different periods between 1973 and 1985. The lowest recorded water temperatures range from 41.9°F to 50°F. The lowest recordings, 41.9°F, occurred on February 10, 1982 at station 11555, Squaw Creek and State Highway 144 (SH 144), Northeast of Glen Rose. (Reference 2.4-245)

Gaging station 11856 is located on Brazos River and gaging station 11976 is located on Paluxy River. The gaging station 11856 on Brazos River at U.S. Highway 67 (US 67) recorded water temperatures from 1968 to 1998. The lowest recorded water temperature at this station was 39.02°F. (Reference 2.4-245) The gaging station 11976 on Paluxy River in City Park recorded water temperatures from 1973 to 1996. The lowest recorded water temperature at this station was 39.2°F. (Reference 2.4-245) This data suggests that Squaw Creek water temperatures generally remain above the freezing point. The recordings are summarized in Table 2.4.7-201.

According to the USACE, ice jams occur in 36 states, primarily in the northern tier of the United States. (Reference 2.4-246) (Figure 2.4.7-201) Texas is not included in this coverage. USACE Cold Regions Research and Engineering Laboratory historical ice jam database (Reference 2.4-247) indicates no ice jams for Squaw Creek. However, the USACE ice jam database reports that Brazos River was obstructed by rough ice at Rainbow near Glen Rose, Texas, on January 22-23 and January 25-28, 1940, with flood stage of 20 ft. (Reference 2.4-247)

CPNPP Units 3 and 4 safety-related facilities are located at elevation 822 ft msl. The SCR spillway elevation is 775 ft msl (Reference 2.4-214). The maximum water surface elevation during a probable maximum flood event and coincident wind waves is at 810.64 ft msl, which is more than 11 ft below the CPNPP Units 3 and 4 safety-related facilities. The possibility of inundating CPNPP Units 3 and 4 safety-related facilities due to an ice jam is remote.

Meteorological records from the Southern Regional Climate Center (SRCC) were examined for areas in the vicinity of CPNPP Units 3 and 4. Records indicate that December and January have the coldest temperatures. For the available period of record from 1971 to 2000, the climate station at Dallas/Fort Worth has a recorded monthly average minimum temperature of 34°F, occurring in January. (Reference 2.4-248)

According to the USACE, frazil ice forms in supercooled turbulent water in rivers and lakes. (Reference 2.4-246) Anchor ice is defined as frazil ice attached to the river bottom, irrespective of the nature of its formation. The potential for freezing (i.e., frazil or anchor ice) and subsequent ice jams on the Squaw Creek and Brazos River is remote. Additionally, sustained periods of subfreezing water

temperatures are not characteristic of the region. The climate and operation of SCR prevent any significant icing on the Squaw Creek. There are no safety related facilities that could be affected by ice induced low flow.

According to U.S. Army Corps of Engineers methods (Reference 2.4-271), the maximum potential ice thickness is a function of accumulated freezing-degree days (AFDD) and the thermal expansion coefficient α. The AFDD parameter was selected using USACE Cold Regions Research and Engineering Laboratory data (Reference 2.4-272). The site is located in a region with AFDDs in the range of 1 to 100 °F days. The maximum of 100 °F days for the region was selected as the AFDD for CPNPP Units 3 and 4. The α coefficient for an "average lake with snow" is 0.50 - 0.70 (Reference 2.4-271). Idealizing SCR or Lake Granbury as an average lake with snow, the maximum of 0.7 was selected as the α coefficient. This is conservative and bounding for the expected range of AFDD and α values for the site. The resulting maximum potential ice thickness is 7 in, which is considered icing under extreme conditions. There are no safety-related facilities located on SCR. Although the intake structure is located on Lake Granbury, that structure is not safety-related. Therefore, ice sheet formation has no effect on the ultimate heat sink (UHS) or Essential Service Water System operation. There are no safety-related facilities that could be affected by ice-induced low flow at CPNPP Units 3 and 4. The freezing protection for the essential service water system (ESWS) four wet mechanical cooling towers is described in Subsection 9.2.1.3. The freezing protection for the ESW Pump House Ventilation System is described in Subsection 9.4.5.2.6.

2.4.8 Cooling Water Canals and Reservoirs

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.8.

There are no current or proposed safety-related cooling water canals or reservoirs required for CPNPP Units 3 and 4. The ultimate heat sink (UHS) is part of the essential service water system (ESWS). Each unit's ESWS consists of four wet mechanical draft cooling towers, each providing 50 percent cooling capacity. Each cooling tower consists of an ESW pump and basin transfer pump and is located over a basin. Each basin is designed to hold 33-1/3 percent of the cooling water inventory to allow safe shutdown up to 30 days after an accident without makeup. The above data indicates that the UHS does not rely on cooling water canals or reservoirs and is not dependent on a stream, river, estuary, lake, or ocean. Therefore, no warning of impending low flow from the lake water makeup system is required. Low lake water conditions would not affect the ability of the emergency cooling water systems and the UHS to provide the required cooling for emergency conditions. The UHS would not be affected by low water conditions. CPNPP Units 3 and 4 and UHS are capable of withstanding flooding events as described in Subsections 2.4.2 through Subsection 2.4.7.

2.4.9 Channel Diversions

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.9.

The Squaw Creek watershed does not contain any significant structures upstream of the Squaw Creek Dam. The Brazos River contains several tributary and instream dams, including DeCordova Bend Dam impounding Lake Granbury. Lake Granbury is the major source of normal makeup cooling water. Lake Granbury is not used as a safety-related water supply source.

The UHS is part of the ESWS. Each unit's ESWS consists of four wet mechanical draft cooling towers, each providing 50 percent cooling capacity. Each cooling tower consists of an ESW pump and basin transfer pump and is located over a basin. Each basin is designed to hold 33-1/3 percent of the cooling water inventory to allow safe shutdown up to 30 days after an accident without makeup. Therefore, channel diversion can not adversely affect CPNPP Units 3 and 4 safety-related structures or systems. Additional details are provided in Subsection 2.4.11. The potential for ice-induced diversion and flooding is discussed in Subsection 2.4.7. Geologic and seismic characteristics of the region are discussed in Section 2.5.

There is no evidence suggesting there have been significant historical diversions or realignments of Squaw Creek or the Brazos River. The topography does not suggest potential diversions. The streams and rivers in the region are characterized by traditional shaped valleys with no steep, unstable side slopes that could contribute to landslide cutoffs or diversions. There is no evidence of ice-induced channel diversion.

As identified in Subsection 2.5.1.2.5.4, there is no evidence of active karst conditions and related subsidence within the CPNPP site or in the surrounding area. Furthermore, Subsection 2.5.1.2.5.5 identifies that withdrawal of groundwater from aquifers beneath the site does not pose a risk of subsidence at the current withdrawal rates. Therefore, channel diversion due to subsidence is not expected.

Channel diversion due to geothermal activity was also investigated and is not expected. The greatest potential for geothermal energy exists in areas of above average heat flow, generally the result of recent volcanic activity or active tectonics. East of the Rocky Mountains is characterized by average heat flow (Reference 2.4-249). The area is also relatively tectonically stable and has experienced no volcanic activity recent enough to produce heat from crystallization (Reference 2.4-250). No thermal anomalies east of the Rocky Mountains are attributed to young-to-contemporary volcanic or other igneous activity (Reference 2.4-251).

2.4.10 Flooding Protection Requirements

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.10.

CPNPP Units 3 and 4 safety-related facilities are not exposed to flooding from all events identified in Subsection 2.4.2. The critical flooding event is identified in Subsection 2.4.2 and discussed in detail in Subsection 2.4.3. The maximum flood level is a result of the probable maximum precipitation on the Squaw Creek watershed and includes the effects of coincident wind wave activity. Based on the design information provided in the referenced subsections, flood protection measures and emergency procedures to address flood protection are not required.

2.4.11 Low Water Considerations

CP COL 2.4(1) Add the following after the second paragraph of DCD Subsection 2.4.11.

2.4.11.1 Low Flow in Rivers and Streams

Low flow conditions of the Brazos River are a function of natural flow in the rivers and streams, available storage capacity of upstream reservoirs, and regulated discharge flow from upstream dams. There are no significant upstream dams in the Squaw Creek watershed. Therefore, low flow conditions of Squaw Creek above the CPNPP Units 3 and 4 are a function of natural flow.

Dam failure could affect normal operation during low-flow conditions. However, there are no safety-related facilities that could be affected by low-flow or drought conditions, since the UHS does not rely on the rivers and streams as a source of water. Adequate non-safety related water supply during drought is addressed in Subsection 2.4.11.5.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

There are no safety-related facilities that could be affected by low water. The site is not at risk to low water resulting from surge, seiche, or tsunami effects, because of the inland location and characteristics of the site. See Subsections 2.4.5 and 2.4.6 for additional details.

Although rough ice had previously been recorded on the Brazos River at Rainbow near Glen Rose, Texas, It is unlikely that an ice jam would occur because water temperatures at the site remain above freezing. Therefore, low flow because of, or exaggerated by, ice effects is not expected to occur at the site. See Subsection 2.4.7 for additional details.

2.4.11.3 Historical Low Water

According to a historical report from 1958 the worst droughts to impact Texas were in:

1) 1954–1956
2) 1916–1918
3) 1909–1912
4) 1901
5) 1953
6) 1933–1934
7) 1950–1952 (Reference 2.4-252)

Recent reports indicate significant drought years in 1996, 1998, and 1999 (Reference 2.4-253). The most recent drought years occurred in 2005 and 2006.

By the end of 2006 the state's major reservoirs stored approximately 71 percent of the total conservation storage capacity (Reference 2.4-254).

The USGS gage (08091000) on the Brazos River located near Glen Rose, Texas downstream of DeCordova Bend Dam has a period of record from 1923 to the present. Prior to the completion of Morris Sheppard Dam in 1941, the minimum historical flow was 0 cfs on several occasions in 1924, 1931, 1934, 1936, and 1939. In 1934 the gage recorded 0 cfs flow continuously for 78 days from June to September. After completion of Morris Sheppard Dam, but prior to completion of DeCordova Bend Dam in 1969, the minimum recorded historical flow was 0.1 cfs for two days in October 1952. After completion of DeCordova Bend Dam, the minimum recorded historical flow was 0.17 cfs on July 14, 1984. (Reference 2.4-220) The annual minimum daily flows for the period of record are presented in Table 2.4.11-201.

The USGS gage (08090800) on the Brazos River located near Dennis, Texas between Morris Sheppard Dam and DeCordova Bend Dam has a period of record from 1969 to the present. The minimum recorded historical flow was 1.2 cfs on August 2, 1978. (Reference 2.4-255) The annual minimum daily flows for the period of record are presented in Table 2.4.11-202.

The USGS water data report 2006 for gage 08090900 indicates that the minimum recorded historical storage of 97,600 ac-ft occurred on August 9, 1978. The corresponding elevation was reported to be 685.28 ft in the USGS water data report 2006 for gage 08090900. (Reference 2.4-255) However, the gage data show records of storage and elevation only from 1987 to present. The gage data also indicates that the minimum recorded historical elevation for the gage during this period was 689.14 ft on April 28, 1990. (Reference 2.4-220) Therefore, 685.28 ft msl is considered the minimum recorded historical elevation for Lake Granbury.

Prior to completion of the Squaw Creek Dam in 1977, the minimum historical flow in Squaw Creek was 0.02 cfs for two days in August 1974. (Reference 2.4-255) The USGS gage (08091750) on Squaw Creek located near Glen Rose, Texas downstream of Squaw Creek Dam has a period of record from 1977 to 2006. The minimum recorded historical flow was 0.38 cfs on March 16, 2005. (Reference 2.4-220) The annual minimum daily flows for the period of record are presented in Table 2.4.11-203.

The USGS water data report 2006 for gage 08091730 indicates that the minimum historical elevation of SCR, impounded by Squaw Creek Dam, was 771.98 ft on September 16, 1992. (Reference 2.4-255) However, the USGS gage (08091730) shows records only from 2000 to present. The gage data indicate that the minimum recorded historical elevation during this period was 772.96 ft for two days in April, 2005. (Reference 2.4-220) Therefore, 771.98 ft msl is considered to be the minimum recorded historical elevation for SCR.

The USACE historical database of ice jams was reviewed for the region. See Subsection 2.4.7 for additional discussion. Due to the climate in the region, ice effects are not a concern for low water considerations.

2.4.11.4 Future Controls

According to the FSAR for Comanche Peak Steam Electric Station Units 1 and 2, an initial study by the Brazos River Authority identified three possible sites between Possum Kingdom Reservoir and Lake Granbury for potential control structures. Additionally, there is a possible site between DeCordova Bend Dam and Whitney Dam for a control structure. Issuance by the Texas Water Rights Commission of the permit to build and operate SCR precludes any significant development and control upstream in the Squaw Creek watershed. (Reference 2.4-214) Although the development of future controls on the Brazos River is possible, there are no safety-related facilities that could be affected.

2.4.11.5 Plant Requirements

Makeup water is supplied by the intake as described in Subsection 2.4.1.2.3.2. The intake structure includes necessary intake screens, pumps, etc. to convey the makeup water to the cooling water system flow. Intake screen locations consider the Lake Granbury minimum level. There are no safety-related plant requirements provided by Lake Granbury.

The maximum expected Lake Granbury intake flow rate is approximately 65,400 gpm for the CPNPP Units 3 and 4. The maximum expected Lake Granbury intake flow includes a circulating water system (CWS) Cooling Tower makeup flow rate of 31,200 gpm per unit for Units 3 and 4, an ESWS Cooling Tower makeup flow rate of 274 gpm per unit for Units 3 and 4 and miscellaneous plant use such as make up water flow to raw water storage tanks. The makeup flows to both CWS and ESWS Cooling Towers are provided periodically based on the basin level. The flows are normally controlled with basin water levels by on/off operation of CWS Cooling Tower makeup water pumps or ESWS Cooling Tower basin makeup control valves. These controls are described in in FSAR Subsections 9.2.5 and Section 10.4.5. Water use and annual mean flow are discussed in Subsection 2.4.1.2. Although the Texas Water Code requires a permit for water use, there are no specific limitations set by state regulations. Water use from the Brazos River and Lake Granbury is administered by the Brazos River Authority.

Low-flow frequency analysis was performed in accordance with USGS Bulletin 17B using the Log-Pearson Type III distribution method. The USGS gage (08090800) on the Brazos River located near Dennis, Texas between Morris Sheppard Dam and De Cordova Bend Dam was used to analyze the current regulated conditions of the Brazos River at the intake. Table 2.4.11-204 provides a summary of low flow frequencies for selected durations and return periods.

The 30-day 100-yr drought flow rate for Brazos River near Dennis, TX is estimated to be 9.7 cfs. A 100-yr return period is defined as a 1 percent chance the event will occur during any one year. Therefore, the 30-day 100-yr drought flow rate has a 1 percent chance each year that the flow rate or less will occur for at least 30 consecutive days. The 30-day 100-yr drought flow rate is less than the maximum expected Lake Granbury intake flow rate of 146 cfs. Therefore, adequate water supply during 100-yr drought conditions is provided by the storage volume of Lake Granbury.

The worst historical drought for the Texas region occurred from 1954 to 1956 (Reference 2.4-252). The USGS gage (08091000) has a period of record from 1923 to the present. The USGS gage (08091000) on the Brazos River located near Glen Rose, Texas downstream of De Cordova Bend Dam was used to assess the historical drought. During the period from 1954 to 1956, the gage recorded flows less than the maximum expected Lake Granbury intake flow rate of 146 cfs for a total of 337 days. The longest continuous period of flow lower than the expected Lake Granbury intake flow rate of 146 cfs was 51 consecutive days.

The normal storage capacity of Lake Granbury is 136,823 ac-ft. The minimum storage capacity since completion in 1969 was 97,600 ac-ft on August 9, 1978. (Reference 2.4-255) A concentration of 51 days would require approximately 14,756 ac-ft of storage water from Lake Granbury to operate CPNPP Units 3 and 4 continuously at maximum power conditions. Of these amounts an estimated 5877 ac-ft of storage would be returned at a steady rate to Lake Granbury. Thus the net volume of water taken from Lake Granbury during the low flow would approximately be 8879 ac-ft.

The CWS for the station is a closed-loop type system coupled with mechanical draft, wet cooling towers. The circulating water system flow rates for each unit are identified in Subsection 10.4.5. Emergency cooling is discussed in Subsection 2.4.11.6.

2.4.11.6 Heat Sink Dependability Requirements

The UHS is part of the ESWS. Each unit's ESWS consists of four wet mechanical draft cooling towers, each providing 50 percent cooling capacity. Cooling towers consist of ESW pumps and basin transfer pumps and is located over a basin. Each basin is designed to hold 33-1/3 percent of the cooling water inventory to allow safe shutdown up to 30 days after an accident without makeup, in accordance with Regulatory Guide 1.27. Therefore, no warning of impending low flow from the lake water makeup system is required. Low lake water conditions would not affect the ability of the emergency cooling water systems and the UHS to provide the required cooling for emergency conditions. There are no other uses of water drawn from the UHS.

The UHS design-bases are discussed in Subsection 9.2.5. Site-related events and natural phenomena would not affect the UHS. As described in Subsections

2.4.2 through 2.4.7 and 2.4.9, the station and UHS are capable of withstanding the detailed phenomena. Seismic design is addressed in Section 3.8.4.

2.4.12 Groundwater

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.12.

This subsection provides a summary of the groundwater data collected for the CPNPP Combined Operating License (COL) application.

2.4.12.1 Description and On-Site Use

This subsection describes regional and local aquifers, formations, sources and sinks, types of groundwater use, wells, pumps, storage facilities, and flow requirements for CPNPP. Groundwater is not used as an operational or safety-related source of water for CPNPP Units 3 and 4, and Luminant has implemented a conservation plan for future groundwater withdrawals at the CPNPP site. During construction of CPNPP Units 3 and 4, and during operation of CPNPP Units 1 through 4, potable water is to be supplied by the Somervell County Water District's water supply system. Water for temporary fire protection, concrete batching, and other construction uses is expected to be supplied by the Somervell County Water District.

2.4.12.1.1 Regional Aquifers, Formations, Sources, and Sinks

The CPNPP site lies within the Brazos River Basin of the Comanche Plateau subdivision of the Central Texas Section of the Great Plains Physiographic Province. The relationship of the site to these features and to other physiographic provinces in the region is shown on Figure 2.4.12-201. To the north is the Central Lowland Physiographic Province; to the east is the Coastal Plain Physiographic Province; and to the south is the Great Plains Province. The boundary separating the Great Plains Province from the Coastal Plain Province coincides with the contact of the upper and lower Cretaceous formations on which the CPNPP site is located.

Maximum relief in the site area is approximately 220 feet (ft), with elevations ranging from 640 ft to 860 ft above sea level with slopes that are typically steep, ranging from 15 to 30 degrees or more, and generally exhibits a stair-stepped appearance. Rock outcrops of limestone and claystone comprise approximately 40 to 60 percent of these slopes. The remaining areas, including the higher flat-topped plateau remnants, are mantled by a thin cover of soil, which at the surface generally consists of silt and sand (Reference 2.4-201).

Portions of six major and nine minor aquifers extend into the Brazos Region G Area (Reference 2.4-208). Brazos Region G is a 37-county planning area, which extends generally along the Brazos River from Kent, Stonewall, and Knox counties in the northwest to Washington and Lee counties in the southeast. The CPNPP site is located on outcrops of the Trinity Group aquifer, which occurs mostly in Callahan, Eastland, Erath, Hood, Somervell, Comanche, Hamilton, Coryell, and Lampasas counties. The confined aquifer area is mostly in Johnson,

Hill, Bosque, McLennan, Coryell, Bell, and Williamson counties (Figure 2.4.12-202).

Most of the groundwater in the site region occurs in bedrock. Some groundwater does exist in the shallow floodplain alluvium along stream valleys but is not withdrawn for use. The Trinity Group aquifer is a major aquifer that occurs in a north-south-trending band as shown on Figure 2.4.12-202 from the Texas Water Development Board (TWDB). The Trinity aquifer, composed of Cretaceous-aged Trinity Group Formations, is characterized as a major aquifer by the TWDB. The aquifer supplies drinking water to numerous communities, homes, and farms in Central Texas, and irrigation water to many farms, especially in Comanche and Erath counties.

The Trinity aquifer is composed of the Paluxy, Glen Rose, and Travis Peak formations. In the vicinity of the CPNPP site, and north, the Travis Peak Formation is known as the Twin Mountains Formation. South of the CPNPP site, the formation retains the Travis Peak name. The three formations are regional in extent; their outcrops form a strip of land tens of miles wide that extends south from central Oklahoma, strikes westward in Central Texas and extends into Mexico. In the site region, these formations dip gently eastward. Up dip where the Glen Rose thins or is missing, the Paluxy and Travis Peak formations coalesce to form the Antlers Formation. The uppermost water-bearing zone is the Paluxy Formation. The lower water-bearing zone consists of the Travis Peak Formation and is divided into the Hensell and Hosston members in much of the eastern part of Brazos Region G Area (Reference 2.4-208).

The Twin Mountains and Paluxy Formations are principally sandstone, but also have shale, limestone, claystone, and siltstone inclusions. Limestone is the dominant rock type in the Glen Rose Formation, but the stratum also contains significant quantities of shale, siltstone, and claystone. In these formations, groundwater percolates slowly along bedrock joints and fractures, and through interstices in the rock fabric.

The Twin Mountains Formation is the only moderately productive bedrock zone in the site vicinity, though the Paluxy Formation has nominal pumpage near the site. The Glen Rose Formation yields very little water in the site area and is usually less productive than the others. At distances of 20 to 50 mi, down-dip from the outcrop, the groundwater becomes saline, and the formations lose their importance as sources of fresh water. The three water-bearing formations are discussed individually below in ascending order:

Twin Mountains Formation

The principal origins of groundwater in the Twin Mountains Formation are rainfall and streamflow occurring in the outcrop area. Down-dip from the outcrop, groundwater in the Twin Mountains Formation is confined by fine-grained materials of the overlying Glen Rose Formation. Hydrostatic pressure in the Twin

Mountains is great enough to create static water levels that rise above the formation and, sometimes, to cause flowing wells (Reference 2.4-214).

Groundwater is discharged in the outcrop area by evapotranspiration, localized springs, and seepage into drainage channels incised below the water table. Down-dip from the outcrop area where the formation is confined, the natural discharge is limited to a small upward movement into overlying formations.

Glen Rose Formation

The principal origins of groundwater in the Glen Rose Formation are rainfall in the outcrop area, and minor seepage from both the overlying Paluxy Formation and underlying Twin Mountains Formation (Reference 2.4-214). The Glen Rose Formation outcrop area is shown on Figure 2.4.12-203.

In its outcrop area, the Glen Rose Formation discharges water naturally through springs and seeps. In confined portions of the formation, there is little transfer of water into overlying or underlying formations when differential pressures occur (Reference 2.4-214).

Paluxy Formation

Groundwater discharges from the Paluxy Formation as springs and seeps in some outcrop areas. Where the Paluxy Formation is confined, there is limited water movement into overlying confining units when those units are at a lower hydraulic head (Reference 2.4-214).

Recharge to the Paluxy Formation occurs in the outcrop areas from infiltration of rainfall and seepage from streams. It also receives water from water-bearing units under greater hydraulic heads, which adjoin the Paluxy Formation. South of the CPNPP site, the formation is confined by overlying fine-grained strata (Reference 2.4-214).

2.4.12.1.2 Local Aquifers, Formations, Sources, and Sinks

Locally, CPNPP and SCR are situated on the Glen Rose Formation outcrop, which, in turn, is underlain by the Twin Mountains Formation. The Paluxy Formation is absent at the CPNPP location and within the limits of SCR.

CPNPP Units 3 and 4 are to be constructed on the Glen Rose Formation. The Glen Rose limestones are essentially impermeable due to slight amounts of argillaceous impurities present. These limestones are resistant to solution effects; open voids, caverns, joints, collapse features, and frequent fractures. Groundwater, therefore, moves very slowly into and through the formation; entrance is afforded principally through existing joints and fractures. Occasional isolated sand lenses also contain groundwater (Reference 2.4-214).

Northwest of the CPNPP site, where the Glen Rose Formation is covered by outliers of the Paluxy, a few domestic water wells are completed in the Glen Rose Formation as well as south of the CPNPP site. No water wells have been completed in the Glen Rose Formation at the CPNPP site. The nearest water wells identified as completed in the Glen Rose Formation are located 4 mi south of the CPNPP site as shown on Figure 2.4.12-204. These wells produce potable water and are reliable during droughts, generally due to the slow release of groundwater to the Glen Rose Formation from the overlying Paluxy Formation. Elsewhere, wells completed in the Glen Rose are often unreliable during droughts (Reference 2.4-214).

The Glen Rose Formation ranges from 217-to-271 ft thick. The mean elevation for the top of the Twin Mountains Formation is approximately 592 ft mean sea level (msl). Based on a plant grade elevation of 822 ft msl, the thickness of the Glen Rose Formation under CPNPP Units 3 and 4 is approximately 230 ft. The Glen Rose Formation below the CPNPP site is considered impermeable and confines the groundwater in the underlying Twin Mountains Formation, which is more than 220 ft thick in the vicinity of the CPNPP site.

2.4.12.2 Sources

Makeup water to the ultimate heat sink (UHS) for CPNPP Units 3 and 4 is normally supplied from Lake Granbury; therefore it does not rely on groundwater. The CPNPP is not expected to use groundwater as a source of water for any purpose. Additional information related to local on-site groundwater use is presented in Subsection 2.4.12.3 and the effects of local area groundwater users are discussed in Subsection 2.4.12.3.2.

2.4.12.2.1 Regional and Local Groundwater Uses

Groundwater in Texas is managed locally by groundwater conservation districts. There are 91 such districts established in Texas, each having its own rules, permitting program, and permit records. As of March 2008, the Upper Trinity Groundwater Conservation District that includes Hood County was identified as created. Somervell County was not identified as part of a Groundwater Conservation District (Reference 2.4-256).

Somervell County estimated water use was reported at 46,611 ac-ft in 2004, of which 96 percent was reported as surface water use and 4 percent groundwater use. Hood County estimated water use was reported at 11,857 ac-ft in 2004, of which 62 percent was reported as surface water use and 38 percent groundwater use. Total water use for Hood and Somervell counties represents 1.65 percent of the total reported water use in the Brazos River Basin (Reference 2.4-257).

Groundwater withdrawal from the Trinity aquifer in 2003 was estimated at 172,098 ac-ft, of which approximately 64 percent was municipal use, 20 percent irrigation, 10 percent livestock, 3 percent mining, 3 percent manufacturing, and less than one percent steam electric (Reference 2.4-257). The primary groundwater source

for Hood and Somervell counties is the Trinity aquifer in which a majority is obtained from the Twin Mountains Formation. Groundwater well information obtained from the TWDB indicated a total of 394 wells in the 2-county area (Reference 2.4-258). Of the 394 wells listed, 43 were listed as unused, and no use was given for nine of the wells. Information regarding these wells is provided in Table 2.4.12-201. The well locations, use category, and recharge area are shown on Figure 2.4.12-205. A review of the well database indicated that of the 342 wells with identified uses, 52 percent were for public supply use, 27 percent were for domestic use, 8 percent were for industrial use, 7 percent were for livestock use, and 6 percent were for irrigation use.

The estimated 2003 groundwater withdrawal in Somervell County was 1726 ac-ft, which is approximately 1.00 percent of the total withdrawals from the Trinity aquifer. Approximately 55 percent of this monthly withdrawal was for municipal use, 41 percent was for mining use, 2 percent was for steam electric use, 2 percent was for livestock use, and less than 1 percent was for manufacturing use. Table 2.4.12-202 shows 2003 groundwater withdrawals by use category for Hood and Somervell counties (Reference 2.4-257).

The estimated 2003 groundwater withdrawal in Hood County was 5729 ac-ft, which is approximately 3.33 percent of the total withdrawals from the Trinity aquifer. Approximately 91 percent of this withdrawal was for municipal use, 5 percent was for livestock use, 3 percent was for mining use, and less than 1 percent was for steam electric use (Reference 2.4-257).

Twelve existing water wells were identified on the CPNPP site. The wells include seven inactive potable water wells that support CPNPP Units 1 and 2 operations, one inactive potable water well associated with Squaw Creek Park, and four observation wells. Two of these wells use vertical centrifugal pumps and five wells use submersible pumps. Information regarding these wells is provided in Table 2.4.12-203, and the well locations are shown on Figure 2.4.12-206. On-site groundwater withdrawal information for 2006 was obtained from an annual report prepared by Luminant (Reference 2.4-217). The report indicated on-site withdrawals of 27.90 ac-ft (9,092,700 gal) from five active wells in 2006, which is a use rate of 24,911.5 gallons per day (gpd) or approximately 17.3 gallons per minute (gpm). Monthly use data for 2006 is provided in Table 2.4.12-204.

Luminant is not anticipating using groundwater as an operational or safety-related source of water for CPNPP Units 3 and 4, and has implemented a conservation plan for future groundwater withdrawals at the CPNPP site. See Subsection 2.4.1.2.5 for additional information regarding existing water wells at the CPNPP site.

2.4.12.2.2 Projected Future Regional Water Use

Future consumptive water use information was obtained from the 2006 Brazos Region G Water Plan, which forecasts water demands by category for the years 2010 to 2060 (Reference 2.4-208). The water demand estimates compiled for

each type of water use do not specify future ground or surface water demand. Estimated demand surpluses or shortages are based on projected surface and groundwater supplies. Additionally, projections for non-consumptive water uses, such as navigation, hydroelectric generation, environmental flows, and recreation are not presented. As shown in Table 2.4.12-205, total water use for the region is projected to increase from 835,691 ac-ft in 2010 to 1,150,973 ac-ft in 2060, a 38 percent increase. The projections indicate that municipal, manufacturing, and steam-electric water use as percentages of the total water use increase from 2000 to 2060, while mining, irrigation, and livestock water use are projected to decrease or remain constant as percentages of the total.

As shown on Table 2.4.12-206 water demands in Hood and Somervell counties are projected to increase from 44,939 ac-ft in 2010 to 62,600 ac-ft in 2060, a 39 percent increase (Reference 2.4-208). It should be noted that the Somervell County steam-electric water user group demands identified in the 2006 Brazos Region G Water Plan do not account for CPNPP Units 3 and 4 water demands, subsequently the additional demands for CPNPP Units 3 and 4 are not included in the regional water demand projections provided in Table 2.4.12-205 nor the county water demands provided in Table 2.4.12-206. The revised projected regional and county water demands are to be included in the 2011 Brazos G Water Plan.

The 2006 Brazos Region G Water Plan identifies ten water user groups within Hood County and seven water user groups within Somervell County (Reference 2.4-208). Table 2.4.12-207 identifies each water user group and their corresponding water surplus or shortage in the years 2030 and 2060. For each water user group with a projected shortage, a water supply plan has been developed to mitigate the shortage. Strategies to meet the identified shortages in Hood and Somervell counties include the purchase of water from local municipalities, the use of surface water over groundwater, conservation, and voluntary redistribution of surface water from steam electric users (Luminant). No projected shortages for the steam electric water user group were identified in Hood or Somervell counties.

2.4.12.2.3 Historical On-Site/Vicinity Groundwater Level Fluctuations

Most of the CPNPP Units 3 and 4 site is situated in areas disturbed by previous construction activities associated with the construction of the existing CPNPP Units 1 and 2. Those areas are covered with undifferentiated and engineered fill, gravel roadways and parking areas, and concrete building foundation pads. During the preliminary work at the CPNPP Units 1 and 2 site, 17 piezometers were installed in the upper zone of the Glen Rose Formation and monitored for a period of one year. Static water levels observed in 1972 are depicted on the hydrographs presented in Figure 2.4.12-207 and ranged from 749 ft msl to 830 ft msl. All piezometric levels recorded were measures of perched water in the upper zone of the Glen Rose Formation in the immediate area of each piezometer. The resulting elevation range of 740 ft msl to 830 ft msl is attributed to surface run-off and is not a true measure of permanent groundwater in the Glen Rose Formation.

No groundwater was encountered during excavation or construction of CPNPP Units 1 and 2; therefore, there was no dewatering at the site during or after construction of the units (Reference 2.4-214).

2.4.12.2.4 On-Site/Vicinity Groundwater Level Fluctuations

In October 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for the CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation on-site. Figure 2.4.12-208 shows the monitor well locations. Details regarding well construction are presented in Table 2.4.12-208.

Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Monitoring wells were designated as follows, where XX denotes the well or cluster number for the three zones:

A-zone wells: Regolith or undifferentiated fill monitoring wells (MW-12XXa) were installed if greater than 10 ft of soil was encountered above hollow-stem auger refusal.

B-zone wells: Shallow bedrock monitoring wells (MW-12XXb) were generally completed in the upper 40 to 65 ft of bedrock in an apparent zone of alternating stratigraphy; i.e., claystone, mudstone, limestone, and shale sequences.

C-zone wells: Bedrock monitoring wells (MW-12XXc) were generally completed in deeper bedrock zones consisting of alternating stratigraphy and competent bedrock.

Following well development, water levels were measured from November 2006 to May 2008 (Figure 2.4.12-209) to characterize seasonal trends in groundwater levels. Measured groundwater levels from November 2006 to November 2007 are presented in Table 2.4.12-209. The hydrographs for this groundwater data are presented on Figure 2.4.12-209 and also show precipitation data. The groundwater elevation data is presented by well/cluster location and includes approximate screen elevations for each well in the cluster. In addition, the hydrographs depict rainfall totals for the period of interest. Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Overall, the hydrographs show that water levels in the deeper Glen Rose Formation (C-Zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating the wells were dry (no groundwater infiltration into the well) or exhibiting slow recharge with the static water level not in equilibrium with the groundwater within the formation. With the exception of seven monitoring wells (MW-1201b, MW-1205b, MW-1207b, MW-1209b, MW-1211b, MW-1212b,and

MW-1217b), hydrographs from the shallow bedrock wells (B-Zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting the static water level within the wells are not in equilibrium with the groundwater within the formation. Available historical information on groundwater and groundwater trends in the Glen Rose Formation is presented in Subsection 2.4.12.2.3.

Water Levels and Potentiometric Elevations in the Regolith (A – Zone)

Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to February 2008. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells, there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.

Monitoring well MW-1211a was installed on the northeast portion of CPNPP Units 3 and 4 in undifferentiated fill material. Water levels in this monitoring well were consistent with the normal pool elevation of SCR (775 ft msl) indicating possible hydraulic communication between the former drainage swale and SCR.

Representative potentiometric surface maps for the four quarters (Figure 2.4.12-210 [Sheets 1 through 4]) show that the general shallow (A-Zone) groundwater movement in the vicinity of CPNPP Units 3 and 4 mimics the surface topography, with an apparent groundwater divide along the long axis of the site peninsula. On the northern portion of the peninsula, a northerly flow toward SCR is observed, and a southerly flow toward the Safe Shutdown Impoundment (SSI) is observed on the south side of the site peninsula.

Water Levels and Potentiometric Elevations in the Shallow Bedrock (B – Zone)

Nine of the 16 wells completed in this zone contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of these wells exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions over the monitoring period.

Well MW-1211b was installed east of CPNPP Unit 3 in the undifferentiated fill material. During installation, an effort was made to install this well in bedrock; however, due to the thickness and nature of the undifferentiated fill material, the boring was terminated at the bedrock surface (approximately 75 ft below ground surface [bgs]). Water level measurements for this well were consistent with those of regolith monitoring well MW-1211a and the normal pool elevation of SCR over the monitoring period; therefore, the groundwater elevation in monitoring well MW-1211b is not considered to be a measurement of groundwater within the shallow bedrock (B-Zone).

Well MW-1209b was installed northeast of CPNPP Unit 3 in the shallow bedrock below the undifferentiated fill material. Water level measurements for this well were consistent with those of the normal pool elevation of SCR over the monitoring period, showing the shallow bedrock at this location is in communication with SCR; therefore, the groundwater elevation in monitoring well MW-1209b is not considered to be a measurement of groundwater within the shallow bedrock (B-Zone).

Well MW-1212b was installed southeast of CPNPP Unit 3 in the shallow bedrock at the apparent southern extent of the undifferentiated fill material. Water level measurements for this well were approximately 10 feet above the normal pool elevation of SCR over the monitoring period. Due to its location on the southern side of the undifferentiated fill material, which isolates the groundwater in this portion of the site from that in the location of the nuclear islands, the groundwater elevation in monitoring well MW-1212b was not used to determine groundwater flow direction within the shallow bedrock (B-Zone).

Only four shallow bedrock (B-Zone) monitoring wells (MW-1201b, MW-1205b, MW-1207b, and MW-1217b) exhibited consistent water levels, indicating equilibrium conditions. After obtaining static conditions between November 29, 2006, and January 23, 2007, groundwater elevations in these four wells stayed within a 13.76 ft range between 820.08 ft msl (MW-1217b; March 24, 2008) and 833.84 (MW-1215b; October 16, 2007). Monitoring well MW-1217b, located near the center point of CPNPP Unit 3, exhibited the greatest variation following attainment of static conditions, showing water level variations within a 6.97 ft range from January 2007 to May 2008. Comparison with recorded rainfall data at the Opossum Hollow Rain Gage did not show a correlation between water level variations and recorded rainfall data during the monitored period.

Groundwater potentiometric surface maps could not be produced based on only four wells completed in the shallow bedrock (B-Zone) that exhibited consistent equilibrium conditions and evidence that the groundwater within the shallow bedrock is recharged from the perched groundwater within the overlying soils. However, the groundwater levels within the four wells show a general groundwater gradient trend towards SCR and it is expected that the groundwater potentiometric surface will follow that of the overlying soils.

Water Levels and Potentiometric Elevations in the Bedrock Monitoring Wells (C - Zone)

Of the 14 groundwater monitoring wells screened in bedrock, six contained no, or negligible, amounts of water over the monitoring period and eight exhibited a slow to steady recharge, with no indication of reliable equilibrium conditions.

Groundwater potentiometric surface maps could not be produced due to the lack of reliable groundwater, or evidence of non-equilibrium conditions within the deeper C-Zone monitoring wells.

Based on the above-mentioned observations, groundwater at the CPNPP 3 and 4 site appears to be limited to a perched interval within the overlying soils on top of the weathered upper Glen Rose Formation limestone (upper bedrock). Based on the lack of reliable groundwater within the bedrock beneath the site soils, groundwater availability decreases significantly with depth. From site observations, it is concluded that the groundwater within the regolith recharges the weathered, upper portions of the bedrock, with little infiltration to deeper bedrock zones.

Groundwater flow direction within the regolith is toward SCR. Flow direction of groundwater within the upper bedrock (groundwater B-Zone) appears to flow eastward toward SCR. However, based on the limited groundwater availability within the bedrock, depicted by long-term, non-equilibrium water levels within most bedrock monitoring wells, groundwater flow within the upper bedrock is limited and likely linked to flow within the overlying perched groundwater.

2.4.12.2.5 Aquifer Characteristics

Groundwater has been identified within the undifferentiated fill, regolith and bedrock beneath the CPNPP Units 3 and 4 sites; therefore, this subsection provides characteristics of these zones. During construction, the undifferentiated fill material and regolith are expected to be removed in the power block area. The foundation elevation is estimated to be approximately 782 ft msl on the bedrock. Groundwater currently measured in the soil zones (undifferentiated fill material and regolith) and the Glen Rose Formation is considered “perched” and will be removed during construction activities. Characteristics of the Glen Rose Formation indicate that it is not a groundwater bearing unit and a permanent dewatering system will not be required.

2.4.12.2.5.1 Porosity

Soil Zones

The soils occurring on the CPNPP site are described in the Hood and Somervell counties soil survey information provided by the USDA Natural Resources Conservation Service’s on-line Soil Data Mart website (Reference 2.4-259). A total of 18 soil mapping phases representing 17 soil series occur within the CPNPP site boundary. Descriptions of each soil series are provided in Table 2.4.12-210 and the location of the soil mapping phases are shown on Figure 2.4.12-211.

The two soil types mapped in the vicinity of the CPNPP Units 3 and 4 build areas include the Tarrant – Bolar association and Tarrant – Purves association. Physical properties for these soil types indicate clay content ranges of 20 to 60 percent, moist bulk densities of 1.10 gram per cubic centimeter (g/cc) to 1.55 g/cc, saturated hydraulic conductivities between 4.2 x 10-5 centimeters per second (cm/sec) and 1.4 x 10-3 cm/sec, and available water capacities of 0.05 inch per inch (in/in) to 0.18 in/in (Reference 2.4-260).

The site is underlain by a sedimentary rock sequence of the Glen Rose Formation which, at the surface, has been weathered to a clayey, silty, sandy overburden soil with some rock fragments (referred to as regolith). However, most of the CPNPP site is situated in areas disturbed by previous construction activities associated with the construction of CPNPP Units 1 and 2. Porosity in the undifferentiated fill or regolith materials was evaluated based on the grain size distributions from the current investigation:

- Undifferentiated Fill - Based on the grain size distribution of the on-site soils, the total porosity was determined by averaging the porosity range for sand, silt, and clay. The average total porosity of the on-site regolith and undifferentiated fill is assumed to be 0.45. Based on a lack of information regarding effective porosity in the undifferentiated fill, an effective porosity of 0.45 was assumed.

- Regolith – As mentioned above, the average total porosity of the on-site regolith and undifferentiated fill/regolith (soils) is assumed to be 0.45. To estimate the effective porosity of the on-site soils, the arithmetic mean of the effective porosities for fine grained sand, silt, and clay were averaged (Reference 2.4-261). The average effective porosity of the on-site regolith and undifferentiated/regolith is assumed to be 0.20.

Bedrock Zones

The bedrock is comprised of limestone from the Glen Rose Formation. The results of the geotechnical analysis performed at the CPNPP Units 3 and 4 site indicated that an average total porosity of the shallow bedrock (limestone and shale) is 25.6 percent and the average total porosity of limestone is 11.9 percent. The Argonne National Laboratory publication, Data Collection Handbook to Support Modeling Impacts of Radioactive Material in Soil, dated April 1993 (Reference 2.4-261) references an arithmetic mean of the effective porosity for limestone of 14 percent. Consequently, the most conservative approach when determining velocity and travel time is to use the measured 11.9 percent porosity value which provides a higher calculated velocity through the shallow bedrock.

2.4.12.2.5.2 Permeability

The permeability of a material is a measure of the ability to transmit water. To assist in determining permeability of the Glen Rose Formation, forty packer-pressure tests were performed in five test borings at 5-foot intervals of varying depth at CPNPP Units 3 and 4 in 2007. The results of these packer tests indicated little to no water take into the Glen Rose Formation; therefore, the formation is essentially impermeable. Detailed examination of cores from test borings revealed minor solutioning features and minimal fractures. Drill water occasionally was lost while drilling through the upper weathered zone and is believed to have occurred at the soil/bedrock interface.

2.4.12.3 Subsurface Pathways

Subsurface pathways include the unsaturated zones and saturated zones beneath the CPNPP Units 3 and 4. Groundwater is the primary transport mechanism for possible liquid effluent release. Groundwater movement and velocity will vary depending on the matrix through which it flows. The rate of flow (i.e. the velocity) of groundwater depends on (1) the hydraulic conductivity and porosity of the medium through which it is moving and (2) the hydraulic gradient. Higher groundwater velocities occur with greater hydraulic conductivity and hydraulic gradient.

It is assumed that a release from either unit would first encounter the engineered fill surrounding the A/B and R/B. This engineered fill material is connected to the fill surrounding various site systems, but in particular to the ESW piping tunnels and UHS basins, since these are embedded at an equal depth as the A/B and R/B (Figure 2.4.12-212). Portions of the engineered fill surrounding these systems are in contact with the existing fill to the east of Unit 3 and to the north of Unit 4; therefore, a release from the unit will flow within the engineered fill until it comes in contact with the existing fill. As stated in Subsection 2.4.12.2.4, the existing fill is in communication with SCR and has a higher hydraulic conductivity; therefore, groundwater within the engineered fill surrounding the A/B and R/B will be drained through the contact with the existing fill into SCR. As the hydrogeologic properties of the engineered fill are unknown at this time, the groundwater transport time through the engineered fill will be considered negligible and any release will be conservatively assumed to begin at the engineered fill/existing fill boundary closest to SCR.

Due to very slow groundwater recharge, single well slug tests were performed on six monitoring wells using the Bouwer & Rice method in April of 2007 at CPNPP Units 3 and 4. Of the six wells tested, two were screened in the regolith, one was screened in an undifferentiated fill/regolith zone, and three were screened in the shallow bedrock zone. Hydraulic conductivity for the wells screened in the regolith or undifferentiated fill/regolith zone ranged from 2.93×10^{-5} cm/s to 5.00×10^{-4} cm/s. Hydraulic conductivity for the wells screened in the shallow bedrock ranged from 6.29×10^{-6} cm/s to 1.37×10^{-5} cm/s.

A step test and 72-hr pumping test were performed on aquifer pump test well RW-1 in April of 2007. To investigate groundwater communication with SCR, pump test well RW-1 was installed in an area of undifferentiated fill within a former drainage swale on the northeast portion of CPNPP Units 3 and 4. The step test was performed to determine the pumping rate for the 72-hr pumping test. Data for the step test and 72-hr pumping test were analyzed using the Cooper-Jacob Step Test and Theis Recovery Test methods. The results of the 72-hr pump test estimated hydraulic conductivity at 1.70×10^{-3} cm/s during pumping and 3.5×10^{-3} cm/s during recovery.

Due to site grading activities during plant construction, maximum groundwater elevations within the plant site will be limited to the invert elevation of the southern and western drainage trench, which has a maximum elevation of 820 ft msl. Recharge to the upper bedrock zone in the plant site will be restricted by drainage into this trench; therefore limiting the maximum conservative groundwater elevation in the plant site to 820 ft. msl.

Soil distribution characteristics for radiological isotopes (i.e., Co_{60}, Cs_{137}, Fe_{55}, I_{129}, Ni_{63}, Pu_{239}, Tc_{99}, U_{235}) were determined from soil and water samples collected along the preferred groundwater flow path. This data is discussed in detail in Subsection 2.4.13 to assist in the development of transport calculations for fate and transport analyses in the event of accidental releases of effluents to groundwater.

2.4.12.3.1 Groundwater Pathways

Although the discussions of groundwater movement is a reasonable scenario for groundwater flow, it is assumed that the actual groundwater is subject to three-dimensional control structures (horizontal, vertical, and any secondary porosity that may be present) and does not have uniform flow across the site.

Groundwater pathways are considered from the CPNPP Unit 3 and 4 Auxiliary Buildings where the boric acid tank (BAT) is located, to SCR, which is the nearest potential receptor.

Placement of engineered fill surrounding the A/B, R/B, ESW piping, UHS basins, and circulating water piping will affect the direction and flow rate of groundwater infiltrating from the remaining bedrock. Portions of the engineering fill surrounding these subsurface structures are in communication with the existing fill on the site (Figure 2.4.12-212). The existing fill is in communication with SCR, and due to the low hydraulic conductivity of the bedrock, it is expected that groundwater infiltrating into the engineered fill will migrate through the engineered fill into the existing fill and then enter SCR, with little to no groundwater transport through the upper bedrock. Since the geohydrologic properties of the engineered fill are unknown at this time, groundwater transport time through the engineered fill is conservatively assumed to be negligible.

Two postulated groundwater pathway scenarios, CPNPP Unit 3 to SCR through the existing fill east of CPNPP Unit 3, and CPNPP Unit 4 to SCR through the existing fill north of CPNPP Unit 4, represent the most conservative pathways from a two-reactor site where groundwater flow is possibly in different directions from each unit (Figure 2.4.12-212). Both flow paths utilize a conservative, straight-line flow path approach from the point of release and the shortest distance and highest measured hydraulic conductivity for the pathway assessed. A straight-line flow path is considered the most conservative as the actual groundwater pathways are expected to be tortuous, resulting in longer transport times and hydraulic conductivities (K_h) that are expected to be lower than the highest measured.

To estimate groundwater travel time through the existing fill, the effective porosity of the site soil (0.20 from Subsection 2.4.12.2.5.1) is used as a conservative estimate. As post-construction groundwater levels within the existing fill are unknown, groundwater elevation within the existing fill is conservatively assumed to be at the maximum expected groundwater level of 820 ft msl. The normal operating pool elevation for SCR is 775 ft. msl; however, the minimum operating SCR pool elevation of 770 ft msl is used to produce the highest conservative hydraulic gradient.

The swale east of Unit 3 was filled with the excavation debris from Units 1 and 2; thus, it is considered to be a haphazard mélange of clay through boulder size material with some debris present. The swale north of Unit 4 appears to have been constructed in a more methodical manner to support building foundations. Construction data for the swale fills are not available. However, it is assumed the fill properties are sufficiently different to allow the conservative use of the individual hydraulic conductivities from each swale fill testing in the groundwater pathway analysis based on the following:

- evidence from visual observations
- data obtained from the geotechnical drilling program
- results of the pump and slug test analysis performed on monitoring wells within the individual existing fill materials
- there is no connection between the two filled areas
- the appearance of different placement methods and dates of the swale fill materials.

For the groundwater velocity and travel time assessment described below, the groundwater pathway 1 hydraulic conductivity (K_h), measured from observation well RW-1 recovery test (3.50 x 10^{-3} cm/s) represents the hydraulic conductivity measured in the existing fill east of Unit 3. The groundwater pathway 2 K_h, measured from monitoring well MW-1219a slug testing (5.00 x 10^{-4} cm/s) represents the hydraulic conductivity measured in the existing fill north of Unit 4.

For groundwater pathway 1 (Figure 2.4.12-213), it is assumed that an instantaneous release from the BAT would travel out of the CPNPP Unit 3 A/B into the engineered fill surrounding the A/B and R/B. It would then travel to the closest engineered/existing fill interface, located to the east of the Unit 3 turbine building. For conservatism, it is assumed that the transport time to the fill interface will be negligible. It will then travel 600 ft through the existing fill to the closest release location in SCR. The travel time from the release point to SCR via the existing fill east of Unit 3 is conservatively estimated at 145 days.

For groundwater pathway 2 (Figure 2.4.12-214), it is assumed that an instantaneous release from the BAT would travel out of the CPNPPUnit 4 A/B into the engineered fill surrounding the A/B and R/B. It would then travel to the closest engineered/existing fill interface, located to the north of the CPNPP Unit 4 UHS basin. For conservatism, it is assumed the transport time to the fill interface will be negligible. It will then travel 350 ft through the existing fill to the closest release location in SCR. The travel time from the release point to SCR via the existing fill north of Unit 4 is conservatively estimated at 346 days.

Groundwater gradients, velocities, and travel times are summarized in Table 2.4.12-211.

Cross-sections depicting the post-construction groundwater flow pathways from CPNPP Unit 4 to SCR are presented in Figure 2.4.12-214.

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith) and B-zones (shallow bedrock) discussed in Subsection 2.4.12.2.4 will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the power block area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the CPNPP Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water infiltration into the limestone on which the foundation will be constructed.

Based on the excavation of the perched zones in the A-zone and B-zones in power block area; the impermeable nature of the Glen Rose Formation, and the absence of any water wells producing from the Glen Rose Formation in the CPNPP Units 3 and 4 site area, impact to present and projected groundwater users is not anticipated. The postulated groundwater pathway scenarios discussed in this subsection and further in Subsection 2.4.13, project SCR to be the nearest receptor. Evaluation of the accident effects of a contaminant release to groundwater from CPNPP Units 3 and 4 is discussed in detail in Subsection 2.4.13.

2.4.12.3.2 Nearby Groundwater Users

While no use of groundwater at the CPNPP site is planned, consideration is given for the movement of groundwater beneath the site because of pumping. Potable-use wells at CPNPP are completed in the Twin Mountains Formation, a confined aquifer below the impermeable Glen Rose Formation. Most domestic wells in the area are completed in the Twin Mountains Formation (Table 2.4.12-212). The on-site wells completed in the Twin Mountains Formation are not considered capable of reversing groundwater flow beneath the CPNPP Units 3 and 4 site. There are no domestic or public water supply wells within a 0.5-mi. radius of the site that are completed in the Glen Rose Formation. (Figure 2.4.12-204). No off-site wells are considered capable of reversing groundwater

flow beneath the site, or vice versa, based on the geographic positions of these wells (i.e., the distance of the domestic wells from the power block area and their completion in the Twin Mountain Formation).

2.4.12.4 Monitoring or Safeguard Requirements

Accident effects are discussed in Subsection 2.4.13 and the radiation protection program is discussed in Section 12.5. Additionally, analysis of the relationship of the CPNPP groundwater to seismicity and the potential for related soil liquefaction and the potential for undermining of safety-related structures is discussed in Section 2.5. A groundwater monitoring program will be developed before fuel load that will include radiological sampling based upon post-construction configuration.

2.4.12.5 Site Characteristics for Subsurface Hydrostatic Loading

According to the Design Control Document (DCD) for the US-APWR, the design maximum groundwater elevation is 1 ft below plant grade. The CPNPP plant grade elevation is 822 ft msl; therefore, the design maximum groundwater elevation is 821 ft msl relative to the current elevation of the Glen Rose Formation. The Glen Rose Formation is an impermeable limestone that confines the groundwater in the underlying Twin Mountains Formation aquifer. Not all of the wells completed in the Glen Rose Formation were sampled; however, the wells that were sampled and purged, purged dry and water did not return for several days to weeks. All deep Glen Rose wells have been reported as "dry" or reported with less than 1-foot of water. The Twin Mountains Formation is at least 230 ft below the Glen Rose Formation; therefore, the installation and operation of a permanent dewatering system is not planned. A dewatering system will not be required during construction. Normal construction practices will be employed to remove water from seepage and rainfall. Based on the removal of the soil overlying the bedrock surrounding the site foundations, and the maximum groundwater elevation within the engineered fill constrained by the southern and western trench drain to less than 820 ft. msl, the design maximum groundwater elevation is expected to be satisfied.

2.4.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surfacewaters

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CP COL 2.4(1) Add the following at the end of the DCD Subsection 2.4.13.

Historical and projected groundwater flow paths were evaluated in Subsection 2.4.12 to characterize groundwater movement from the nuclear island area to a point of exposure. Figure 2.4.12-203 depicts subsurface conditions that control the movement of groundwater beneath the CPNPP Unit 3 and 4 site. Based on groundwater flow directions (Figure 2.4.12-209, Sheets 1, 4, 7, and 10), different flow paths are applicable from Units 3 and 4 via horizontal groundwater movement to the nearest surfacewater body (SCR). Subsection 2.4.12 provides the locations and users of surface water in the CPNPP site area.

A conceptual model of radionuclide transport through groundwater to the nearest surfacewater body is described below. The conceptual model and alternate conceptual model developed consider both vertical and horizontal radioactive liquid effluent transport based upon the post-construction configuration of CPNPP Units 3 and 4 (see Figures 2.4.12-212 through 2.4.12-214).

2.4.13.1 Identification of Source Term and Soil/Water Distribution of Liquid Effluent

In performing the evaluation of Postulated Radioactive Releases Due to Liguid-Containing Tank Failures, the following tanks were considered in determining which tank would have the highest concentration and the largest volume of radionuclides:

Holdup Tank - located in the Auxiliary Building (A/B), a Seismic Category II building.

Waste Holdup Tank - located in the A/B

Boric Acid Evaporator - located in the A/B

Boric Acid Tank - located in the A/B

Volume Control Tank -located in the Reactor Building (R/B). a Seismic Category I Building.

Auxiliary Building Sump Tank - located in the A/B

Reactor Building Sump Tank - located in the R/B

Primary Makeup Water Tank - located outside

Refueling Water Storage Auxiliary Tank - located outside

Chemical Drain Tank - located in the A/B

The Volume Control Tank, the Chemical Drain Tank, and Sump Tanks were eliminated from consideration based on smaller volumes and lower radionuclide contents than the Boric Acid Tank (BAT). The Primary Makeup Water Tank was eliminated from consideration based upon the fact that the Primary Makeup Water Tank stores demineralized water from the Treatment System. and low level radioactive condensate water from the Boric Acid Evaporator. Condensate water contains low levels of radionuclide concentrations. including tritium. Additionally, the Refueling Water Storage Auxiliary Tank (RWSAT) was eliminated from consideration because it stores refueling water. Prior to refueling, tank water is supplied to the refueling cavity where the reactor coolant radionuclide concentration dilutes with refueling cavity water. Radionuclide concentration of cavity water is reduced by the purification system of the Chemical and Volume Control System (CVCS) and the Spent Fuel Pit Cooling and Purification System (SFPCS) during refueling operations. Upon refueling completion, part of the cavity water is returned to this tank where the radionuclide concentration is low. Accordingly. the impact of RWST or Primary Makeup Water Storage Tank failure is small.

After eliminating the tanks described above, the remaining tanks left to consider for the failure analysis are those in the A/B, which is a seismic category II Building. As shown in DCD Figure 1.2-29, these tanks are located on the lowest elevation of the A/B at elevation 793 ft ms. In selecting the appropriate tank for the failure analysis, the guidance in Branch Technical Position (BTP) 11-6 was utilized based upon the concentrations generated from the RATAF Code for Pressurized Water Reactors. The concentration of the radioactive liquid in the tanks, such as the Boric Acid Evaporator, the Holdup Tank, and the BAT, are larger than the Waste Holdup Tank since they receive reactor coolant water extracted from the Reactor Coolant System. Since the enrichment factor of 50 is considered for the liquid phase of the Boric Acid Evaporator. the radioactive concentrations in the liquid phase of the Boric Acid Evaporator, and in the BAT (which receives the enriched liquid from the Boric Acid Evaporator) becomes large when compared to the other tanks. The BAT has been selected since its volume is larger than the liquid phase of the Boric Acid Evaporator. Credit is taken for the removal effect by demineralizers or other treatment equipment for the liquid radioactive waste prior to entering the tank. No chelating agents are used in the plant system design in order to provide chemical control of the reactor-coolant. Only a very small amount of chelating agents is used in the sampling system for analysis. The sampling drain, which contains only a small amount of chelating agents is directly sent to the dedicated chemical drain tank and treated separately. Chemical agents used in laboratory analysis are also sent to the chemical drain tank for treatment. Therefore, neither the chelating agents nor the chemical agents used in the sampling analysis will have any effect on the transport characteristics of the source term liquid effluent release analysis.

The source term concentrations considered for these tanks are identified in DCD Table 11.2-17, and were calculated using NUREG-0133 and the RATAF Code for Pressurized Water Reactors. The BAT is located in the northeast (NE) corner of the A/B (see DCD Figure 12.3-1). The A/B basemat elevation is at approximately 785 ft msl. The BAT elevation is expected to be at 793 ft msl. Ground level at the site is expected to be at 822 ft msl. The BAT contained the largest concentration and volume of radionuclides that was closest to the effluent concentration limits (ECLs). Isotope concentrations less than 1.0 x 10^{-3} in fraction of concentration limits are excluded from the evaluation. Since credit cannot be taken for liquid retention by unlined building foundations, it is assumed that 80 percent of the content of the tank is released to the environment, consistent with the guidance in BTP 11-6, March 2007.

While groundwater functions as the transport media for fugitive radionuclides, interaction of individual radionuclides with the soil matrix delays their movement. The solid/liquid distribution coefficient, K_d, is, by definition, an equilibrium constant that describes the process wherein a species (e.g., a radionuclide) is partitioned by adsorption between a solid phase (soil) and a liquid phase (groundwater). Soil properties affecting the distribution coefficient include the texture of soils (sand, loam, clay, or organic soils), the organic matter content of the soils, pH values, the soil solution ratio, the solution or pore water concentration, and the presence of competing cations and complexing agents. Because of its dependence on many soil properties, the value of the distribution coefficient for a specific radionuclide in soils can range over several orders of magnitude under different conditions. The measurement of distribution coefficients of radionuclides within the preferential groundwater pathways allows further characterization of the rate of movement of fugitive radionuclides in groundwater.

The site-specific K_d coefficients were selected based upon radionuclides listed in 10 CFR Part 20, Appendix B, Table 2. Three soil borings were chosen for sampling characteristics. Soil and groundwater samples were collected from monitoring wells MW-1201 (located southwest of the CPNPP Unit 4 nuclear island), MW-1208 (located east of the Unit 3 nuclear island), and MW-1219 (located northeast of the Unit 4 nuclear island) (Figure 2.4.12-207). Soil samples from each monitoring well were collected, based on the availability of recovered soils, at depths ranging from approximately 18 to 54 feet below ground surface. Dry wells exhibiting very slow recharge, and the aquifer testing observations wells were not considered for sampling. Soil boring samples gathered from the two hydraulically upgradient wells and hydraulically downgradient wells were submitted to Argonne National Laboratory for analysis of the radionuclides listed in FSAR Section 2.4.13 based upon the radionuclides listed in 10 CFR Part 20, Appendix B and those radionuclides that would be expected to exist in the tanks were considered for the failure analysis. The soil boring samples were submitted for laboratory analysis of soil distribution characteristics for specific radiological isotopes (i.e., Co-60, Cs-137, Fe-55, I-129, Ni-63, Pu-242, Sr-90, Tc-99, U-235). Results of the K_d analyses are presented in Table 2.4.13-201.

Since the A/B is where the BAT, the Holdup Tank and the Waste Holdup Tanks are to be located at Units 3 and 4, appropriate values were evaluated for "nuclides of interest" (Table 2.4.13-201) based on transport to SCR without retardation or retention through subsurface media. Thus, using the conservative transport time analysis, and considering nuclide decay times, those nuclides which could be expected to challenge 10 CFR Part 20, Appendix B, concentration limits were considered. The BAT was selected as the tank that had the greatest volume and largest concentration of radionuclides, where credit is taken for removal equipment and demineralizer beds. The purpose of the K_d analysis was to estimate the potential migration of accidental releases from the footprint areas of the proposed new units. The K_d results presented in Table 2.4.13-201 indicate that the radionuclides would be delayed in their movement through the groundwater pathway to SCR. The tank failure analysis assumed no distribution of contaminants (no K_d coefficients used) based upon the site-specific hydrogeological characteristics. It is conservatively assumed that the contaminants would transport along the groundwater pathway horizontally to SCR without retardation or retention in the subsurface media, and that there would be no groundwater dilution prior to reaching SCR.

2.4.13.2 Development of Alternate Conceptual Model and Site-Specific Geological and Hydrogeological Parameters

The alternative conceptual models were used to determine a bounding set of plausible groundwater flow paths by considering the nearest surface water body, SCR, current groundwater elevations measured in wells near the proposed power block area, the measured pool elevation of SCR (gradient to the SCR) and a conservative pathway from a postulated release point to SCR.

After exploring alternative transport pathways, two plausible pathways were determined to bound potential release pathways. Refer to Figure 2.4.13-212 and associated cross section Figures 2.4.12-213 and 2.4.12-214 for the horizontal release pathways. Vertical release pathways are eliminated from consideration as discussed in Subsection 2.4.13.4. Alternate horizontal groundwater pathways from each unit moving south or west from the BAT A/B location were eliminated from consideration as this movement would be away from SCR and would not be consistent with the hydraulic gradients for the area surrounding the CPNPP Units 3 and 4 shown on Figure 2.4.12-210, Sheets 1 through 4.

CPNPP Units 3 and 4 are to be constructed on the Glen Rose Formation. The Glen Rose limestone is essentially impermeable, ranging from 217 to 271 ft thick, and is underlain by the Twin Mountains Formation, which contains the first aquifer beneath the site. Figures 2.5.5-202 and 2.5.5-203 provides a generalized cross section of the pre-construction site conditions. Figures 2.4.12-213 and 2.4.12-214 show the post-construction pathway cross-sections for the shortest distance releases to SCR via groundwater pathways. The groundwater flow pathways were developed based on groundwater measured in monitoring wells in the CPNPP Unit 3 and 4 plant area and measured elevations in SCR. Wells were installed across the site in zones to define the groundwater bearing capabilities and

properties of the zones, and identify the hydraulic connectivity between the zones, if any. The well zones are defined as A-Zone (regolith or undifferentiated fill material), B-Zone (shallow bedrock) and C-Zone (deeper bedrock) and are described in Subsection 2.4.12.2.4.

The process used to develop alternative conceptual models of groundwater flow included the following:

- Groundwater flow pathways were developed based on groundwater measured in monitoring wells in the Units 3 and 4 plant area, measured elevations in SCR, surface topography, and observed water levels over time.

- Groundwater measured in all three zones was considered perched based on measurements. Groundwater in the A-zone regolith was attributed to surface water infiltration. Groundwater measured in the undifferentiated fill near SCR was attributed to SCR.

- Groundwater in the B-zone was not continuous across the site Non-equilibrium conditions and the reported dry wells in the B-zone wells indicated that the groundwater was perched. Groundwater located in fill areas near SCR was found to be in communication with SCR.

- Negligible groundwater was gauged in the C-zone wells, representing essentially dry conditions. Consequently, this zone was not considered a groundwater bearing unit.

- Post-construction section configuration of the A/B building, the Ultimate Heat Sink (UHS) cooling tower structure area and other structures were used in identifying the bounding set of plausible pathways. In addition to Figures 2.4.12-213 and 2.4.12-214 horizontal pathway cross sections, the following site plan views and section plans were utilized in identifying the bounding set of plausible pathways:

  - Site Plan View Figure 1.2-1R;

  - Power Block at Elevation 793” ft msl Plan View Figure 1.2-2R;

  - ESW Pipe Tunnel Sectional View A-A’ Figure 1.2-202; and

  - Ultimate Heat Sinks A and B Sectional Views Figure 1.2-206.

- Rainfall infiltration effect on the liquid effluent and plausible release pathway is also considered based upon post-construction structures and building configurations. Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis. Rainfall infiltration effects are discussed in Subsection 2.4.13.3.

2.4.13.3 Potential Effects of Construction on Groundwater Flow Paths

The current soil and rock material comprising the hydrologic A-zone (undifferentiated fill and regolith), and the B-zone (shallow bedrock) will be removed for construction of plant foundations, resulting in the removal of the perched groundwater from the plant area. Post-construction surface water infiltration to the Glen Rose Formation limestone will be reduced with the construction of surface water impoundments and an improved drainage system throughout the Units 3 and 4 site. The grading and drainage plan and placement of engineered fill material are designed to preclude surface water buildup near the plant foundation, reducing the possibility of surface water infiltration into the limestone on which the foundation will be constructed.

During construction, the undifferentiated fill material and regolith will be removed in the power block area, and replaced with engineered fill material. A dewatering system will not be used but rainfall and seepage will be removed during construction.

In October 2006, a groundwater investigation was initiated as part of the subsurface study to evaluate hydrogeologic conditions for CPNPP Units 3 and 4. As part of this groundwater investigation, 47 monitoring wells were installed at 20 locations within the Glen Rose Formation onsite. Due to the variable nature of groundwater reported at the CPNPP site, the well clusters were installed across the footprint of CPNPP Units 3 and 4 from west to east of the reactor areas to define the groundwater bearing capabilities and properties of the zones likely to be affected, and to identify the hydraulic connectivity between the zones, if any. Following well development, water levels were measured from November 2006 to May 2008 to characterize seasonal trends in groundwater levels.

Rainfall data presented was collected from the Opossum Hollow rain gauge located approximately 3.4-mi southwest of the CPNPP Unit 3 and 4 site. Hydrographs were developed and are presented in Figure 2.4.12-209. These hydrographs show that water levels in the deeper Glen Rose Formation (C-zone) do not fluctuate and remain at a constant level near the base of the well or depict a steadily increasing water level, indicating that this water is not actual groundwater. Hydrographs from the shallow bedrock wells (B-zone) show a slow and steady increase of water levels over time with little to no fluctuations, also suggesting water levels are related to infiltration from the overlying soils and not actual groundwater. Hydrographs from the regolith/fill material wells (A-zone) indicate some slight fluctuations that may be tied to seasonal rainfall. In some of the A-zone wells there appears to be a slight increase in water levels that may correspond to the spring seasons but there is no significant correlation in the A-zone wells across the site in response to rainfall.

The water levels in the regolith/fill material and the upper zone of the Glen Rose Formation (A-zone and B-zone, respectively) were attributed to surface run-off and were not a true measure of permanent groundwater in the formation.

Groundwater steadily increased from December 2006 to July 2007. Water levels remained constant or decreased slightly from August 2007 to February 2008.

Nine of the 16 wells completed in Shallow Bedrock (B – Zone) contained no, or negligible, amounts of water for up to eight months before exhibiting measurable water (greater than 1 ft). The majority of these wells exhibited a slow to steady recharge with no indication of reliable equilibrium conditions over the monitoring period.

Of the 14 groundwater monitoring wells screened in Bedrock (C-Zone), six contained negligible to amounts of water over the monitoring period and eight exhibited a slow to steady recharge with no indication of reliable equilibrium conditions.

The Grading and Drainage Plan shown on Figure 2.4.2-202 was developed based upon the effects of local intense precipitation, as discussed in Subsection 2.4.2.3, and aids in moving precipitation away from structures and buildings considered in the plausible pathways for the liquid effluent release analysis.

Rainfall infiltration is not considered a contributing factor affecting the source term release pathway. No dilution effects of groundwater or rainfall are considered in the liquid effluent release analysis.

2.4.13.4 Vertical Release Pathway Elimination

Both SCR and the CPNPP Units 1 and 2 restricted potable water supplies wells were considered as receptors. The CPNPP Units 1 and 2 potable water supply wells are restricted access potable water supply wells completed in the Twin Mountains Formation aquifer and approximately 1990 feet south of the CPNPP Unit 3 A/B. The nearest unrestricted potable water supplies completed in the Glen Rose Formation are approximately 4 miles south of the Unit CPNPP 3 A/B. and the nearest unrestricted potable water supply wells completed in the Twin Mountains Formation is approximately 1 mi west of the Unit CPNPP 4 A/B (FSAR Subsection 2.4.12.3.2 and Figures 2.4.12-204 and 2.4.12-206). The restricted potable water supply wells in Units 1 and 2 (Figure 2.4.1-213) were not considered as possible receptors based upon the following:

The BAT is at elevation 793 ft msl, while the Auxiliary Building basemat elevation is at 785 ft msl. Because the Auxiliary Building is a Seismic Category II Building, it is assumed that a crack will form in the building during a seismic event or some other physical phenomena, and the radioactive liquid would travel vertically into the surrounding formation. At this basemat elevation of 785 ft msl, the hydrogeologic formation is in the deeper portion of the Glen Rose Formation, which consists primarily of impermeable limestone. For the release to reach the Twin Mountains Formation, which is approximately 150 feet below the Glen Rose Formation, the liquid release would have to travel completely through the Glen Rose Formation. Vertical migration pathways are considered improbable due to

the thickness (approximately 150 ft) and extremely low hydraulic conductivity of the lower Glen Rose limestone:

- Packer tests in the power block areas show low hydraulic conductivities (10^{-8} to 10^{-9} cm/sec range, or no water takes) from plant grade elevation (822 ft msl) to 677 ft msl (Table 2.5.4-206).
- Transport of contaminates through formations with hydraulic conductivities less than 10^{-6} cm/sec is controlled by diffusion rather than advection (Reference 2.4-295)
- Units 1 and 2 utilized diffusion for contaminant movement and assumed no groundwater transport.
- Discrete engineering layers in the Glen Rose formation can be traced in the subsurface throughout the site and correlated approximately 2000 feet away in the CPNPP Units 1 and 2 borings and historical excavation photographs.
- Known post-construction excavation limits can be correlated with the stratigraphy exposed in the Glen Rose formation photographs.

A complete discussion of the core borings stratigraphy and CPNPP Units 1 and 2 historical excavation photographs as compared to CPNPP Units 3 and 4 borings is provided in Subsection 2.5.4.3.1.

The closest CPNPP Units 1 and 2 potable water supply well is approximately 1.25 miles away (Figure 2.4.1-213) from either the CPNPP Unit 3 or Unit 4 A/B (Figure 2.4.12-208). The liquid release would be in the Glen Rose formation, which at the level of the BAT is essentially impermiable to groundwater flow.Because the vertical migration pathway was considered implausible, the only plausible release scenario would involve a horizontal release to SCR. Therefore, the alternate conceptual models chosen were to transport the liquid radioactive release through the engineered fill and undifferentiated fill/regolith pathway to SCR (as described in Subsection 2.4.12.3.1 and shown on Figures 2.4.12-212 through 2.4.12-214).

2.4.13.5 Liquid Effluent Groundwater Release Pathway to SCR and Summary Analysis Results

Potential groundwater pathways for the transport of contaminants to possible receptors are discussed in Subsection 2.4.12. These potential groundwater pathways are evaluated for a postulated release of the source term activity from the either CPNPP Unit 3 or 4 BAT in this subsection.

After evaluating alternative pathways, the most plausible pathway is groundwater transport of source term activity horizontally towards the east from CPNPP Unit 3, or towards the north from CPNPP Unit 4, to SCR surface water where the nearest receptor is located (Figure 2.4.12-212). The nearest receptor is considered to be

the Roto-cone gravity flow spillway device located at the south end of SCR (Figure 2.4.13-205). An existing Term Permit with the TCEQ, in accordance with the Brazos River Authority, CP-20 (Reference 2.4-296), Section 6.4.1, requires a minimum flow of 1.5 cfs be maintained at the Highway. 144 crossing over Squaw Creek, which eventually flows into the Brazos River. This requires a constant flow from the Roto-cone into Squaw Creek, which is verified at least daily by Luminant. Vertical migration of the source term from a postulated release is evaluated, but not considered a plausible pathway, for groundwater transport to the Twin Mountains Formation aquifer (Subsections 2.4.12.3 and 2.4.13.3). Groundwater transport west and south from either unit are also potential pathways (Subsections 2.4.12.3 and 2.4.13.2), but are not plausible based upon the hydrogeology and hydraulic gradients that exist pre-construction, and would exist post-construction.

The tank failure analysis focuses on the release of the source term from CPNPP Unit 3 because this pathway has the least amount of time through existing fill, least amount of SCR dilution and mixing volume, and the least amount of transport time to the Roto-cone.

As a result, the tank failure release analysis focuses on the bounding CPNPP Unit 3 pathway where the BAT source term activity could quickly be drawn into the CPNPP Units 1 and 2 circulating water (CW) intake (short-circuited) and be discharged closer to the release point, the Roto-cone device.

For the bounding CPNPP Unit 3 pathway (Figure 2.4.12-212), various cases of CW pump operation (no-flow, half-flow or full-flow) were considered to ensure the most bounding scenario is identified, and the resulting effect on mixing and dilution of the source term activity concentration (Table 2.4.13-203).

A postulated source term release from CPNPP Unit 4 as depicted on Figure 2.4.12-212 is also considered a plausible groundwater pathway to enter SCR. The Unit 4 pathway is groundwater transport via existing fill where it will infiltrate into SCR. The source term activity transports via existing fill groundwater at a velocity of 1.01 ft/day (groundwater velocity) with an overall travel time of 346 days as compared to the Unit 3 pathway, where groundwater velocity is 4.13 ft/day for a travel time of 145 days (Table 2.4.12-211) over the 600 feet through existing fill to SCR (Figure 2.4.12-212). Slower travel time through existing fill with similar characteristics to CPNPP Unit 3 existing fill results in a greater dispersion of material, and larger water volume dilution effect. As depicted on Figure 2.4.12-214, once the source term activity infiltrates at the groundwater interface, it will slowly diffuse into SCR surface water. As the source term activity diffuses further into SCR surface water, it will be transported southward with surface water flow. As depicted on Figures 2.4.12-212 and 2.4.13-206, the influence of the CPNPP Units 1 and 2 CW pumps affects surface water flow, especially during summer months with very little inflow into SCR. The source term activity would most likely become entrained in the CW intake and exit similarly to the CPNPP Unit 3 release. Thus, a larger volume of SCR could be credited for this release.

Because the ECLs are met for the CPNPP Unit 3 cases of no-flow, full-flow or half-flow of CW pump (Subsection 2.4.13.5.4 through 2.4.13.5.6), the ECLs are also met for the Unit 4 diffusion case since additional diffusion time and SCR surface water volume could be credited.

This tank failure analysis concludes that, using the most conservative analysis, the BAT activity concentration will be sufficiently diluted by a portion of the existing fill groundwater and further diluted and mixed with SCR water to meet the ECLs specified in 10 CFR 20, Appendix B, Table 2.

The following factors or calculations are utilized in assessing the source term activity concentrations from a postulated release from either CPNPP Units 3 or 4 to the nearest plausible receptor (Roto-cone):

- The source term activity for the BAT was calculated using the RATAF code with 1 percent fuel defect, scaled down to 0.12 percent fuel failure, with appropriate tank factors applied.
- The calculated source term activity concentration remaining after 0.4 years or 145 days of decay is provided in Table 2.4.13-202.
- Potential groundwater pathways are CPNPP Unit 3 to the east or CPNPP Unit 4 to the north (Figure 2.4.12-212).
- Groundwater velocity travel time (Table 2.4.12-211).
- Volume of groundwater available for source term activity dilution.
- Volume of SCR surface water available for source term activity dilution.
- Mixing rate in SCR based upon half-flow or full-flow CW pumps.
- Diffusion in SCR with no-flow CW pumps operating.

In developing the most conservative scenarios, the following are not factored into the analysis. If factored into the analysis, these would provide lower concentrations at the receptor:

- No credit is taken for travel time through the engineered fill into the overall groundwater transport time. This is conservative because travel time increases and allows for additional decay time, dilution, retardation and retention, thereby further reducing the source term activity concentration prior to reaching SCR.
- No credit is taken for retardation, retention or dilution in the engineered fill. This is conservative as these effects would further reduce the source term activity concentration.

- The engineered fill surrounding the ESW tunnel in communication with the existing fill on the east side of the ESW tunnel as depicted on Figures 2.4.12-213 (CPNPP Unit 3 pathway) and 2.4.12-214 (CPNPP Unit 4 pathway) is completely saturated. This is conservative because it allows for the source term activity as a slug to be transported to the existing fill where it subsequently infiltrates into SCR. The engineered fill will not likely be in complete communication with the existing fill and it will not likely be completely saturated at all times allowing for retention, retardation and dilution.

- Only a portion (25 percent) of the total available groundwater is assumed to be available for dilution. This is conservative because a considerable amount of groundwater (approximately 9.98E06 gal) can be found in the existing fill that communicates with SCR.

The following subsection describes the bounding CPNPP Unit 3 pathway scenario to the nearest receptor (Roto-cone gravity drain device).

2.4.13.5.1 Bounding CPNPP Unit 3 Pathway Scenario

A postulated release from CPNPP Unit 3 is the most conservative scenario. It is assumed that a physical phenomenon occurs causing the BAT to rupture and its contents spill to the floor or sides of the A/B (El. 785 ft, which is adjacent to the engineered fill outside the A/B). The tank is assumed to be 80 percent full in accordance with BTP 11-6. The bottom of the BAT cubicle is at El. 793 ft. As shown on Figure 2.4.13-201, the engineered fill is just outside of the BAT cubicle area in the A/B and around the R/B. Since the engineered fill has not been specified at this time, it is also assumed that the source term moves as a slug volume through the groundwater in the fully saturated engineered fill. This is very conservative because it is highly unlikely that the engineered fill would be fully saturated throughout the travel pathway. Additionally, travel through the saturated engineered fill increases travel time, and allows for dispersion and retardation that is not credited in the analysis.

The engineered fill surrounding the ESW tunnel is in contact with the existing fill on the east side of the ESW tunnel as depicted on Figure 2.4.12-213. As depicted on Figures 2.4.12-213 and 2.4.13-201, a stormwater retention pond is located east of Unit 3 that has an overflow elevation of approximately 810 ft msl,and a bottom elevation of approximately 800 ft msl. Groundwater elevations within the existing fill will be approximately equal to the surface elevation of SCR. For the purpose of the existing fill groundwater calculation, an SCR minimum operating elevation of 770 ft msl was used. The bottom of the stormwater retention pond is located within the existing fill east of CPNPP Unit 3, and is approximately 30 feet above the groundwater surface within the existing fill. Therefore, the presence of the stormwater retention pond will not affect the existing fill groundwater volume, nor intercept groundwater impacted by the postulated release from CPNPP Unit 3. Although not expected, recharge from the stormwater retention pond would serve to produce a shallower groundwater gradient, thereby producing a slower

groundwater velocity and travel time for the postulated release and a less conservative analysis of groundwater transport from CPNPP Unit 3. The existing fill is in communication with the SCR surface water.

Based upon site-specific hydrogeological data, the groundwater travel time through the existing fill is 145 days. Groundwater velocity within the existing fill material is based on (Table 2.4.12-211):

- The engineered fill surrounding the ESW pipe tunnel is saturated to a maximum groundwater elevation of Elevation High (E_h) = 820 ft msl.
- SCR operating low range is used for volume calculations (before makeup from Lake Granbury) elevation (E_l) = 770 ft msl.
- Distance to SCR (L_G) from the ESW and groundwater interface = 600 ft.
- Groundwater hydraulic gradient (E_h-E_l) / L_G = 0.0833 ft/ft.
- Hydraulic Conductivity (K_h) of the existing fill material = 3.50E-03 cm/sec = 1.15E-04 ft/sec = 9.92 ft/day.
- Effective Porosity (η_e) = 0.2.
- Velocity (V) of groundwater through existing fill = (K_h (E_h-E_l) / L_G) / η_e = 4.13 ft/day.
- Groundwater travel time (T) T = L_G / V = 0.4 years or 145 days.

Table 2.4.13-202 shows the source term activity concentration remaining after 145 days of decay from the initial activity concentrations in DCD Table 11.2-17. As shown in Table 2.4.13-202, some of the isotopes are at or below the ECLs. Therefore, any dilution will reduce these concentrations well below the ECLs. From Table 2.4.13-202, the primary radioisotopes of consideration are H-3, Fe-55, Co-58, Co-60, Sr-90, Cs-134, and Cs-137, which are typically the primary radioisotopes contributing to groundwater contamination.

2.4.13.5.2 Modeling Equations Used in the Tank Failure Analysis

Figure 2.4.13-202 diagram depicts the simple process equations used in modeling the source term activity flow, dilution effects and mixing once the source term activity infiltrates into SCR from the groundwater. The governing differential equations for the time-dependent activity in each compartment are the following:

$$\frac{dA_A}{dt} = S_A + F_{CW,B}[A_B(t)] - (\lambda + F_{env,A} + [F_{CW,A} - F_{env,A}])[A_A(t)] \quad \text{Eq. 1}$$

$$\frac{dA_B}{dt} = -(\lambda + F_{CW,B})[A_B(t)] + ([F_{CW,A} - F_{env,A}])[A_A(t)] \quad \text{Eq. 2}$$

Where:

$F_{CW,i}$ = Normalized circulation water flow for Units 1 and 2 for compartment “i” [1/hr], defined as $F_{CW,i} = F_{CW}/V_i$

$F_{CW,i}$ = Circulation water flow for Units 1 and 2 [gallon/h]

$F_{env,\ i}$ = Normalized flow to the environment for compartment “i” [i/hr], defined as $F_{env,i} = F_{env}/V_i$

F_{env} = Flow to the environment [1/hr],

λ = Decay coefficient [1/hr],

S_A = Constant source for compartment A [μCi/hr], and

A_i = Activity in compartment “i” [μCi].

The following assumptions are included in this model:

- The source term activity infiltration rate into SCR is assumed to be constant.
- The flow to the environment is negligible (conservative for concentration calculations because it retains all of the activity in SCR).
- Only long-lived isotopes are considered; therefore, radioactive decay is neglected prior to the source term being completely infiltrated into SCR.
- SCR is at constant level (no significant changes in volume due to rainwater or other water sources being added provides conservatism because it retains the activity in SCR).
- Following the release of all the source term, the concentration decreases with time due to mixing with the large SCR bulk volume available for dilution (1.73E10 based upon the CW discharge volume plus the recirculation volume in SCR).

Using these assumptions, the equations simplify to:

$$\frac{dA_A}{dt} = S_A - (F_{CW,A})[A_A(t)] \quad \text{Eq. 3}$$

$$\frac{dA_B}{dt} = F_{CW,A}[A_A(t)] \quad \text{Eq. 4}$$

The SCR mixing volume (Volume “SCR_A”) while the source is being added becomes:

$$A_A(t) = A_A(t = 0)e^{-(F_{CW,A})t} + S_A\left[\frac{1 - e^{-(F_{CW,A})t}}{F_{CW,A}}\right] \quad \text{Eq. 5}$$

Because the activity is deposited in the SCR bulk volume, the source is assumed to be constantly added to the volume over the release period. No activity from the tank is assumed to be present in SCR prior to the event; therefore, the final equation during the release phase becomes:

$$A_A = S_A\left[\frac{1 - e^{-(F_{CW,A})t}}{F_{CW,A}}\right] \quad \text{Eq. 6}$$

Based on the above simplified equation, as time progresses, the equilibrium concentration simplifies to:

$$A_{A,eq} = \frac{S_A}{F_{CW,A}} \quad \text{Eq. 7}$$

Because:

$$\lim_{t \to \infty}(1 - e^{-(F_{CW,A})t}) = 1 \quad \text{Eq. 8}$$

Therefore, to calculate the maximum concentration this model Equation is used. Note that this conservatively assumes that equilibrium is achieved prior to the source being depleted.

The equilibrium concentration in compartment A can then easily be determined by:

$$C_{A,eq} = \frac{A_{A,eq}}{V_A} = \left(\frac{1}{V_A}\right)\left(\frac{S_A}{F_{CW,A}}\right) = \left(\frac{S_A}{(V_A)\left(\frac{F_{CW}}{V_A}\right)}\right) = \frac{S_A}{F_{CW}} \quad \text{Eq. 9}$$

2.4.13.5.3 Infiltration Area of Existing Fill Groundwater and Effect on Volumetric Flow Rate into SCR

Due to the hydrostatic pressure head of SCR pushing against the existing fill surface area (Figure 2.4.12-213), where the groundwater in the existing fill communicates with SCR, it is realistically expected that the groundwater

infiltration rate is much, much slower. Groundwater infiltration into SCR from existing fill would most likely occur at times when SCR hydrostatic pressure is decreasing due to a change in level or a considerable temperature change. However, to determine the actual flow infiltration to SCR would require another model and more data acquisition. As a result, the flow into SCR from the existing fill is assumed to occur at the groundwater volumetric flow rate through the existing fill. This is conservative because the groundwater flow rate through the existing fill does not have enough driving force to infiltrate at this rate when compared to the hydrostatic head of SCR. A discussion on the effect of a smaller infiltration surface area and its effect on infiltration rate and dilution in SCR follows.

The existing fill material is an irregular surface. However, the cross sections (Figure 2.4.13-203 and 2.4.13-204) reveal that it is roughly equivalent to one-half of a reposed conical shape with an elliptical base. Therefore, the fill volume below 770 ft msl was conservatively calculated as one-half the volume of an elliptical-based cone with basal surface area twice that of the calculated infiltration area from cross section 3c and a length equivalent to the distance of the farthest existing fill base at 770 ft msl (Figure 2.4.13-203). This results in a total fill volume below 770 ft msl of 6,671,033.8 cu. ft. and a total infiltration surface area of 34,854.49 sq. ft. Elevation 770 ft msl is conservatively chosen as SCR surface water level, which is the lower end of the normal SCR operating range, and provides the least amount of dilution volume and hydrostatic pressure head for the analysis.

Multiplying the total fill volume and infiltration area by the effective porosity of 0.2 yields a groundwater volume of approximately 9.98 million gallons and an effective infiltration surface area of approximately 6970.9 ft^2. This is also a conservative assumption because the slug of source term activity would have to have dispersed across this entire area for this to occur. The infiltration flow rate of groundwater into SCR is given by:

F_{GW} – flow rate of contaminated groundwater to SCR

A_{GW} – Area of existing fill groundwater contribution

V_{GW} – Velocity of groundwater in existing fill

$F_{GW} = A_{GW} * V_{GW}$ = 6970.9 ft_2 * 4.13 ft/day = 28,789.8 ft3/day or 149.7 gpm

Using the volumetric flow rate of 149.7 gpm as the infiltration rate into SCR is extremely conservative inasmuch as this was based upon the entire half-elliptical cone surface infiltration area of 6970.9 ft^2, which would have required the source term activity to disperse and dilute throughout the existing fill for this to occur. Using this volumetric flow rate is also conservative because the SCR hydrostatic head is much greater resulting in very little actual infiltration into SCR.

The source term activity, however, is assumed to move as a slug through the existing fill where it would not readily disperse over the entire surface area of the half elliptical cone base. If only the effective surface area of the BAT is considered as infiltration area, the resulting infiltration rate is much slower and longer time to flow into SCR.

The surface area for the BAT is based upon DCD general arrangement drawing Figure 1.2-29 that shows a BAT diameter of approximately 19 feet. Actual dimensions of the BAT have not been designated; however, using an approximate 19 foot diameter tank top or bottom is a close approximation of actual dimensions of the top or bottom of the BAT. Thus, the surface area is $\pi d^2/4$ = 283.5 ft^2, and can be used to demonstrate the slug surface area form traveling in the existing fill groundwater from the engineered fill.

$F_{GW} = A_{GW} * V_{GW}$ = 283.5 ft^2 * 4.13 ft/day = 1179.1 ft^3/day or 6.12 gpm

The source term slug flow rate into SCR is 24 times slower than the half-elliptical cone infiltration rate of 149.7 gpm where the source term is dispersed across the entire existing fill surface area.

This demonstrates that with the time it takes a smaller surface area of source term activity mixed with the groundwater to flow into SCR, a portion of the activity will combine with the recirculating water flow back to the intake through SCR, providing a much greater dilution volume. It also demonstrates that choosing a high volumetric flow rate as the infiltration rate into SCR is very conservative because this infiltration rate would be indicative of the source term activity dispersing, mixing and diluting with the entire half elliptical cone surface area groundwater. Finally, using the higher infiltration rate of 149.7 gpm is very conservative considering that the actual infiltration rate into SCR is much, much slower due to the hydrostatic head difference between SCR and the existing fill.

2.4.13.5.4 Dilution Effect of the Existing Fill Groundwater

Because a dispersion model with additional groundwater and soil data would need to be taken to predict the dilution, retardation and retention effects of the existing fill groundwater, only 25 percent of the total amount available is conservatively credited in the tank failure analysis. It is reasonable to credit 25 percent of the existing fill groundwater because the source term activity has been conservatively assumed to be moving as a slug through the engineered fill before it reaches the existing fill with no credit taken for dilution, retardation, retention or dispersion. Once the source term activity reaches the existing fill, it will disperse, mix with and be diluted by some of the existing fill groundwater. As discussed in Subsection 2.4.13.5.3, due to the hydrostatic head difference between the existing fill and SCR, there is a considerably longer stay time in the existing fill groundwater before it would infiltrate into SCR, thus allowing for greater dilution, retardation and dispersion of source term activity. The dilution effect of crediting various quantities of existing fill groundwater is provided in Table 2.4.13-204.

Using the concentration of each radioisotope from the effects of just 25 percent dilution from the existing fill groundwater gives the source term activity concentration into SCR for the conservatively larger infiltration area rate of 149.7 gpm (Table 2.4.13-205).

When it is realistically assumed that some (25 percent) groundwater dilution, retardation and retention occurs, the total activity takes 16666.67 min (277.78 hrs or 11.6 days) to infiltrate into SCR. This conservative infiltration rate for groundwater infiltration over the one-half elliptical shape shows that the infiltration is not instantaneous, that there is some expected retardation and retention by the existing fill groundwater, and that over the 11.6 days to completely infiltrate into SCR, a portion of the activity would be combined with the recirculation flow back to the CW intake; thus, a larger SCR water volume could be credited for the recirculation flow (Figure 2.4.13-206).

2.4.13.5.5 Effects of Circulating Water Pump Operation on Mixing and Dilution

Based upon the simplified Equation 9 in Subsection 2.4.13.5.2, the small dilution effect of Units 1 and 2 CW pumps at maximum capacity (2.0E06 gpm) or one Unit's CW pumps operating at maximum capacity (1.0E06 gpm) reduces the source term activity below the ECLs (Table 2.4.13-206).

The 25 percent dilution effect of the total available existing fill groundwater, with the higher infiltration rate into SCR (149.7 gpm), mixing with the CW intake at 2.0E06 gpm or 1.0E06 gpm, demonstrates that the ECLs are met. The Summation (∑) of the total activity concentration as a ratio of the ECL < 1.0 is shown in Table 2.4.13-207 for the 149.7 gpm infiltration flow rate into SCR from existing fill groundwater for maximum CW pump operation (2.0E06 gpm).

Where:

∑ (Concentration Nuclide / ECL Nuclide) < 1.0

∑ (Concentration Nuclide / ECL Nuclide) = 3.2E-01 at the 149.7 gpm infiltration rate is well below 1.0 for all CW pumps operating at 2.0E06 gpm.

For the case of half-flow CW pumps operating at maximum capacity (1.0E06 gpm), the ratio of activity concentration to the ECL is provided in Table 2.4.13-208.

∑ (Concentration Nuclide / ECL Nuclide) = 6.43E-01 at the 149.7 gpm infiltration rate is well below 1.0 for half the CW pumps operating at 1.0E06 gpm.

2.4.13.5.6 Dilution Effect and Mixing of SCR

Once the source term activity infiltrates into SCR through the existing fill (calculated to be approximately 145 days), the source term activity will enter SCR and be drawn into the CPNPP Units 1 and 2 CW intake pumps and discharged to

the south side of the CPNPP Unit 1 and 2 peninsula at 2 million gpm, where it will eventually encounter the Roto-cone drain to SCR spillway. Because the Roto-cone gravity flow device constantly discharges water to Squaw Creek and ultimately to the Brazos River in order to meet the TCEQ Term Permit CP-20 described previously, the limiting case for dilution becomes when both CPNPP Units 1 and 2 are in operation and the CW pumps are running at 2 million gpm (greatest driving force with least amount of time to reach the Roto-cone gravity flow device). Therefore, the CW discharge point becomes the location for highest source term concentration prior to dilution by SCR discharge volume. The entire 11.6 days release duration is irrelevant because some source term activity would combine with recirculating water back to the CW intake (greater dilution volume) and some activity could potentially reach the Roto-cone and be released to the environment during the first minute. Both CW pumps fully operating provides the greatest driving force and sufficient mixing for the contamination to reach the Roto-cone in the shortest time.

The flow from the CW pumps will potentially reach the Roto-cone fairly rapidly and only be diluted (11,217 ac-ft. or 3.66E09 gallons) by the effect of the small CW intake volume plus the discharge CW volume on the opposite side of the peninsula (Figure 2.4.13-205). The CPNPP Units 1 and 2 CW pumps provide a strong driving and mixing force for the dilution of the source term activity. No-flow conditions are also examined due to the possibility of CPNPP Units 1 and 2 eventually being decommissioned during the life of CPNPP Units 3 and 4, or both CPNPP Units 1 and 2 in an outage. As shown on Figure 2.4.13-206, no water volume in the inlet areas, intake area or the discharge area is included. A detailed flow model of SCR has not been performed. Thus, only an estimate of this water volume can be attributed to recirculation flow from CW discharge to CW intake.

SCR volume was calculated using bathymetry data from a July 11, 2007 bathymetry study (Reference 2.4-297). If the CW pumps were not operating at full capacity or one unit was down, there would be a lower driving force to reach the Roto-cone, and a greater volume of water to dilute the source term activity due to the recirculating water volume east of the existing fill area of SCR plus the water volume north of the Roto-cone plus the discharge point on the south side of the peninsula. This would result in dilution of the source term concentration well below the ECLs prior to discharge at the Roto-cone (Figures 2.4.13-205 and 2.4.13-206).

The mixing volume for half-flow operations is the mixing volume shown on Figure 2.4.13-207, Area 1 (11,217 ac-ft. or 3.66E09 gallons) plus the mixing volume from Area 3 (41,757 ac-ft. or 1.36E10 gallons) for a total of 1.73E10 gallons. This volume does not include depths in SCR greater than 66 feet. This is a conservative assumption because some mixing would most likely occur at greater depths in SCR, depending on the CPNPP Units 1 and 2 operating conditions, depth in SCR, seasonal fluctuations, rain events or other conditions that effect temperature changes in SCR. As a result, no credit is taken for water dilution at El. 704 ft. or deeper. The volume does not include any contribution from inlets or areas where it is expected that CW discharge would not have a credible effect on

diffused dilution or mixing. Recirculation flow time to the intake is unknown and depends on CW flow rate, SCR level, time of year, where in the fuel cycle the unit is operating, and other parameters. However, using the CW pumps in full operation provides the greatest driving force and allows for a simple estimate of the recirculation time:

1.73E10 gal / 2E06 gpm = 8635 min or 143.92 hours or 6 days recirculation time

The time for complete source term activity infiltration into SCR from existing fill is 11.6 days, which is greater than the recirculation flow time. Therefore, additional SCR dilution volume from CW recirculation flow (Figure 2.4.13-206) can be credited.

For no-flow conditions (Figure 2.4.13-206), the source term activity would diffuse with the water volume east of the existing fill and very slowly diffuse southward toward the Roto-cone release point because the Roto-cone discharge rate to Squaw Creek would be the only driving force in this scenario. Using the bathymetry study described previously, an estimated volume of SCR water at no-flow conditions is 41,757 ac-ft. or 1.36E10 gallons. This volume does not include inlet areas close to the existing fill release point, nor does it include depths greater than 66 ft. in SCR where it is not expected that much mixing or diffusion will occur. Additionally, it is unknown how long it would take the diffused source term water volume to flow southward towards the Roto-cone release point.

No-flow conditions would result in the source term activity infiltrating SCR via the existing fill groundwater interface and slowly diffusing into the SCR water adjacent to the east side of CPNPP Unit 3. As shown on Figure 2.4.13-206, no water volume in the inlet areas or intake area is included. The credited volume as discussed previously is 1.36E10 gallons and does not include any water below a depth of 66 feet in the reservoir. The infiltration rate into SCR is discussed previously, but in this case is irrelevant as diffusion throughout SCR surface water would be very slow. The only driving force to reach the Roto-cone area is the discharge through the Roto-cone. An additional model would have to be developed to calculate the diffusion rate of source term activity into SCR and the time to reach the Roto-cone. However, Table 2.4.13-209 shows that the ECLs would be met before any contamination reached the Roto-cone by simple diffusion with the SCR surface water above the 66 ft depth. In this case, no credit is taken for dilution effect of existing fill groundwater. If credit were taken, the resulting ratio of activity to ECL would be further diminished as demonstrated in Subsection 2.4.13.5.5.

$\sum$ (Concentration Nuclide / ECL Nuclide) = 7.87E-01

2.4.13.5.7 Summary

Considerable conservative assumptions include:

- No credit taken for the dilution, retardation or retention effects of the engineered fill;
- No credit taken for the travel time through the engineered fill that is assumed to be completely saturated;
- The source term activity moves as a slug volume through both the engineered fill and existing fill;
- The infiltration rate into SCR is one-half elliptical cone surface area of the existing fill (149.7 gpm). This flow rate is excessive when compared to actual very slow infiltration into SCR resulting from a decrease in hydrostatic head between SCR and the adjacent existing fill surface area in communication with SCR;
- Crediting only 25 percent of the existing fill groundwater when actually there would be greater dispersion, dilution and retention in the groundwater.
- Using the surface area of the one-half elliptical cone existing fill volume demonstrates that there would have to be greater dispersion in the groundwater; and
- For the limiting case, crediting only the 2 million or 1 million gpm mixing and dilution flow of CW intake when further dilution will occur based upon the CW discharge volume prior to reaching the Roto-cone release point.

Additionally, it has been adequately demonstrated that a smaller infiltration flow rate from the existing fill into SCR results in a longer time for the total activity to infiltrate into SCR. This longer infiltration time (11.6 days) ensures a larger dilution volume because some of the source term activity will combine with recirculation flow and be diluted by the bulk volume of SCR. Furthermore, it has been demonstrated that adequate mixing occurs in SCR using the mixing driving force of the CW pumps only. For no-flow pump conditions, it is demonstrated that simple diffusion and dilution by SCR surface water is adequate to meet the ECLs for the case of either a Unit 3 or 4 tank failure without crediting existing fill groundwater dilution.

Crediting 25 percent of the existing fill groundwater for dilution of the source term activity prior to entering SCR, combined with the slow infiltration effect of the existing fill groundwater into SCR, and only the mixing and dilution effect of the CW intake of either 1 or 2 million gpm results in meeting the ECLs for all radioisotopes that infiltrate into SCR via the existing fill groundwater. The unrestricted potable water supply receptor location is the Roto-cone discharge area in the southeast portion of SCR near the Squaw Creek dam. All activity concentrations reaching the Roto-cone device have been shown to be below the limits of 10 CFR 20, Appendix B, Table 2, and thus the requirements of 10 CFR 20.1301, 20.1302 and 10 CFR 100 are satisfied.

2.4.14 Technical Specifications and Emergency Operation Requirements

CP COL 2.4(1) Add the following after the paragraph in DCD Subsection 2.4.14.

The grade elevation of CPNPP Units 3 and 4 is above the probable maximum flood (PMF) elevation; therefore, due to plant grade elevation and the unique "always in place" four tank design of the UHS there are no requirements for emergency protective measures designed to minimize the impact of adverse hydrology-related events on safety-related facilities, and none are incorporated into the technical specifications or emergency procedures.

2.4.15 Combined License Information

CP COL 2.4(1) Add the following at the end of DCD Subsection 2.4.15.

2.4(1) Hydrologic Related Events

This COL item is addressed in Subsections 2.4.1, 2.4.2, 2.4.3, 2.4.4, 2.4.5, 2.4.6, 2.4.7, 2.4.8, 2.4.9, 2.4.10, 2.4.11, 2.4.12, 2.4.13 and 2.4.14 along with the associated tables and figures.

2.4.16 References

CP SUP2.4(1) Add the following references after the last DCD reference.

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Table 2.4.1-201
Safety-Related Facility and Access Elevations

	Elevation	
Structure	Opening	Grade
Reactor Building		822.0
Tendon Gallery Access Hatch	823.0	
Track Bay Area Door	823.0	
West Area Door to the Turbine Building	823.0	
East Area Door to the Turbine Building	823.0	
Power Source Building		822.0
Access Hatch to the West Power Source Building	823.0	
Auxiliary Building		822.0
Door to the Access Building	823.0	
Track Bay Area Door	823.0	
Turbine Building		822.0
East Truck Bay Area Door	823.0	
North Corridor Access	823.0	
South Corridor Access	823.0	

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Table 2.4.1-202 (Sheet 1 of 3)
Temperature Measurements for Lake Granbury

Waypoints	Surface Temperature (°F)	1-10 ft Temperature (°F)	11-20 ft Temperature (°F)	21-30 ft Temperature (°F)	31-40 ft Temperature (°F)	41-50 ft Temperature (°F)
117	69.3	68.2	65.3	63.3	61.7	61.6
118	69.7	68.2	65.7	64.1	61.9	61.2
119	69.8	67.7	66.2	64.7	62.4	60.9
120	68.4	67.3	67	U/O	U/O	U/O
121	69.5	67.1	66.6	U/O	U/O	U/O
122	69.4	67.2	65.9	64.8	62.2	61.1
123	69.2	67.9	65.7	63.5	62.5	61.9
124	69.8	69	65.8	64	U/O	U/O
125	69.8	67.7	65.8	63.7	62	U/O
126	69.7	67.4	65.9	65.4	62.4	61.2
127	69.2	66.7	U/O	U/O	U/O	U/O
128	69.8	67.2	65.7	65.1	62.6	U/O
129	70	67.8	65.8	65.1	62.8	61.3

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Table 2.4.1-202 (Sheet 2 of 3) Temperature Measurements for Lake Granbury

Waypoints	Surface Temperature (°F)	1-10 ft Temperature (°F)	11-20 ft Temperature (°F)	21-30 ft Temperature (°F)	31-40 ft Temperature (°F)	41-50 ft Temperature (°F)
130	69.8	68	65.5	64	U/O	U/O
131	69.8	67.8	65.6	65.4	62.7	61.3
132	69.7	67.3	65.8	U/O	U/O	U/O
133	69.1	66.8	66.2	U/O	U/O	U/O
134	69.8	67.8	65.6	65	63.1	61.1
135	70.1	69.5	65.7	64.1	U/O	U/O
136	70.2	68.7	65.7	65.3	63	61.8
137	69.2	67.2	65.5	U/O	U/O	U/O
138	69.9	68.5	65.5	65.2	63.1	61.2
139	70.2	68.8	65.7	65.5	64.1	61.8
140	70.2	68.9	65.5	65.6	U/O	U/O
141	70.1	67.7	U/O	U/O	U/O	U/O
142	70	68.8	66.4	U/O	U/O	U/O

CP COL 2.4(1)

Table 2.4.1-202 (Sheet 3 of 3)
Temperature Measurements for Lake Granbury

Waypoints	Surface Temperature (°F)	1-10 ft Temperature (°F)	11-20 ft Temperature (°F)	21-30 ft Temperature (°F)	31-40 ft Temperature (°F)	41-50 ft Temperature (°F)
143	71.1	69.2	65.9	66.4	63.4	61.8
144	71.1	66.8	65.5	65.2	U/O	U/O
145	71.4	70.2	U/O	U/O	U/O	U/O
146	71.5	69.3	66	66.8	63.1	62.1
Average Temperature	69.89	68.02	65.83	64.87	62.69	61.45

Notes:

Waypoint locations illustrated on Figure 2.4.1-204

U/O - unobtainable due to shallow depth or water velocity

Temperature measurements acquired using a Cline Finder Digital Thermometer from the surface to 50 feet (ft), with an accuracy range of ±0.5 degrees Fahrenheit (°F) over the operating range

Average Temperature 66.24 (°F)

Temperature measured May 2, 2007

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Table 2.4.1-203 (Sheet 1 of 2)
Dam and Reservoir Information

Reservoir Name	Possum Kingdom Lake	Lake Palo Pinto	Lake Mineral Wells	Lake Granbury	Squaw Creek Reservoir	Wheeler Branch Reservoir	Lake Whitney
Dam Name	Morris Sheppard Dam	Palo Pinto Creek Dam	Mineral Wells Dam	DeCordova Bend Dam	Squaw Creek Dam	Wheeler Branch Dam	Whitney Dam
Owner	Brazos River Authority	Palo Pinto MWD No. 1	City of Mineral Wells	Brazos River Authority	TXU Generation Co. LP	Somervell County Water District	Corps of Engineers-SWF
Dam Length (Feet)	2740	1255	1650	2200	4360	1750	17,695
Dam Top Elevation (Feet msl	1024	898	873.9	706.5	796.0	NR	584
Elevation at Top of Flood Pool (Feet msl)	NA	NA	NA	NA	NA	NA	571
Elevation at Top of Conservation Pool (Feet MSL)	1000	867	863	693	775	785	533
Dead Pool Elevation (Feet msl)	874.8	835	nr	640	653	NR	448.83
Elevation at Bottom of Lake (Feet msl)	870	815	817	628	648.2	NR	429
Flood Pool Capacity (Acre-Feet)	NA	NA	NA	NA	NA	NA	2,000,204
Conservation Pool Capacity Original (Acre-Feet)	724,739	44,100	6760	153,500	151,047	4,118	627,100
Conservation Pool Storage Survey (Acre-Feet)	540,340	27,650	7065	129,011	151,418	NR	554,203
Storage at Dead Pool Capacity (Acre-Feet)	236	500	NR	965	51	NR	859
Surface Area at Top of Conservation Pool Original (Acre)	19,800	2498	646	8700	3228	180	23,560
Surface Area at Top of Conservation Pool Survey (Acre)	16,714	NR	440	7945	3297	NR	23,220
Last Survey Date	Jun 2005	NR	Jul 1992	July 2003	May 1997	NR	June 2005
Drainage Area (Square Miles)	22,550	471	63	25,679	64	NR	27,189
Main Purposes	water supply, hydroelectric, irrigation, Mining, Industrial	water supply	water supply	water supply, irrigation, industrial, mining	industrial, recreation	water supply	flood control, water supply, hydroelectric

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Table 2.4.1-203 (Sheet 2 of 2)
Dam and Reservoir Information

Reservoir Name	Possum Kingdom Lake	Lake Palo Pinto	Lake Mineral Wells	Lake Granbury	Squaw Creek Reservoir	Wheeler Branch Reservoir	Lake Whitney
Dam Name	Morris Sheppard Dam	Palo Pinto Creek Dam	Mineral Wells Dam	DeCordova Bend Dam	Squaw Creek Dam	Wheeler Branch Dam	Whitney Dam
Owner	Brazos River Authority	Palo Pinto MWD No. 1	City of Mineral Wells	Brazos River Authority	TXU Generation Co. LP	Somervell County Water District	Corps of Engineers-SWF
Year of Completion	1941	1964	1920	1969	1977	2007	1951
Stream	Brazos River	Palo Pinto Creek	Rock Creek	Brazos River	Squaw Creek	Wheeler Branch	Brazos River
County	Palo Pinto	Palo Pinto	Parker	Hood	Somervell, Hood	Somervell	Hill, Bosque
Nearest Town	Graham	Mineral Wells	Mineral Wells	Granbury	Glen Rose	Glen Rose	Whitney
Direction to Nearest Town	11.3 miles NE	15 miles SW	4 miles E	8 miles NW	4 miles N	2 miles SSE	5.5 miles SW
Water Planning Region	G	G	G	G	G	G	G
Dam Central Latitude	32.87	32.6467	32.8167	32.3733	32.2883	NR	NR
Dam Central Longitude	-98.425	-98.2683	-98.0417	-97.6883	-97.76	NR	NR
Reservoir Gage	8088500	8090300	8090700	8090900	8091730	NR	8092500
Upstream USGS Streamflow Gage	8088000	NR	NR	8090800	8091730	NR	8091000
Downstream USGS Streamflow Gage	8088610	NR	NR	8091000	8091750	NR	8093100
Major Water Rights	C5155	C4031	C4039	C5156	C4097	NR	C5157

NA - Not Applicable

NR - Not Reported

Sources:
(Reference 2.4-206), (Reference 2.4-209), (Reference 2.4-212), (Reference 2.4-264), (Reference 2.4-265), (Reference 2.4-266)

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Table 2.4.1-204
Major Tributaries Contributing Flow to Brazos River Between Morris Shepherd Dam and DeCordova Bend Dam

Contributing Flow	Drainage Area (mi^2)	Gradient (ft/mi)	Slope Percentage	Length (mi)	Brazos River Mile (BRM)	Tributaries
Palo Pinto Creek	461	12.17	0.23	60.0	609.5	Palo Pinto Creek - South Fork, Lake, Gibson, Barton, Little Sunday, Big Sunday, Lost, and Buck Creeks
Rock Creek	63	21.67	0.41	24.0	599.7	Wilson Creek, Dry Creek, Moreland Creek, Rippy Branch, and Grassy Branch

Note:

Location, length, and slopes of streams calculated from USGS Topographic Maps and information from the TSHA.

CP COL 2.4(1)

Table 2.4.1-205
Local Stream Tributaries

Contributing Flow	Drainage Area (mi^2)	Gradient (ft/mi)	Slope Percentage	Length (mi)	Reservoir Fed
Lusk Branch	Unknown	65.55	1.241	2.38	Lake Granbury
Walnut Creek	Unknown	46.94	0.889	7.84	Lake Granbury
Contrary Creek	Unknown	76.83	1.455	5.87	Lake Granbury
Rough Creek	Unknown	74.65	1.414	3.67	Lake Granbury
Lambert Branch	Unknown	48.02	0.909	4.79	Lake Granbury
Rucker Creek	461	33.95	0.643	12.49	Lake Granbury
Squaw Creek	64	25.23	0.478	12.96	SCR
Panter Branch	Unknown	42.44	0.804	7.47	SCR
Lollar Branch	Unknown	46.03	0.872	4.91	SCR
Panther Branch	Unknown	60.08	1.138	2.43	SCR
Million Branch	Unknown	53.50	1.013	3.29	SCR
Unnamed Stream	Unknown	89.51	1.695	2.67	SCR

Note:

Stream lengths and gradients measured from headwaters to normal pool elevation of the receiving reservoir using: All Topo Maps: Texas V6 Professional Map Reference Set, iGage Mapping Corp.

CP COL 2.4(1)

Table 2.4.1-206
Lake Granbury Municipal Water Systems

Public Water System	Use	Population Count	Average Daily Consumption
Oak Trail Shores	Municipal	6354	0.362 Mgd
City of Granbury[(a)]	Municipal	See Note	See Note
Action Municipal Utility District[(a)]	Municipal	See Note	See Note
Johnson County Fresh Water Supply District No. 1[(a)]	Municipal	See Note	See Note
Johnson County Special Utilities District[(a)]	Municipal	See Note	See Note

a) Treated Water Provided by the Lake Granbury Surface Water and Treatment System (SWATS)

Note: SWATS Total Population Count = 60,692, Total Average Daily Consumption = 5.360 million gallons per day (Mgd)

(Reference 2.4-215), (Reference 2.4-268)

CP COL 2.4(1)

Table 2.4.1-207
Annual Water Use - Brazos River Basin (Acre-Feet)

Year	Municipal	Manufacturing	Steam Electric	Irrigation	Mining	Livestock	Total
2004	399,847	228,739	160,944	2,661,345	24,718	69,292	3,544,885

(Reference 2.4-216)

CP COL 2.4(1)

Table 2.4.1-208
2006 Area Surface Water Withdrawals (Acre-Feet)

County	User Name	Stream Name	Use Type	January	February	March	April	May	June	July	August	September	October	November	December	Total
Palo Pinto	Brazos River Authority	Brazos River	1,2,3,4,5,6	4852	1761	4657	7436	32,815	12,630	24,703	21,601	1298	30,973	11,882	5703	160,311
Palo Pinto	Palo Pinto MWD 1	Palo Pinto Creek	1	365	288	322	366	416	497	561	577	385	377	323	324	4800
Palo Pinto	Rocking W Ranch, LP	Brazos River	3	0	0	0	0	18	0	217	231	133	47	0	0	647
Palo Pinto	W.J. Rhodes	Brazos River	3	0	0	0	0	0	8	10	5	0	0	0	0	23
Parker	City of Mineral Wells	Rock Creek	1	0	2	0	0	0	6	19	27	0	0	0	0	54
Parker	TXI Operations, LP	Brazos River	2,3	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Hood	Brazos River Authority	Brazos River	1,2,3,4	1542	2769	2966	5399	5410	6775	7155	7710	6771	5574	4134	610	56,815
Somervell	TXU Electric	SCR, Panther Branch, Lake Granbury	2	227,102[(a)]	210,025[(a)]	269,807[(a)]	296,577[(a)]	305,253[(a)]	297,050[(a)]	306,579[(a)]	306,898[(a)]	297,130[(a)]	253,082[(a)]	295,190[(a)]	303,111[(a)]	3,367,805[(a)]
Somervell	Somervell County Water District	Paluxy River	1,2,3	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Bosque	City of Clifton	North Bosque River	1	63	8	65	38	0	26	0	0	0	9	28	21	256
Bosque	City of Meridian	North Bosque River	1	0	0	0	0	0	0	0	0	0	0	0	0	0
Bosque	Chisholm Trails Ventures, LP	Brazos River	3	141	141	345	345	576	576	576	435	345	141	0	0	3621
Bosque	Lakeview Recreation Association Inc.	Brazos River, Rock Branch	3	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Bosque	John McPherson	Brazos River	3	0	0	0	0	0	35	35	35	35	0	0	0	140
Bosque	Smith Bend Ranch, Ltd.	Brazos River	3	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Hill	Brazos River Authority	Brazos River	1,2	109	107	122	113	120	140	184	229	5854	137	69	118	7302

a) Includes CPNPP Units 1 and 2 once though cooling.

<u>Notes</u>:

Use Types

1 - Municipal
2 - Industrial
3 - Irrigation
4 - Mining
5 - Hydroelectric
6 - Other

NR - Not Reported

CP COL 2.4(1)

Table 2.4.1-209
CPNPP Water Well Information

CPNPP Well ID	State Well Number	Location	Primary Use	Well Depth (ft)	Latitude	Longitude	Well Type
1	3242903	Ball Bark Road	Not Used	479	321651	974623	Observation
2	3242902	Training Center	Not Used	318	321707	974515	Observation
3	3242901	Training Center	Public Supply	350	321707	974516	Withdrawal of Water
4	3242601	Batch Plant	Public Supply	466	321748	974733	Withdrawal of Water
5	3242602	Met Tower	Public Supply	490	321750	974650	Withdrawal of Water
6	N/A	Plant Entrance	Not Used	>280(1)	321749	974859	Observation
7	3242503	NOSF - North	Public Supply	517	321760	974828	Withdrawal of Water
8	3242504	NOSF - South	Public Supply	400	321757	974826	Withdrawal of Water
9	3242603	Squaw Creek Park	Public Supply	471	321905	974659	Withdrawal of Water
10	3242604	Squaw Creeak Park	Not Used	470	321905	974660	Observation
11	N/A	Squaw Creek Park Office	Public Supply	Unknown(2)	321946	974648	Withdrawal of Water
12	N/A	Rifle Training Facility	Public Supply	485	321905	974659	Withdrawal of Water

Notes:

Onsite water wells are owned by Luminant and completed in the Twin Mountains (Trinity) Aquifer
(1) Total depth of well is unknown due to obstruction. Static water level has been measured at approximately 280 ft below top of casing.
(2) Inactive public supply well, total depth of well is unknown.
NOSF Nuclear Operations Support Facility
N/A Not Assigned

CP COL 2.4(1)

Table 2.4.1-210
2006 CPNPP Monthly Groundwater Use

Month	Self Supplied (Gallons)
January	835,600
February	759,800
March	1,050,700
April	904,400
May	688,300
June	762,600
July	697,500
August	679,000
September	628,500
October	930,000
November	568,800
December	587,500
Total	9,092,700

Source: (Reference 2.4-217)

CP COL 2.4(1)

Table 2.4.2-201 (Sheet 1 of 4)
Peak Streamflow of the Brazos River near Glen Rose, Texas
(USGS Station 08091000) 1923–2006

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1924	10/17/1923	13.00	37500
1925	5/8/1925	15.10	45700
1926	6/21/1926	13.20	38300
1927	10/19/1926	14.00	41400
1928	5/20/1928	10.40	27700
1929	9/12/1929	13.42	38400
1930	6/17/1930	19.60	68300
1931	10/7/1930	12.18	31700
1932	9/10/1932	16.37	49300
1933	5/27/1933	13.19	36600
1934	3/4/1934	4.11	5240
1935	5/18/1935	23.68	97600
1936	9/27/1936	19.42	67300
1937	6/9/1937	9.93	22200
1938	3/29/1938	15.12	45200
1939	6/23/1939	9.85	22600
1940	8/19/1940	13.62	38300
1941	11/25/1940	14.90	44200
1942	4/26/1942	19.23	66400
1943	10/18/1942	17.47	54100
1944	5/2/1944	10.21	24100
1945	3/30/1945	13.85	39200

CP COL 2.4(1)

Table 2.4.2-201 (Sheet 2 of 4)
Peak Streamflow of the Brazos River near Glen Rose, Texas
(USGS Station 08091000) 1923–2006

Water Year[(a)]	Date	Gage Height[(b)] (ft)	Discharge (cfs)
1946	9/27/1946	8.24	11500
1947	12/12/1946	16.89	38900
1948	2/25/1948	8.68	12500
1949	5/17/1949	26.70	74000
1950	7/28/1950	11.92	20700
1951	6/18/1951	5.05	5680
1952	5/24/1952	14.19	27900
1953	5/17/1953	5.21	5920
1954	5/15/1954	17.34	25600
1955	9/30/1955	19.74	42300
1956	10/9/1955	15.78	30600
1957	5/27/1957	33.89	87400
1958	5/2/1958	21.00	36100
1959	7/8/1959	11.50	8900
1960	10/5/1959	28.10	65500
1961	6/19/1961	16.80	21700
1962	7/29/1962	25.32	50500
1963	4/30/1963	13.37	13100
1964	9/22/1964	11.01	8110
1965	5/20/1965	17.43	23500
1966	5/3/1966	25.90	49800
1967	7/22/1967	14.19	15000

CP COL 2.4(1)

**Table 2.4.2-201 (Sheet 3 of 4)
Peak Streamflow of the Brazos River near Glen Rose, Texas
(USGS Station 08091000) 1923–2006**

Water Year[(a)]	Date	Gage Height[(b)] (ft)	Discharge (cfs)
1968	3/21/1968	19.01	28400
1969	5/9/1969	21.20	35700
1970	12/30/1969	16.65	21300
1971	9/2/1971	12.66	11400
1972	10/20/1971	13.05	12200
1973	4/23/1973	13.61	13600
1974	10/15/1973	11.94	9190
1975	11/1/1974	25.42	46800
1976	5/26/1976	15.20	16000
1977	3/27/1977	25.88	48500
1978	8/11/1978	24.70	41200
1979	5/4/1979	27.60	55400
1980	9/30/1980	8.28	2990
1981	10/5/1980	16.56	18100
1982	10/15/1981	35.19	86400
1983	5/24/1983	16.40	17700
1984	1/26/1984	8.14	3220
1985	1/2/1985	14.44	14200
1986	9/2/1986	12.65	10600
1987	5/29/1987	17.25	20900
1988	6/2/1988	8.49	3790
1989	5/18/1989	27.08	53300

CP COL 2.4(1)

Table 2.4.2-201 (Sheet 4 of 4)
Peak Streamflow of the Brazos River near Glen Rose, Texas
(USGS Station 08091000) 1923–2006

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1990	4/28/1990	35.76	79800
1991	6/10/1991	19.17	28300
1992	12/21/1991	34.00	89600
1993	12/14/1992	11.50	7800
1994	10/20/1993	18.10	23400
1995	8/3/1995	21.21	32200
1996	9/17/1996	17.65	22100
1997	2/22/1997	28.99	61300
1998	3/16/1998	25.80	48200
1999	3/21/1999	11.41	7650
2000	6/4/2000	17.46	21600
2001	2/17/2001	18.47	24400
2002	3/21/2002	15.18	15100
2003	9/19/2003	11.73	5170
2004	6/9/2004	25.71	42700
2005	8/25/2005	18.00	18100
2006	3/19/2006	14.88	11200

a) Water Year = October 1 to September 30

b) Water Years 1924 to 2001 Datum = 567.82 ft above sea level NGVD29
Water Years 2002 to present Datum = 561.79 ft above sea level NGVD29
(References 2.4-218 and 2.4-219)

CP COL 2.4(1)

Table 2.4.2-202 (Sheet 1 of 4)
Peak Streamflow of the Paluxy River at Glen Rose, Texas
(USGS Station 08091500) 1908–2006

Water Year[(a)]	Date	Gage Height[(b)] (ft)	Discharge (cfs)
1908	1908	27.20	59000
1918	1918	26.00	53000
1922	1922	26.00	53000
1948	2/25/1948	13.92	11000
1949	5/17/1949	25.10	48500
1950	10/24/1949	9.18	4570
1951	6/3/1951	8.80	4130
1952	5/23/1952	22.30	36200
1953	5/15/1953	8.64	3930
1954	4/12/1954	10.00	5510
1955	5/19/1955	22.50	37000
1956	5/1/1956	16.60	17300
1957	4/26/1957	24.12	44000
1958	7/6/1958	9.50	4900
1959	4/19/1959	8.22	3530
1960	10/4/1959	25.40	50000
1961	7/17/1961	8.63	4100
1962	10/9/1961	17.48	19800
1963	10/8/1962	18.23	21900
1964	4/21/1964	13.13	9960
1965	5/10/1965	12.79	9480
1966	4/30/1966	12.38	8840

CP COL 2.4(1)

**Table 2.4.2-202 (Sheet 2 of 4)
Peak Streamflow of the Paluxy River at Glen Rose, Texas
(USGS Station 08091500) 1908–2006**

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1967	7/19/1967	11.39	7240
1968	5/20/1968	15.92	15500
1969	4/17/1969	14.05	12700
1970	10/12/1969	11.97	8150
1971	5/29/1971	8.14	3740
1972	10/19/1971	14.49	12500
1973	4/24/1973	19.05	24600
1974	9/20/1974	7.09	2820
1975	10/31/1974	10.75	6450

a) Water Year = October 1 to September 30

b) Water Years 1924 to 2001 Datum = 567.82 ft above sea level NGVD29
Water Years 2002 to present Datum = 561.79 ft above sea level NGVD29
(References 2.4-220 and 2.4-225)

CP COL 2.4(1)

Table 2.4.2-202 (Sheet 3 of 4)
Peak Streamflow of the Paluxy River at Glen Rose, Texas
(USGS Station 08091500) 1908-2006

Water Year[(a)]	Date	Gage Height[(b)] (ft)	Discharge (cfs)
1976	5/25/1976	12.78	9430
1977	3/27/1977	17.48	19700
1978	5/11/1978	7.89	3520
1979	5/3/1979	22.91	38800
1980	5/14/1980	6.13	2120
1981	6/7/1981	4.11	728
1982	5/22/1982	19.15	24800
1983	5/23/1983	4.79	1150
1984	5/23/1984	6.28	2190
1985	4/29/1985	10.12	5700
1986	9/2/1986	9.14	4700
1987	5/29/1987	9.77	5340
1988	6/1/1988	9.33	4890
1989	3/28/1989	21.08	31500
1990	4/26/1990	19.38	25600
1991	6/2/1991	10.23	5820
1992	12/20/1991	21.28	32300
1993	2/25/1993	5.47	1970
1994	5/11/1994	7.76	3950
1995	7/31/1995	17.78	20600

CP COL 2.4(1)

Table 2.4.2-202 (Sheet 4 of 4)
Peak Streamflow of the Paluxy River at Glen Rose, Texas
(USGS Station 08091500) 1908-2006

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1996	8/30/1996	5.73	2140
1997	2/20/1997	14.68	13100
1998	3/16/1998	17.07	18600
1999	6/2/1999	6.99	3200
2000	6/15/2000	8.61	4790
2001	2/16/2001	8.56	4740
2002	3/19/2002	8.69	4870
2003	9/19/2003	6.54	2800
2004	6/9/2004	13.17	10500
2005	11/17/2004	4.94	1540
2006	3/19/2006	8.67	4850

a) Water Year = October 1 to September 30

b) Datum = 609.66 ft above sea level NGVD29
(References 2.4-220 and 2.4-225)

CP COL 2.4(1)

Table 2.4.2-203 (Sheet 1 of 2)
Peak Streamflow of Squaw Creek near Glen Rose, Texas
(USGS Station 08091750) 1973–2006

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1974	10/12/1973	5.42	730
1975	4/8/1975	11.90	9030
1976	5/25/1976	10.53	3170
1977	3/27/1977	6.16	1200
1978	5/11/1978	3.90	108
1979	5/3/1979	9.10	4290
1980	5/14/1980	3.89	65
1981	7/4/1981	4.44	220
1982	4/15/1982	5.23	486
1983	5/23/1983	5.17	520
1984	3/23/1984	5.31	619
1985	10/20/1984	4.75	373
1986	5/8/1986	6.30	1350
1987	6/12/1987	7.42	2230
1988	6/1/1988	4.54	309
1989	6/13/1989	11.85	8940
1990	5/3/1990	9.90	5630
1991	8/14/1991	6.52	1470
1992	12/20/1991	11.79	8820
1993	6/26/1993	3.03	71
1994	5/12/1994	3.11	76
1995	7/31/1995	6.95	1670

CP COL 2.4(1)

Table 2.4.2-203 (Sheet 2 of 2)
Peak Streamflow of Squaw Creek near Glen Rose, Texas
(USGS Station 08091750) 1973–2006

Water Year[(a)]	Date	Gage Height[(b)] (ft)	Discharge (cfs)
1996	8/30/1996	5.20	561
1997	2/20/1997	6.02	953
1998	3/16/1998	9.54	5000
1999	11/13/1998	4.87	441
2000	6/4/2000	9.09	4280
2001	10/29/2000	4.93	403
2002	3/19/2002	7.02	1730
2003	9/19/2003	3.80	145
2004	6/9/2004	10.47	2640
2005	11/24/2004	3.20	149
2006	5/6/2006	2.87	111

a) Water Year = October 1 to September 30

b) Datum = 599.00 ft msl NGVD29
(References 2.4-220 and 2.4-226)

CP COL 2.4(1)

Table 2.4.2-204
Peak Streamflow of Panter Branch near Tolar, Texas (USGS Station 08091700) 1966–1973

Water Year[a]	Date	Gage Height[b] (ft)	Discharge (cfs)
1966	4/29/1966	14.49	880
1967	5/20/1967	16.90	1650
1968	5/9/1968	21.70	3650
1969	5/7/1969	13.50	610
1970	10/11/1969	13.61	640
1971	7/29/1971	14.53	890
1972	9/16/1972	21.88	3750
1973	4/23/1973	17.72	1990
1974	10/30/1973	10.20	5

a) Water Year = October 1 to September 30

b) Datum = 883 ft msl NAVD88
(Reference 2.4-220)

CP COL 2.4(1)

Table 2.4.2-205
Hourly Rainfall Depth for PMP

Hour	Cumulative PMP (in)	Incremental PMP (in)	Hour	Cumulative PMP (in)	Incremental PMP (in)
1	19.00	19.00	37	43.88	0.19
2	22.39	3.39	38	44.07	0.18
3	24.61	2.23	39	44.24	0.18
4	26.44	1.82	40	44.41	0.17
5	28.04	1.60	41	44.58	0.16
6	29.50	1.46	42	44.74	0.16
7	30.86	1.36	43	44.89	0.16
8	32.12	1.26	44	45.04	0.15
9	33.26	1.14	45	45.19	0.15
10	34.29	1.03	46	45.33	0.14
11	35.20	0.91	47	45.47	0.14
12	36.00	0.80	48	45.60	0.13
13	36.70	0.70	49	45.73	0.13
14	37.30	0.61	50	45.85	0.13
15	37.84	0.54	51	45.98	0.12
16	38.33	0.48	52	46.10	0.12
17	38.76	0.43	53	46.21	0.12
18	39.16	0.39	54	46.32	0.11
19	39.52	0.36	55	46.43	0.11
20	39.85	0.33	56	46.54	0.11
21	40.16	0.31	57	46.64	0.10
22	40.45	0.29	58	46.74	0.10
23	40.73	0.28	59	46.84	0.10
24	41.00	0.27	60	46.94	0.10
25	41.26	0.26	61	47.03	0.10
26	41.51	0.25	62	47.13	0.09
27	41.76	0.25	63	47.22	0.09
28	42.00	0.24	64	47.31	0.09
29	42.23	0.23	65	47.40	0.09
30	42.46	0.23	66	47.49	0.09
31	42.68	0.22	67	47.57	0.09
32	42.90	0.21	68	47.66	0.09
33	43.11	0.21	69	47.75	0.09
34	43.31	0.20	70	47.83	0.09
35	43.51	0.20	71	47.92	0.08
36	43.70	0.19	72	48.00	0.08

CP COL 2.4(1)

Table 2.4.2-206
5 Minute Rainfall Depth for Local Intense PMP

Minutes	Cumulative PMP (in)	Rainfall Intensity (in/hr)	Minutes	Cumulative PMP (in)	Rainfall Intensity (in)
5	6.20	74.4	185	24.78	8.0
10	8.12	48.7	190	24.94	7.9
15	9.70	38.8	195	25.10	7.7
20	11.23	33.7	200	25.25	7.6
25	12.73	30.6	205	25.41	7.4
30	14.20	28.4	210	25.56	7.3
35	15.55	26.7	215	25.71	7.2
40	16.59	24.9	220	25.86	7.1
45	17.38	23.2	225	26.01	6.9
50	18.02	21.6	230	26.15	6.8
55	18.55	20.2	235	26.29	6.7
60	19.00	19.0	240	26.44	6.6
65	19.40	17.9	245	26.58	6.5
70	19.76	16.9	250	26.72	6.4
75	20.09	16.1	255	26.85	6.3
80	20.40	15.3	260	26.99	6.2
85	20.69	14.6	265	27.12	6.1
90	20.96	14.0	270	27.26	6.1
95	21.23	13.4	275	27.39	6.0
100	21.48	12.9	280	27.52	5.9
105	21.72	12.4	285	27.65	5.8
110	21.95	12.0	290	27.78	5.7
115	22.17	11.6	295	27.91	5.7
120	22.39	11.2	300	28.04	5.6
125	22.60	10.8	305	28.16	5.5
130	22.80	10.5	310	28.29	5.5
135	23.00	10.2	315	28.41	5.4
140	23.20	9.9	320	28.54	5.4
145	23.39	9.7	325	28.66	5.3
150	23.57	9.4	330	28.78	5.2
155	23.75	9.2	335	28.90	5.2
160	23.93	9.0	340	29.02	5.1
165	24.11	8.8	345	29.14	5.1
170	24.28	8.6	350	29.26	5.0
175	24.45	8.4	355	29.38	5.0
180	24.61	8.2	360	29.50	4.9

CP COL 2.4(1)

Table 2.4.2-207
Site Drainage Area Details

Drainage Sub Basin	Area A (ac)	Total Tc (min)	PMP Intensity I (inch/hr)	Runoff Coefficient (C)	Peak Runoff Q (cfs)
1	9.22	15.8	38.0	1.00	350.36
2	8.53	10.9	47.0	1.00	400.91
3	5.97	5.0	74.4	1.00	444.17
4	8.83	15.8	38.0	1.00	335.54
5	9.66	15.6	38.2	1.00	369.01
6	6.22	5.1	74.3	1.00	462.15
7	24.68	5.2	74.2	1.00	1,831.26
8	20.49	34.6	27.0	1.00	614.70
9	31.32	16.4	37.5	1.00	1,174.50
10	13.49	10.6	47.5	1.00	640.78
11	56.40	13.4	41.0	1.00	2,312.40

CP COL 2.4(1)

Table 2.4.2-208
Resulting PMP Water Surface Elevation at Points of Discharge

Point Of Discharge	Drainage Sub Basins	Peak Runoff at Point of Discharge (cfs)	Crest Length L (ft)	Tailwater Elevation (ft msl)	Discharge Coefficient	Weir Elevation (ft msl)	Over Topping Depth Hw (ft)	Resulting Water Surface Elevation (ft msl)
W1	1+2+3	1,195.44	560	793.66	2.50	820	0.90	820.90
W2	4+5+6	1,166.70	365	793.66	2.50	815	1.18	816.18
W3	7+8	2,384.49	490	793.66	2.50	810	1.56	811.56
W4	9+10+11	4,127.68	315	793.66	2.50	814	3.02	817.02

CP COL 2.4(1)

Table 2.4.3-201
Watershed PMP (in) Depth-Area-Duration Relationship

Area	Duration (hr)				
(sq mi)	6	12	24	48	72
10	29.7	35.3	40.0	45.0	48.0
200	22.2	26.8	32.0	36.0	39.6
1000	15.9	20.7	25.8	30.0	33.4
5000	9.3	13.1	17.8	22.0	25.0
10,000	7.1	10.3	14.4	18.5	21.0
20,000	5.1	8.3	11.5	15.0	17.8

Note: Values derived from the all-season PMP charts published in HMR 51.

CP COL 2.4(1)

Table 2.4.3-202
Squaw Creek Watershed 6-hr Incremental PMP Estimates

Duration (hr)	Incremental PMP (in)
6	0.59
12	0.72
18	0.91
24	1.24
30	1.96
36	5.10
42	21.10
48	2.82
54	1.52
60	1.05
66	0.80
72	0.65
Total	38.46

Note: Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. The critical storm was determined to be 700 sq mi, with a 145 degree storm orientation, centered near the centroid of the Squaw Creek watershed.

CP COL 2.4(1)

Table 2.4.3-203 (Sheet 1 of 3)
Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates

Time (hr)	Hourly Incremental PMP (in) Basin 2
0100	0.10
0200	0.10
0300	0.10
0400	0.10
0500	0.10
0600	0.10
0700	0.11
0800	0.11
0900	0.11
1000	0.11
1100	0.11
1200	0.11
1300	0.12
1400	0.12
1500	0.12
1600	0.12
1700	0.12
1800	0.12
1900	0.13
2000	0.13
2100	0.15
2200	0.15
2300	0.15
2400	0.15
2500	0.15
2600	0.15
2700	0.21
2800	0.21
2900	0.21
3000	0.21
3100	0.21

CP COL 2.4(1)

Table 2.4.3-203 (Sheet 2 of 3)
Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates

Time (hr)	Hourly Incremental PMP (in) Basin 2
3200	0.21
3300	0.29
3400	0.30
3500	0.32
3600	0.33
3700	0.35
3800	0.37
3900	0.60
4000	0.66
4100	0.73
4200	0.81
4300	0.92
4400	1.04
4500	1.42
4600	2.12
4700	3.10
4800	6.42
4900	2.67
5000	1.89
5100	0.56
5200	0.51
5300	0.47
5400	0.44
5500	0.41
5600	0.39
5700	0.25
5800	0.25
5900	0.25
6000	0.25
6100	0.25
6200	0.25

CP COL 2.4(1)

**Table 2.4.3-203 (Sheet 3 of 3)
Squaw Creek Subbasin, Basin 2, Hourly Incremental PMP Estimates**

	Hourly Incremental PMP (in)
Time (hr)	Basin 2
6300	0.18
6400	0.18
6500	0.18
6600	0.18
6700	0.18
6800	0.18
6900	0.13
7000	0.13
7100	0.13
7200	0.13

CP COL 2.4(1)

Table 2.4.3-204
Paluxy River Watershed 6-hr Incremental PMP Estimates

Duration (hr)	Incremental PMP (in)
6	0.60
12	0.72
18	0.92
24	1.25
30	1.97
36	4.64
42	18.18
48	2.77
54	1.52
60	1.06
66	0.81
72	0.65
Total	35.08

Note: Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. Critical storm was determined to be 450 sq mi, with a 172 degree storm orientation, centered near the centroid of the upper Paluxy River watershed.

CP COL 2.4(1)

Table 2.4.3-205 (Sheet 1 of 3)
Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates

	Hourly Incremental PMP (in)	
Time (hr)	Basin 3	Basin 4
0100	0.10	0.10
0200	0.10	0.10
0300	0.10	0.10
0400	0.10	0.10
0500	0.10	0.10
0600	0.10	0.10
0700	0.11	0.11
0800	0.11	0.11
0900	0.11	0.11
1000	0.11	0.11
1100	0.11	0.11
1200	0.11	0.11
1300	0.12	0.12
1400	0.12	0.12
1500	0.12	0.12
1600	0.12	0.12
1700	0.12	0.12
1800	0.12	0.12
1900	0.14	0.14
2000	0.14	0.14
2100	0.16	0.15
2200	0.16	0.15
2300	0.16	0.15
2400	0.16	0.15
2500	0.16	0.15
2600	0.16	0.15
2700	0.21	0.21
2800	0.21	0.21
2900	0.21	0.21
3000	0.21	0.21

CP COL 2.4(1)

Table 2.4.3-205 (Sheet 2 of 3)
Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates

	Hourly Incremental PMP (in)	
Time (hr)	Basin 3	Basin 4
3100	0.21	0.21
3200	0.21	0.21
3300	0.30	0.29
3400	0.31	0.31
3500	0.32	0.32
3600	0.34	0.34
3700	0.36	0.35
3800	0.38	0.38
3900	0.60	0.60
4000	0.65	0.65
4100	0.71	0.72
4200	0.79	0.80
4300	0.89	0.91
4400	1.00	1.03
4500	1.34	1.43
4600	1.99	2.16
4700	3.01	3.25
4800	6.85	7.27
4900	2.54	2.76
5000	1.77	1.92
5100	0.56	0.56
5200	0.51	0.51
5300	0.47	0.47
5400	0.44	0.44
5500	0.42	0.41
5600	0.40	0.39
5700	0.26	0.26
5800	0.26	0.26
5900	0.26	0.26
6000	0.26	0.26

CP COL 2.4(1)

Table 2.4.3-205 (Sheet 3 of 3)
Paluxy River Watershed Subbasin Hourly Incremental PMP Estimates

	Hourly Incremental PMP (in)	
Time (hr)	Basin 3	Basin 4
6100	0.26	0.26
6200	0.26	0.26
6300	0.18	0.18
6400	0.18	0.18
6500	0.18	0.18
6600	0.18	0.18
6700	0.18	0.18
6800	0.18	0.18
6900	0.14	0.14
7000	0.14	0.14
7100	0.14	0.14
7200	0.14	0.14

Table 2.4.3-206
Not Used

CP COL 2.4(1)

Table 2.4.3-207
Watershed Subbasin Characteristics

Basin	Area (sq mi)	Baseflow (cfs)	L (mi)	Lca (mi)	C_t	C_p
Basin 1a & 1c	43.9	42.01	13.7	6.5	0.4	0.8
Basin 1b	20.3	42.01	5.3	2.5	0.4	0.8
Basin 2	10.65	6.97	4.6	3.0	0.4	0.8
Basin 3	24.3	15.90	4.9	5.6	0.4	0.8
Basin 4	410.0	268.28	59.3	25.8	0.4	0.8

L = length of the main stream from outlet to basin divide

L_{ca} = length along the main stream from the outlet to a point nearest the watershed centroid

C_t & C_p values resulting in higher water surface elevation at the CPNPP Units 3 and 4 were used.

Table 2.4.3-208
Squaw Creek Reservoir Watershed, Basin 1, 6-hr Incremental PMP Estimates

Duration (hr)	Incremental PMP (in)
6	0.61
12	0.74
18	0.94
24	1.28
30	2.02
36	5.01
42	24.93
48	2.87
54	1.57
60	1.08
66	0.82
72	0.67
Total	42.53

Note:

Values derived from HMR 51, HMR 52, and the use of HMR 52 computer software. The critical storm was determined to be 100 sq mi with a 181 degree storm orientation, centered near the centroid of the Squaw Creek watershed.

Table 2.4.3-209
Squaw Creek Reservoir Sub-basin, Basin 1, Hourly Incremental PMP Estimates

Time (hr)	Incremental PMP (in)	Time (hr)	Incremental PMP (in)
0100	0.10	3700	0.36
0200	0.10	3800	0.38
0300	0.10	3900	0.63
0400	0.10	4000	0.69
0500	0.10	4100	0.76
0600	0.10	4200	0.86
0700	0.11	4300	0.97
0800	0.11	4400	1.10
0900	0.11	4500	1.51
1000	0.11	4600	2.33
1100	0.11	4700	3.84
1200	0.11	4800	12.11
1300	0.12	4900	3.12
1400	0.12	5000	2.03
1500	0.12	5100	0.58
1600	0.12	5200	0.53
1700	0.12	5300	0.49
1800	0.12	5400	0.45
1900	0.14	5500	0.42
2000	0.14	5600	0.40
2100	0.16	5700	0.26
2200	0.16	5800	0.26
2300	0.16	5900	0.26
2400	0.16	6000	0.26
2500	0.16	6100	0.26
2600	0.16	6200	0.26
2700	0.21	6300	0.18
2800	0.21	6400	0.18
2900	0.21	6500	0.18
3000	0.21	6600	0.18
3100	0.21	6700	0.18
3200	0.21	6800	0.18
3300	0.30	6900	0.14
3400	0.31	7000	0.14
3500	0.33	7100	0.14
3600	0.34	7200	0.14

Table 2.4.3-210
Snyder’s Unit Hydrograph Characteristics

	Tp (hr)	Tb (hr)	Qp (cfs)	W75 (hr)	W50 (hr)	Q75 (cfs)	Q50 (cfs)	Nonlinear Qp +20%
Basin 1a & 1c	1.54	4.61	14,615	0.83	1.45	10,961	7308	17,538
Basin 1b	0.87	2.61	11,969	0.45	0.78	8977	5985	14,363
Basin 2	0.88	2.64	6203	0.45	0.79	4653	3102	7444
Basin 3	1.08	3.24	11,516	0.57	0.99	8367	5758	13,820
Basin 4	3.61	9.23	58,156	2.09	3.65	43,617	29,078	69,788

Tp = basin lag (hr); $C_t\ (LL_{ca})^{0.3}$
where
C_t = lag time coefficient
L = length of the main stream from the outlet to divide (mi)
L_{ca} = length along the main stream to a point nearest the watershed centroid (mi)

Tb = time base of the unit hydrograph (hr); 3+Tp/8 or 3 to 5 times Tp for small watersheds

Qp = peak discharge of the unit hydrograph (cfs); $640C_p$ A/Tp
where
C_p = peaking coefficient
A = drainage area (sq mi)

W75 = unit hydrograph width at 75 percent; $440(Qp/A)^{-1.08}$

W50 = unit hydrograph width at 50 percent; $770(Qp/A)^{-1.08}$

Q75 = unit hydrograph discharge at W75

Q50 = unit hydrograph discharge at W50

Table 2.4.4-201 (Sheet 1 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Volume Capacity[4] Maximum (ac-ft)
58	Running Water Draw Site 1 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Running Water Draw	692	128	1975	RE	3208	65	1581	2170	25,120
57	Running Water Draw WS SCS Site 3 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Running Water Draw	649	124	1979	RE	3250	55	233	4427	18,499
56	Lower Running Water Draw WS SCS Site 2 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	N Fork Running Water Draw	618	30	1977	RE	3430	41	42	5429	7383
55	Lower Running Water Draw WS SCS Site 3 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Running Water Draw	606	390	1982	RE	2500	37	54	8213	14,312
54	McMillan Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Double Mountain Fork Brazos R	577	236	1960	RE	1600	76	200	4200	8280

Table 2.4.4-201 (Sheet 2 of 8) Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Maximum (ac-ft)
53	White River Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	White River	518	172	1963	RE	4400	80	1477	31,537	80,000
52	Big Tank Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	TR-Double Mtn Fk Brazos River	539	ns	1965	RE	600	65	ns	185	490
51	Parks Lake Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Tr-Green Creek	539	ns	1971	RE	1142	50	6	110	220
50	John T Montford Dam (excluded - smaller volume and drainage area, and farther distance compared to Hubbard Creek Dam)	Double Mountain Fork Brazos R	513	394	1994	RE	440	141	2884	115,937	354,500
49	Duck Creek WS SCS Site 7 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Dockum Creek	502	12	1968	RE	2900	61	33	200	4712
48	Duck Creek WS SCS Site 5 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Cottonwood Creek	500	22	1969	RE	2550	71	148	2249	7900

Table 2.4.4-201 (Sheet 3 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Volume Capacity[4] Maximum (ac-ft)
47	Duck Creek WS SCS Site 1 Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Duck Creek	502	20	1968	RE	3600	62	79	634	10,750
46	Hagins Panther Canyon Lake Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Damn)	Tr-Salt Fork Brazos River	483	ns	1969	RE	300	50	10	140	320
45	So Relle Lake Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Stinking Creek	453	ns	1964	RE	1000	50	40	412	1000
44	**Lake Stamford Dam (included based on future conditions)**	**Paint Creek**	**332**	**360**	**1953**	**RE**	**3600[5]**	**78[5]**	**4690**	**57,927**	**150,000**
43	Lake Trammel Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Sweetwater Creek	439	49	1915	RE	1160	59	160	2500	5890
42	Lake Sweetwater Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Bitter Creek	429	104	1930	RE	3030	58	221	2544	19,340

Table 2.4.4-201 (Sheet 4 of 8) Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Volume Capacity[4] Maximum (ac-ft)
41	Lake Abilene Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Elm Creek	409	101	1921	RE	5040	64	583	45,000	45,000
40	Lake Kirby Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Cedar Creek	399	42	1928	RE	4200	50	780	7620	17,811
39	**Fort Phantom Hill Dam (included based on future conditions)**	**Big Elm Creek**	**375**	**478[6]**	**1938**	**RE**	**3740[6]**	**84**	**4246**	**70,036**	**127,000**
38	Lake Davis Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard	Dutchman Creek	347	ns	1959	RE	6864	32	ns	5395	19,000
37	Millers Creek Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Millers Creek	305	ns	1974	RE	8000	75	2882	29,171	131,000
36	Mexia Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Mexia Creek	307	ns	1950	RE	1660	52	ns	2070	3370
35	Williamson Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Sandy Creek	292	26	1923	CB	1064	96	1817	45,000	45,000

Table 2.4.4-201 (Sheet 5 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Volume Capacity[4] Maximum (ac-ft)
34	McCarty Lake Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Salt Prong Hubbard Creek	290	44	1942	RE	1250	50	263	2600	6696
33	Gonzales Creek Dam (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	Gonzales Creek	271	115	1948	RE	2700	50	954	11,400	38,242
32	Hubbard Creek Dam (excluded - future conditions more critical)	Hubbard Creek	261	1107	1962	RE	15,150[7]	112[7]	15,250	317,750	720,000
31	Eddleman Dam (excluded - smaller volume, height, and drainage area compared to Hubbard Creek Dam)	Flint Creek	218	42	1929	RE	4495	57	650	13,386	35,000
30	Graham Dam (excluded - smaller volume, height, and drainage area compared to Hubbard Creek Dam)	Salt Creek	219	42	1958	RE	4300	82	1900	39,000	105,000
29	**Morris Sheppard (included)**	**Brazos R**	**162**	**13,310**	**1941**	**CD**	**2747[8]**	**189[9]**	**17,624**	**556,220**	**556,220**
28	Lake Tucker Dam (excluded - small volume)	Russell Creek	126	24	1937	RE	900	97	81	1600	2500
27	Waddell Ranch Dam No 3 (excluded - small volume)	Joes Creek	110	ns	1975	RE	613	54	16	307	488
26	Lake Palo Pinto Dam (excluded - smaller volume and height compared to Morris Sheppard Dam)	Palo Pinto	104	471	1964	RE	1255	93	2661	44,100	170,735

Table 2.4.4-201 (Sheet 6 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Maximum (ac-ft)
25	Lake Mineral Wells Dam (excluded - small volume)	Rock Creek	91	63	1920	RE	1760	70	668	7065	16,356
24	Star Hollow Lake Dam (excluded - small volume)	Star Hollow Creek	84	ns	1967	RE	1120	54	92	1454	1959
23	Ruckers Creek WS SCS Site 1 Dam (excluded - small volume)	Rucker Creek	49	6	1968	RE	2080	50	33	133	2375
22	**De Cordova Bend Dam (included)**	**Brazos River**	**33**	**16,113[10]**	**1969**	**CBRE[10]**	**2200**	**84[10]**	**1350**	**136,823**	**240,640**
21	Safe Shutdown Impoundment Dam (excluded - adjacent to site)	Panther Branch	6	7	1977	ER	1520	70	7	367	900
20	Squaw Creek Dam (excluded - adjacent to site)	Squaw Creek	5	64	1977	RE	4690	152	3228	151,047	199,427
19	Paluxy River Channel Dam[11] (excluded - small volume)	Paluxy River	3	428	2007	PG	ns	8	9	35	35
18	Wheeler Branch Dam[11] (excluded - small volume)	Wheeler Branch	5	1.6	2007	RE	1750	80	180	4118	4118
17	Paluxy River WS SCS Site 5 Dam (excluded - small volume)	Germany Creek	39	160	1988	RE	1640	58	25	171	1604
16	Paluxy River WS SCS Site 1 Dam (excluded - small volume)	Tr-North Paluxy River	40	4	1984	RE	850	54	24	160	1512
15	Paluxy River WS SCS Site 6 Dam (excluded - small volume)	Straight Creek	38	5	1980	RE	1168	53	41	150	1211
14	Paluxy River WS SCS Site 3 Dam (excluded - small volume)	Tr-Paluxy River	39	2	1987	RE	865	51	16	110	821

Table 2.4.4-201 (Sheet 7 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Maximum (ac-ft)
13	Paluxy River WS SCS Site 9 Dam (excluded - small volume)	Tr- South Paluxy River	36	3	1984	RE	920	45	20	164	1107
12	Paluxy River WS SCS Site 12 Dam (excluded - small volume)	Tr- South Paluxy River	33	5	1985	RE	1240	45	25	123	1841
11	Paluxy River WS SCS Site 15 Dam (excluded - small volume)	Tr-Berry S Creek	25	12	1983	RE	1740	55	42	236	4064
10	Paluxy River WS SCS Site 16 Dam (excluded - small volume)	Goss Hollow	20	5	1980	RE	1848	53	32	200	2392
9	Paluxy River WS SCS Site 19 Dam (excluded - small volume)	Sycamore Creek	25	11	1981	RE	1910	64	38	200	4216
8	Paluxy River WS SCS Site 20 Dam (excluded - small volume)	Pony Creek	21	18	1981	RE	1950	74	65	200	6756
7	Paluxy River WS SCS Site 21 Dam (excluded - small volume)	Lallah Branch	21	16	1982	RE	2000	73	56	725	6140
6	Paluxy River WS SCS Site 23 Dam (excluded - small volume)	Rough Creek	18	5	1984	RE	1260	55	22	196	1762
5	Paluxy River WS SCS Site 25 Dam (excluded - small volume)	White Bluff Creek	11	11	1983	RE	2114	60	49	200	4485
4	Lake Virginia Dam3 (excluded - downstream dam and small volume)	11	11	1	1987	RE	845	56	47	898	1169
3	Cleburne State Park Lake Dam (excluded - downstream dam and small volume)	West Fork Camp Creek	17	ns	1940	RE	1300	62	ns	1450	2900

Table 2.4.4-201 (Sheet 8 of 8)
Information for Dams Upstream of Lake Whitney Dam

No.	Dam Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Date Completed	Type[2]	Length[3] (ft)	Height[3] (ft)	Surface Area (ac)	Volume Capacity[4] Normal (ac-ft)	Volume Capacity[4] Maximum (ac-ft)
2	Lake Pat Cleburne Dam (excluded - downstream dam small volume)	Nolan River	52	100	1964	RE	5190	78	1550	25,600	66,700
1	Lake Whitney (excluded - downstream dam)	Brazos River	56	17,656	1951	REPG	17,695	159	23,560	627,100	2,100,400

NOTES:
Highlighted entries identify dams evaluated for the quantitative dam failure analysis. Bold type entries identify damsn included in the critical dam failure scenario.
Information obtained from National Atlas, unless otherwise noted.
ns = not specified

1. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.
2. Type of dam:
   RE = Earth
   ER = Rockfill
   PG = Gravity
   CB = Buttress
3. Information obtained from the U.S. Army Corps of Engineers National Inventory of Dams database, unless otherwise noted.
4. Normal storage is the total storage below the normal retention level, including dead and inactive storage and excluding any flood control or surcharge storage. Maximum storage is the total storage below the maximum attainable water surface elevation, including any surcharge storage.
5. Information obtained from the Texas Water Development Board, Volumetric Survey of Lake Stamford, January 24, 2000.
6. Information obtained from the Texas Water Development Board, Volumetric Survey of Fort Phantom Hill Reservoir, March 10, 2003.
7. Information obtained from the Texas Water Development Board, Volumetric Survey of Hubbard Creek Reservoir, March 10, 2003.
8. Information obtained from Freeze and Nichols, Inc., Brazos River Authority Morris Sheppard Dam Breach Analysis Report, September 2001.
9. Information obtained from the Texas Water Development Board, Volumetric Survey Report of Possum Kingdom Lake December 2004-January 2005 Survey, May 2006.
10. Information obtained from the Texas Water Development Board, Volumetric Survey Report of Lake Granbury July 2003 Survey, September 2005.
11. Information obtained from Somervell County Water District and the 2011 Brazos G Regional Water Plan.

Table 2.4.4-202 (Sheet 1 of 2)
Information from the 2011 Brazos G Regional Water Plan for Strategies Upstream of Lake Whitney Dam

No.	Strategy Name (inclusion/exclusion from dam failure analysis)	Status[1]	River	Distance (river mi)[2]	Drainage Area (sq mi)	Type[3]	Length (ft)	Height (ft)	Surface Area (ac)	Volume Capacity (ac.-ft)
I	Double Mountain Fork West Reservoir (excluded - not proposed)	I	Double Mountain Fork of the Brazos River	433	1669	ns	ns	ns	6632	215,254
H	Double Mountain Fork East Reservoir (excluded - not proposed)	I	Double Mountain Fork of the Brazos River	403	1937	ns	ns	ns	10,814	280,814
G	Millers Creek Reservoir Augmentation (excluded - smaller volume, height, and drainage area, and farther distance compared to Hubbard Creek Dam)	R	Millers Creek	301	292	RE	ns	ns	2541	46,645
F	Millers Creek Reservoir Augmentation (excluded - smaller volume and height, and farther distance compared to Hubbard Creek Dam)	R	Lake Creek	337	ns	RE	5000	8	360	ns
E	Throckmorton Reservoir (excluded - not proposed)	I	North Elm Creek	278	82	ns	ns	ns	1161	15,900
D	**Cedar Ridge Reservoir (included)**	**R**	**Clear Fork of the Brazos River**	**334**	**2748**	**ns**	**ns**	**ns**	**6635**	**227,127**
C	South Bend Reservoir (excluded - not proposed)	I	Brazos River	228	13,168	RE	14,784	ns	29,877	771,604
B	Lake Palo Pinto Off-Channel Reservoir (excluded - not proposed)	I	Wilson Hollow	109	ns	RE	1550	ns	182	10,000 up to 22,000

Table 2.4.4-202 (Sheet 2 of 2)
Information from the 2011 Brazos G Regional Water Plan for Strategies Upstream of Lake Whitney Dam

No.	Strategy Name (inclusion/exclusion from dam failure analysis)	Status[1]	River	Distance (river mi)[2]	Drainage Area (sq mi)	Type[3]	Length (ft)	Height (ft)	Surface Area (ac)	Volume Capacity (ac.-ft)
A	Turkey Peak Reservoir (excluded - smaller volume and height compared to Morris Sheppard Dam)	R	Palo Pinto Creek	101	ns	ns	ns	ns	648	22,577

NOTES:
Highlighted entries identify dams evaluated for the quantitative dam failure analysis. Bold type entries identify damsn included in the critical dam failure scenario.
Information obtained from National Atlas, unless otherwise noted.
ns = not specified

1. Status of water management strategy in the 2011 Brazos G Regional Water Plan
   I = identified as potentially feasible water management strategy
   R = recommended water management strategy
2. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.
3. Type of dam: RE = Earth

Table 2.4.4-203
Information from the Llano Estacado Regional Water Plan for Strategies Upstream of Lake Whitney Dam

No.	Strategy Name (inclusion/exclusion from dam failure analysis)	River	Distance (river mi)[1]	Drainage Area (sq mi)	Type[2]	Length (ft)	Height (ft)	Surface Area (ac)	Volume Capacity (ac.-ft)
L	Lake 7 (excluded - smaller volume compared John T. Montford Dam)	North Fork Double Mountain Fork Brazos River	580	ns	ns	ns	ns	ns	20,700
K	Post Reservoir (excluded - smaller volume compared John T. Montford Dam)	North Fork Double Mountain Fork Brazos River	536	ns	RE	5800	ns	2280	56,000
J	Diversion Reservoir (excluded - smaller volume compared John T. Montford Dam)	North Fork Double Mountain Fork Brazos River	515	ns	ns	ns	ns	ns	1000

NOTES:
ns = not specified

1. Distance in river miles from the dam to the confluence of the Brazos River and Paluxy River.I
2. Type of dam: RE = Earth

Table 2.4.5-201
Summary of SCR Shoreline Slopes Geometry

Cross Section	Slope Height (ft)	Slope Gradient Range (H:V)
1	137	28:1 to 4.8:1
2	139	33:1 to 15:1
3	77	13:1 to 8.5:1
4	137	14:1 to 4.5:1
5	138	19:1 to 7.6:1
6	142	15:1 to 5.2:1
7	135	24:1 to 8.4:1
8	145	30:1 to 3.7:1
9	120	13:1 to 5:1
10	135	10:1 to 3.2:1

Note:

The cross sections are depicted on Figure 2.4.5-201.

Table 2.4.5-202 (Sheet 1 of 2)
Summary of SCR Shoreline Stability Analyses

Cases	Cross Section[2]	Slope Stability Factor of Safety
Permanent, Static	1	13.1
Permanent, Static	2	34.2
Permanent, Static	3	27.6
Permanent, Static	4	9.3
Permanent, Static	5	11.6
Permanent, Static	6	5.8
Permanent, Static	7	7.3
Permanent, Static	8	2.9
Permanent, Static	9	8.1
Permanent, Static	10	5.8
Permanent, Pseudo-Static	1	3.71
Permanent, Pseudo-Static	2	9.12
Permanent, Pseudo-Static	3	8.85
Permanent, Pseudo-Static	4	3.92
Permanent, Pseudo-Static	5	3.88
Permanent, Pseudo-Static	6	3.88
Permanent, Pseudo-Static	7	3.67
Permanent, Pseudo-Static	8	2.05
Permanent, Pseudo-Static	9	3.25
Permanent, Pseudo-Static	10	2.66
Rapid Drawdown[1]	1	13.1
Rapid Drawdown[1]	2	31.9
Rapid Drawdown[1]	3	25.9
Rapid Drawdown[1]	4	9.3

Table 2.4.5-202 (Sheet 2 of 2)
Summary of SCR Shoreline Stability Analyses

Cases	Cross Section[2]	Slope Stability Factor of Safety
Rapid Drawdown[1]	5	11.6
Rapid Drawdown[1]	6	5.8
Rapid Drawdown[1]	7	7.3
Rapid Drawdown[1]	8	2.9
Rapid Drawdown[1]	9	8.1
Rapid Drawdown[1]	10	5.8

Notes:

1. The rapid drawdown represents the case where the SCR water level is instantaneously lowered from the maximum (El. 783 ft) to the minimum (El. 770 ft).

2. The cross sections are depicted on Figure 2.4.5-201.

Table 2.4.5-203 (Sheet 1 of 2)
Summary of SCR Shoreline Stability Analyses
(Extreme Worst Case Scenario of "All Soil" Model)

Cases	Cross Section[2]	Slope Stability Factor of Safety
Permanent, Static	1	3.2
Permanent, Static	2	6.0
Permanent, Static	3	5.7
Permanent, Static	4	2.8
Permanent, Static	5	4.7
Permanent, Static	6	2.4
Permanent, Static	7	3.4
Permanent, Static	8	1.8
Permanent, Static	9	2.3
Permanent, Static	10	1.7
Permanent, Pseudo-Static	1	1.29
Permanent, Pseudo-Static	2	1.52
Permanent, Pseudo-Static	3	1.48
Permanent, Pseudo-Static	4	1.22
Permanent, Pseudo-Static	5	1.43
Permanent, Pseudo-Static	6	1.26
Permanent, Pseudo-Static	7	1.43
Permanent, Pseudo-Static	8	1.15
Permanent, Pseudo-Static	9	1.27
Permanent, Pseudo-Static	10	1.12
Rapid Drawdown[1]	1	3.0
Rapid Drawdown[1]	2	4.9
Rapid Drawdown[1]	3	4.4

**Table 2.4.5-203 (Sheet 2 of 2)
Summary of SCR Shoreline Stability Analyses
(Extreme Worst Case Scenario of "All Soil" Model)**

Cases	Cross Section[2]	Slope Stability Factor of Safety
Rapid Drawdown[1]	4	2.6
Rapid Drawdown[1]	5	4.3
Rapid Drawdown[1]	6	2.1
Rapid Drawdown[1]	7	3.2
Rapid Drawdown[1]	8	1.6
Rapid Drawdown[1]	9	2.1
Rapid Drawdown[1]	10	1.5

Notes:

1. The rapid drawdown represents the case where the SCR water level is instantaneously lowered from the maximum (El. 783 ft) to the minimum (El. 770 ft).

2. The cross sections are depicted on Figure 2.4.5-201.

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Table 2.4.7-201 (Sheet 1 of 4)
Water Temperature Data

Station	Water Body	Degree Celsius	Degree Fahrenheit	Sample Date
11555	SQUAW CREEK AT SH 144	17.20	62.96	12/4/1973
11555	SQUAW CREEK AT SH 144	11.00	51.80	12/16/1974
11555	SQUAW CREEK AT SH 144	12.00	53.60	1/15/1975
11555	SQUAW CREEK AT SH 144	10.00	50.00	2/11/1976
11555	SQUAW CREEK AT SH 144	31.00	87.80	8/16/1977
11555	SQUAW CREEK AT SH 144	15.00	59.00	11/9/1978
11555	SQUAW CREEK AT SH 144	8.50	47.30	2/8/1979
11555	SQUAW CREEK AT SH 144	18.60	65.48	11/17/1981
11555	SQUAW CREEK AT SH 144	5.50	41.90	2/10/1982
11555	SQUAW CREEK AT SH 144	6.60	43.88	2/7/1983
11555	SQUAW CREEK AT SH 144	16.30	61.34	2/15/1984
11555	SQUAW CREEK AT SH 144	26.60	79.88	6/24/1985
11555	SQUAW CREEK AT SH 144	24.10	75.38	5/15/1986
11555	SQUAW CREEK AT SH 144	23.60	74.48	5/14/1987
11555	SQUAW CREEK AT SH 144	24.90	76.82	7/6/1988
11566	SCR	20.00	68.00	4/17/1985
11856	BRAZOS RIVER AT US 67	9.00	48.20	12/7/1968
11856	BRAZOS RIVER AT US 67	7.00	44.60	1/6/1969
11856	BRAZOS RIVER AT US 67	3.90	39.02	1/5/1970
11856	BRAZOS RIVER AT US 67	10.00	50.00	12/1/1971
11856	BRAZOS RIVER AT US 67	9.40	48.92	1/13/1973
11856	BRAZOS RIVER AT US 67	4.40	39.92	2/4/1972
11856	BRAZOS RIVER AT US 67	6.70	44.06	12/8/1972
11856	BRAZOS RIVER AT US 67	6.70	44.06	1/13/1973
11856	BRAZOS RIVER AT US 67	15.00	59.00	12/4/1973
11856	BRAZOS RIVER AT US 67	11.70	53.06	1/7/1974
11856	BRAZOS RIVER AT US 67	10.00	50.00	2/6/1974

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Table 2.4.7-201 (Sheet 2 of 4)
Water Temperature Data

Station	Water Body	Degree Celsius	Degree Fahrenheit	Sample Date
11856	BRAZOS RIVER AT US 67	10.00	50.00	12/16/1974
11856	BRAZOS RIVER AT US 67	10.00	50.00	1/15/1975
11856	BRAZOS RIVER AT US 67	7.00	44.60	2/10/1975
11856	BRAZOS RIVER AT US 67	9.50	49.10	2/11/1976
11856	BRAZOS RIVER AT US 67	31.50	88.70	8/16/1977
11856	BRAZOS RIVER AT US 67	16.00	60.80	11/8/1978
11856	BRAZOS RIVER AT US 67	29.50	85.10	8/16/1979
11856	BRAZOS RIVER AT US 67	12.00	53.60	1/24/1980
11856	BRAZOS RIVER AT US 67	14.50	58.10	2/25/1981
11856	BRAZOS RIVER AT US 67	19.00	66.10	12/9/1981
11856	BRAZOS RIVER AT US 67	5.00	41.00	1/18/1982
11856	BRAZOS RIVER AT US 67	5.00	41.00	2/10/1982
11856	BRAZOS RIVER AT US 67	5.20	41.36	2/7/1983
11856	BRAZOS RIVER AT US 67	7.50	45.50	11/29/1983
11856	BRAZOS RIVER AT US 67	7.00	44.60	1/10/1984
11856	BRAZOS RIVER AT US 67	9.00	48.20	2/8/1984
11856	BRAZOS RIVER AT US 67	9.00	48.20	1/28/1985
11856	BRAZOS RIVER AT US 67	8.00	46.40	1/30/1986
11856	BRAZOS RIVER AT US 67	14.00	57.20	1/29/1987
11856	BRAZOS RIVER AT US 67	30.70	87.26	7/6/1988
11856	BRAZOS RIVER AT US 67	30.60	87.08	8/1/1989
11856	BRAZOS RIVER AT US 67	27.80	82.04	6/12/1990
11856	BRAZOS RIVER AT US 67	33.40	92.12	7/15/1991
11856	BRAZOS RIVER AT US 67	28.00	82.40	7/8/1992
11856	BRAZOS RIVER AT US 67	13.70	56.66	11/16/1993
11856	BRAZOS RIVER AT US 67	14.60	58.28	4/6/1994
11856	BRAZOS RIVER AT US 67	12.70	54.86	12/10/1996

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Table 2.4.7-201 (Sheet 3 of 4)
Water Temperature Data

Station	Water Body	Degree Celsius	Degree Fahrenheit	Sample Date
11856	BRAZOS RIVER AT US 67	16.50	61.70	4/1/1997
11856	BRAZOS RIVER AT US 67	12.90	55.22	10/27/1997
11856	BRAZOS RIVER AT US 67	9.20	48.56	1/20/1998
11976	PALUXY RIVER IN CITY PARK	15.00	59.00	12/4/1973
11976	PALUXY RIVER IN CITY PARK	7.80	46.04	1/7/1974
11976	PALUXY RIVER IN CITY PARK	10.00	50.00	2/6/1974
11976	PALUXY RIVER IN CITY PARK	10.00	50.00	12/16/1974
11976	PALUXY RIVER IN CITY PARK	9.50	49.10	1/15/1975
11976	PALUXY RIVER IN CITY PARK	12.00	53.60	2/10/1975
11976	PALUXY RIVER IN CITY PARK	16.00	60.80	11/4/1975
11976	PALUXY RIVER IN CITY PARK	10.00	50.00	1/11/1976
11976	PALUXY RIVER IN CITY PARK	7.50	45.50	2/8/1979
11976	PALUXY RIVER IN CITY PARK	11.00	51.80	1/24/1980
11976	PALUXY RIVER IN CITY PARK	20.30	68.54	11/12/1980
11976	PALUXY RIVER IN CITY PARK	14.10	57.38	2/25/1981
11976	PALUXY RIVER IN CITY PARK	13.60	56.48	3/24/1981
11976	PALUXY RIVER IN CITY PARK	4.00	39.20	2/10/1982
11976	PALUXY RIVER IN CITY PARK	15.20	59.36	11/4/1982
11976	PALUXY RIVER IN CITY PARK	5.30	41.54	2/7/1983
11976	PALUXY RIVER IN CITY PARK	16.90	62.42	11/21/1983
11976	PALUXY RIVER IN CITY PARK	18.90	66.02	2/15/1984
11976	PALUXY RIVER IN CITY PARK	26.10	78.98	5/16/1984
11976	PALUXY RIVER IN CITY PARK	30.10	86.18	6/24/1985
11976	PALUXY RIVER IN CITY PARK	25.00	77.00	5/15/1986
11976	PALUXY RIVER IN CITY PARK	28.80	83.84	5/14/1987
11976	PALUXY RIVER IN CITY PARK	27.20	80.96	7/6/1988
11976	PALUXY RIVER IN CITY PARK	28.60	83.48	7/8/1992

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Table 2.4.7-201 (Sheet 4 of 4)
Water Temperature Data

Station	Water Body	Degree Celsius	Degree Fahrenheit	Sample Date
11976	PALUXY RIVER IN CITY PARK	27.60	81.68	7/28/1993
11976	PALUXY RIVER IN CITY PARK	31.20	88.16	9/2/1993
11976	PALUXY RIVER IN CITY PARK	11.50	52.70	3/29/1994
11976	PALUXY RIVER IN CITY PARK	9.80	49.64	12/13/1994
11976	PALUXY RIVER IN CITY PARK	21.10	69.98	3/23/1995
11976	PALUXY RIVER IN CITY PARK	12.10	53.78	11/8/1995
11976	PALUXY RIVER IN CITY PARK	5.10	41.18	1/24/1996
11976	PALUXY RIVER IN CITY PARK	16.30	61.34	11/13/1996
11976	PALUXY RIVER IN CITY PARK	11.90	53.42	2/18/1997
11976	PALUXY RIVER IN CITY PARK	5.60	42.08	11/17/1997
11976	PALUXY RIVER IN CITY PARK	12.80	55.04	2/10/1998

(Reference 2.4-248)

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Table 2.4.11-201 (Sheet 1 of 4)
Minimum Daily Streamflow Observed on the Brazos River near Glen Rose, Texas, (USGS Station 08091000) 1923–2007

Climatic Year[a]	Date	Minimum Flow, cfs
1923[b]	2/8-9/1924	132
1924	9/7-9/1924	0.00
1925	4/1-15/1925	2.8
1926	3/28/1927	148
1927	12/6/1927	18
1928	10/27/1928	0.8
1929	9/2-7/1929	0.8
1930	4/25-26/1930	1.5
1931	9/13/1931-10/11/1931	0.00
1932	7/15-17/1932	90
1933	7/15/1933	25
1934	6/28/1934-9/13/1934	0.00
1935	3/31/1936	23
1936	8/24/1936-9/14/1936	0.00
1937	8/21/1937	8
1938	11/1-2/1938	6
1939	10/7-9/1939 & 10/16-27/1939	0.00
1940	4/2-5/1940	3
1941	3/23-24/1941	141
1942	4/4-5/1942	131
1943	11/12-13/1943	33
1944	9/24/1944	51
1945	1/2/1946	118
1946	6/16/1946	171

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Table 2.4.11-201 (Sheet 2 of 4) Minimum Daily Streamflow Observed on the Brazos River near Glen Rose, Texas, (USGS Station 08091000) 1923–2007

Climatic Year[(a)]	Date	Minimum Flow, cfs
1947	11/10/1947	118
1948	10/5/1948	93
1949	4/18/1949	89
1950	3/23/1951	95
1951	3/15/1952	40
1952	10/30-31/1952	0.1
1953	6/17/1953	3
1954	9/15/1954	18
1955	5/9/1955	8.5
1956	9/25/1956	1.8
1957	4/13/1957	156
1958	3/15/1959	61
1959	8/28/1959	47
1960	6/25/1960	88
1961	6/5/1961	37
1962	5/26/1962	18
1963	10/21-22/1963	23
1964	5/31/1964	30
1965	11/30/1965	57
1966	3/19/1967	35
1967	5/14-15/1967	40
1968	3/12/1969	4.9
1969	11/26/1969	20
1970	8/1/1970	3.4

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Table 2.4.11-201 (Sheet 3 of 4)
Minimum Daily Streamflow Observed on the Brazos River near Glen Rose, Texas, (USGS Station 08091000) 1923–2007

Climatic Year[(a)]	Date	Minimum Flow, cfs
1971	8/9/1971	4.2
1972	7/18/1972	16
1973	9/1/1973	2.9
1974	4/10/1974	3.7
1975	3/27/1976	27
1976	8/29/1976	25
1977	7/13/1977	9.4
1978	7/8/1978	7
1979	9/27/1979	24
1980	8/21/1980	7.3
1981	8/14/1981	10
1982	10/8/1982	11
1983	9/30/1983	4.6
1984	7/14/1984	0.17
1985	8/5-6/1985	25
1986	8/31/1986	7.9
1987	8/23/1987	8.5
1988	11/12/1988	3.2
1989	7/23/1989	18
1990	12/23/1990	8.1
1991	5/21/1991	16
1992	10/16/1992	16
1993	8/16-18/1993 & 8/21-22/1993	17
1994	7/30/1994	8

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**Table 2.4.11-201 (Sheet 4 of 4)
Minimum Daily Streamflow Observed on the Brazos River near
Glen Rose, Texas, (USGS Station 08091000) 1923–2007**

Climatic Year[a]	Date	Minimum Flow, cfs
1995	3/25-26/1995	31
1996	8/6/1996	6.1
1997	10/1-2/1997	22
1998	7/31/1998 & 8/3/1998	7.2
1999	11/20/1999	13
2000	5/17/2000	1.6
2001	11/10/2001	1.5
2002	8/21/2002	7.8
2003	7/20/2003	4.9
2004	9/6-7/2004	18
2005	5/21/2005	14
2006	6/30/2006	7.2
2007[b]	10/2/2007 & 10/27-28/2007 & 11/6/2007	18

a) Climatic Year - April 1 to March 31.

b) Year incomplete, available data 10/1/1923 – 3/31/1924
Year incomplete, available data 4/1/2007 – 11/6/2007
(Reference 2.4-220)

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Table 2.4.11-202 (Sheet 1 of 2)
Minimum Daily Streamflow Observed on the Brazos River near Dennis, Texas, (USGS Station 08090800) 1968–2007

Climatic Year[a]	Date	Minimum Flow, cfs
1968[b]	10/31/1968	36
1969	11/22/1969	82
1970	3/16/1971 & 3/19/1971	22
1971	5/28/1971 & 7/20/1971	3.5
1972	7/22/1972	18
1973	8/29/1973	12
1974	7/15/1974	9.7
1975	3/30-31/1976	25
1976	4/3/1976	22
1977	9/26/1977	13
1978	8/2/1978	1.2
1979	3/25/1980	30
1980	4/9-10/1980	26
1981	4/21/1981	42
1982	10/21/1982 & 10/30/1982	27
1983	9/27/1983	47
1984	9/12/1984	6.1
1985	10/9/1985	30
1986	4/19/1986	13
1987	3/30/1988	25
1988	8/12/1988	2.3
1989	4/23/1989	34
1990	12/23/1990	85
1991	4/21/1991	69
1992	12/11/1992	114
1993	2/13/1994	49
1994	4/19/1994	27
1995	2/20/1996	79

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**Table 2.4.11-202 (Sheet 2 of 2)
Minimum Daily Streamflow Observed on the Brazos River near Dennis, Texas, (USGS Station 08090800) 1968–2007**

Climatic Year[a]	Date	Minimum Flow, cfs
1996	8/8/1996	16
1997	11/9/1997	68
1998	3/3/1999	9.4
1999	2/11/2000	16
2000	10/7/2000	30
2001	10/25/2001 & 10/31/2001	12
2002	11/28/2002	68
2003	5/17/2003 & 2/11/2004 & 2/13/2004 & 2/21/2004	31
2004	4/13/2004	49
2005	8/1-2/2005	47
2006	10/8/2006	28
2007[b]	11/6/2007	65

a) Climatic Year - April 1 to March 31.

b) Year incomplete, available data 4/25/1968 – 3/31/1968
Year incomplete, available data 4/1/2007 – 11/6/2007
(Reference 2.4-220)

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**Table 2.4.11-203 (Sheet 1 of 2)
Minimum Daily Streamflow Observed on Squaw Creek near
Glen Rose, Texas, (USGS Station 08091750) 1977–2006**

Climatic Year[(a)]	Date	Minimum Flow, cfs
1977[(b)]	2/20/1978 & 3/9-10/1978 & 3/19-21/1978	1.4
1978	6/23/1978 & 6/25/1978 & 6/30/1978	0.89
1979	2/19-20/1980	2.2
1980	7/17/1980 & 7/21/1980	2.2
1981	10/15/1981	2.5
1982	10/28-31/1982 & 11/1/1982 & 3/28-31/1983	2.9
1983	9/1-6/1983	2.2
1984	8/15/1984	1.6
1985	5/4-5/1985	1.7
1986	7/16/1986	2.6
1987	4/27-30/1987 & 5/1-3/1987	1.7
1988	11/27/1988	1.9
1989	7/23/1989	5.4
1990	10/17/1990 & 3/17/1991 & 3/28-29/1991	2.9
1991	8/16-18/1991 & 8/29/1991	1.4
1992	10/26/1992	0.64
1993	1/5-6/1994	1.6
1994	8/19/1994	0.74
1995	2/19/1996	0.93
1996	8/5/1996	0.54
1997	12/25/1997	0.69
1998	6/10/1998	2
1999	7/25/1999	1.2
2000	10/12-13/2000	2.5
2001	2/24/2002	4.6

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Table 2.4.11-203 (Sheet 2 of 2)
Minimum Daily Streamflow Observed on Squaw Creek near Glen Rose, Texas, (USGS Station 08091750) 1977–2006

Climatic Year[a]	Date	Minimum Flow, cfs
2002	6/12/2002	2
2003	5/21/2003	1.9
2004	3/16/2005	0.38
2005	5/18/2005	0.59
2006[b]	7/29/2006	2.9

a) Climatic Year - April 1 to March 31.

b) Year incomplete, available data 10/1/1977 – 3/31/1977
Year incomplete, available data 4/1/2006 – 9/30/2006
(Reference 2.4-220)

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Table 2.4.11-204
Brazos River Low Flow Frequency for Selected Durations and Return Periods, cfs

	Return Period, years		
Duration, days	5	10	100
1	16.5	11.8	5.1
7	20.4	14.7	6.5
30	31.6	22.4	9.7

Note:
Low flow based on statistical analysis of data for USGS gage on the Brazos River near Dennis, Texas (USGS 08090800) from 1968 to 2007.

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Table 2.4.12-201 (Sheet 1 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3132601	Hood	F.C. Spencer	Domestic	21	Twin Mountains Formation	323312	980025	Withdrawal of Water
3132602	Hood	Signal & Loffland	Unused	5278	Aquifer Code Not Applicable	323314	980151	Oil or Gas Use
3132901	Hood	Herman D. Howard	Stock	46	Twin Mountains Formation	323027	980215	Withdrawal of Water
3132902	Hood	Northern Natural Gas	Industrial	184	Twin Mountains Formation	323022	980056	Withdrawal of Water
3132903	Hood	Shane Butler	Domestic	56	Trinity Group	323026	980214	Withdrawal of Water
3140201	Hood	Lipan Water Works	Public Supply	120	Twin Mountains Formation	322950	980313	Withdrawal of Water
3140301	Hood	City of Lipan	Public Supply	95	Twin Mountains Formation	322925	980227	Withdrawal of Water
3225402	Hood		Not Listed	0	Twin Mountains Formation	323230	975731	Spring
3225501	Hood	N.B. Brewer	Domestic	70	Twin Mountains Formation	323316	975506	Withdrawal of Water
3225701	Hood	T.L. Compton	Domestic	100	Twin Mountains Formation	323053	975830	Withdrawal of Water
3225801	Hood	Intrastate Gathering	Industrial	140	Twin Mountains Formation	323001	975625	Withdrawal of Water
3226501	Hood	L.H. Thomas	Unused	140	Twin Mountains Formation	323246	974937	Withdrawal of Water
3226502	Hood	L.H. Thomas	Domestic	140	Twin Mountains Formation	323251	974947	Withdrawal of Water
3226701	Hood	O.P. Leonard	Domestic	80	Twin Mountains Formation	323028	975017	Withdrawal of Water
3226702	Hood	Rolling Hills Water	Public Supply	100	Twin Mountains Formation	323204	975004	Withdrawal of Water
3226703	Hood	Resort Water Services	Public Supply	150	Twin Mountains Formation	323056	975047	Withdrawal of Water
3226704	Hood	Resort Water Services	Public Supply	92	Twin Mountains Formation	323033	975045	Withdrawal of Water
3226705	Hood	Rolling Hills Water	Unused	84	Twin Mountains Formation	323150	975054	Withdrawal of Water
3226706	Hood	Rolling Hills Water	Public Supply	84	Twin Mountains Formation	323204	975004	Withdrawal of Water
3226707	Hood	Resort Water Services	Public Supply	200	Twin Mountains Formation	323049	975111	Withdrawal of Water
3226801	Hood	C.A. Cassity	Irrigation	170	Twin Mountains Formation	323034	974934	Withdrawal of Water
3226802	Hood	Hood County Water Co.	Public Supply	240	Twin Mountains Formation	323016	974808	Withdrawal of Water
3226803	Hood	Long Creek Water Co.	Public Supply	200	Twin Mountains Formation	323123	974842	Withdrawal of Water

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Table 2.4.12-201 (Sheet 2 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3226804	Hood	Long Creek Water Co.	Public Supply	268	Twin Mountains Formation	323101	974824	Withdrawal of Water
3226805	Hood	Lakeside MHP	Public Supply	110	Twin Mountains Formation	323048	974941	Withdrawal of Water
3226901	Hood	James B. Robinson	Domestic	193	Twin Mountains Formation	323213	974647	Withdrawal of Water
3226902	Hood	R.F. Parkinson	Irrigation	420	Twin Mountains Formation	323220	974526	Withdrawal of Water
3227402	Hood	Kenneth Marczak	Stock	75	Paluxy Sand	323318	974400	Withdrawal of Water
3227403	Hood	Albert W. Hall	Domestic	358	Twin Mountains Formation	323318	974358	Withdrawal of Water
3227404	Hood	D.O. Tankersley	Industrial	140	Paluxy Sand	323244	974350	Withdrawal of Water
3227405	Hood	Kenneth Marczak	Domestic	440	Twin Mountains Formation	323320	974401	Withdrawal of Water
3227503	Hood	Spring Valley Water	Public Supply	240	Twin Mountains Formation	323256	974134	Withdrawal of Water
3227601	Hood	L.W.B. Construction	Unused	360	Paluxy Sand	323253	973901	Withdrawal of Water
3227701	Hood	Earl Porter	Domestic	70	Paluxy Sand	323130	974322	Withdrawal of Water
3227702	Hood	X.A. Myer	Domestic	34	Paluxy Sand	323011	974342	Withdrawal of Water
3227703	Hood	R.L. Tankersley	Irrigation	415	Twin Mountains Formation	323223	974436	Withdrawal of Water
3227704	Hood	R.L. Tankersley	Irrigation	387	Twin Mountains Formation	323222	974439	Withdrawal of Water
3227705	Hood	Doug Crough	Domestic	408	Twin Mountains Formation	323021	974353	Withdrawal of Water
3227706	Hood	Scott Parkinson	Irrigation	425	Twin Mountains Formation	323218	974500	Withdrawal of Water
3227707	Hood		Not Listed	0	Paluxy Sand	323200	974445	Spring
3228704	Hood	Hughie Long	Domestic	353	Paluxy Sand	323147	973709	Withdrawal of Water
3233201	Hood	A.B. Clapp	Domestic	55	Twin Mountains Formation	322937	975556	Withdrawal of Water
3233401	Hood	V.H. Musick	Domestic	342	Twin Mountains Formation	322559	975858	Withdrawal of Water
3233402	Hood	Dan Knouf	Domestic	380	Twin Mountains Formation	322521	975737	Withdrawal of Water
3233403	Hood	Dan Knouf	Domestic	25	Paluxy Sand	322516	975737	Withdrawal of Water
3233404	Hood	Dan Knouf	Irrigation	347	Twin Mountains Formation	322514	975734	Withdrawal of Water

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Table 2.4.12-201 (Sheet 3 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3233801	Hood	C.W. Bridler	Domestic	317	Twin Mountains Formation	322350	975715	Withdrawal of Water
3233802	Hood	Doris Baker	Unused	297	Twin Mountains Formation	322344	975526	Withdrawal of Water
3233803	Hood	Doris Baker	Unused	310	Twin Mountains Formation	322344	975521	Withdrawal of Water
3233804	Hood	Vera Brooks	Domestic	307	Twin Mountains Formation	322329	975508	Withdrawal of Water
3233805	Hood	City of Tolar	Public Supply	535	Twin Mountains Formation	322339	975516	Withdrawal of Water
3233806	Hood	City of Tolar	Public Supply	422	Twin Mountains Formation	322341	975518	Withdrawal of Water
3233807	Hood	City of Tolar	Public Supply	0	Twin Mountains Formation	322343	975508	Withdrawal of Water
3233808	Hood	City of Tolar	Public Supply	450	Twin Mountains Formation	322341	975506	Withdrawal of Water
3233901	Hood	George Chrane	Domestic	348	Twin Mountains Formation	322353	975327	Withdrawal of Water
3233902	Hood	Leonard Leito	Domestic	405	Twin Mountains Formation	322450	975236	Withdrawal of Water
3234101	Hood	Steve Bird	Domestic	120	Twin Mountains Formation	322820	975010	Withdrawal of Water
3234102	Hood	City of Granbury	Public Supply	115	Twin Mountains Formation	322730	975027	Withdrawal of Water
3234103	Hood	Boswell Water Co.	Public Supply	132	Twin Mountains Formation	322730	975024	Withdrawal of Water
3234104	Hood	Oak Trail Shores	Public Supply	190	Twin Mountains Formation	322915	975024	Withdrawal of Water
3234105	Hood	Oak Trail Shores	Public Supply	231	Twin Mountains Formation	322913	975100	Withdrawal of Water
3234106	Hood	Oak Trail Shores	Public Supply	0	Twin Mountains Formation	322916	975017	Withdrawal of Water
3234107	Hood	Oak Trail Shores	Public Supply	206	Twin Mountains Formation	322909	975019	Withdrawal of Water
3234108	Hood	Oak Trail Shores	Public Supply	188	Twin Mountains Formation	322912	975014	Withdrawal of Water
3234109	Hood	Oak Trail Shores	Public Supply	155	Twin Mountains Formation	322903	975008	Withdrawal of Water
3234112	Hood	Dr. Roger Nunnalee	Irrigation	122	Twin Mountains Formation	322738	975034	Withdrawal of Water
3234113	Hood	Oak Trail Shores	Public Supply	190	Twin Mountains Formation	322917	975003	Withdrawal of Water
3234114	Hood	Oak Trail Shores	Public Supply	0	Twin Mountains Formation	322910	975005	Withdrawal of Water
3234201	Hood	C.E. Reese	Stock	114	Twin Mountains Formation	322922	974857	Withdrawal of Water

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Table 2.4.12-201 (Sheet 4 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3234202	Hood	C.E. Reese	Domestic	171	Twin Mountains Formation	322851	974907	Withdrawal of Water
3234203	Hood	Oak Trail Shores	Public Supply	190	Twin Mountains Formation	322908	974947	Withdrawal of Water
3234206	Hood	Oak Trail Shores	Public Supply	80	Twin Mountains Formation	322830	974906	Withdrawal of Water
3234207	Hood	Don Kennon	Irrigation	220	Twin Mountains Formation	322936	974733	Withdrawal of Water
3234208	Hood	Laguna Vista	Public Supply	170	Twin Mountains Formation	322950	974811	Withdrawal of Water
3234209	Hood	Laguna Vista	Public Supply	220	Twin Mountains Formation	322957	974753	Withdrawal of Water
3234210	Hood	Live Oak Water Co.	Public Supply	216	Twin Mountains Formation	322811	974734	Withdrawal of Water
3234211	Hood	Summerlin Estates	Public Supply	258	Twin Mountains Formation	322807	974753	Withdrawal of Water
3234212	Hood	Laguna Vista	Public Supply	205	Twin Mountains Formation	322944	974750	Withdrawal of Water
3234213	Hood		Not Listed	0	Glen Rose Limestone	322830	974931	Spring
3234301	Hood	Laguna Tres	Public Supply	155	Twin Mountains Formation	322858	974716	Withdrawal of Water
3234302	Hood	Community Water Co.	Public Supply	188	Twin Mountains Formation	322738	974528	Withdrawal of Water
3234303	Hood	Sky Harbor Water	Public Supply	500	Twin Mountains Formation	322931	974610	Withdrawal of Water
3234304	Hood	Tri-County Electic	Industrial	140	Twin Mountains Formation	322749	974701	Withdrawal of Water
3234305	Hood	Mesa Grande Water	Public Supply	220	Twin Mountains Formation	322810	974650	Withdrawal of Water
3234306	Hood	First Baptist Church -	Public Supply	240	Twin Mountains Formation	322757	974702	Withdrawal of Water
3234307	Hood	Sky Harbour WSC	Public Supply	215	Twin Mountains Formation	322938	974628	Withdrawal of Water
3234308	Hood	Mallard Pointe on Lake	Public Supply	400	Twin Mountains Formation	322738	974503	Withdrawal of Water
3234309	Hood	Sky Harbour WSC	Public Supply	310	Twin Mountains Formation	322946	974601	Withdrawal of Water
3234401	Hood	City of Granbury	Public Supply	120	Twin Mountains Formation	322727	975034	Withdrawal of Water
3234402	Hood	Rolling Hills Mobil	Unused	244	Twin Mountains Formation	322530	975037	Withdrawal of Water
3234403	Hood	Rolling Hills Mobil	Public Supply	250	Twin Mountains Formation	322529	975039	Withdrawal of Water
3234404	Hood	Boswell Water Co.	Public Supply	105	Twin Mountains Formation	322727	975059	Withdrawal of Water

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Table 2.4.12-201 (Sheet 5 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3234405	Hood	Countryside Trailer	Public Supply	220	Twin Mountains Formation	322543	975005	Withdrawal of Water
3234501	Hood	City of Granbury	Public Supply	202	Twin Mountains Formation	322624	974746	Withdrawal of Water
3234502	Hood	City of Granbury	Public Supply	186	Twin Mountains Formation	322634	974805	Withdrawal of Water
3234503	Hood	City of Granbury	Unused	193	Twin Mountains Formation	322645	974813	Withdrawal of Water
3234504	Hood	Texas Highway Dept.	Industrial	200	Twin Mountains Formation	322543	974740	Withdrawal of Water
3234505	Hood	Hood County Feeders	Industrial	260	Twin Mountains Formation	322637	974946	Withdrawal of Water
3234506	Hood	Hood County Feeders	Industrial	258	Twin Mountains Formation	322637	974946	Withdrawal of Water
3234507	Hood	Mooreland Water Co.	Public Supply	270	Twin Mountains Formation	322535	974902	Withdrawal of Water
3234508	Hood	Mooreland Water Co.	Public Supply	270	Twin Mountains Formation	322536	974900	Withdrawal of Water
3234509	Hood	Mooreland Water Co.	Public Supply	280	Twin Mountains Formation	322534	974904	Withdrawal of Water
3234510	Hood	Mooreland Water Co.	Public Supply	270	Twin Mountains Formation	322537	974858	Withdrawal of Water
3234511	Hood	Mooreland Water Co.	Public Supply	280	Twin Mountains Formation	322516	974857	Withdrawal of Water
3234512	Hood	Mooreland Water Co.	Public Supply	260	Twin Mountains Formation	322539	974914	Withdrawal of Water
3234513	Hood	Mooreland Water Co.	Public Supply	225	Twin Mountains Formation	322556	974916	Withdrawal of Water
3234514	Hood	S & W Water Co.	Public Supply	200	Twin Mountains Formation	322557	974743	Withdrawal of Water
3234515	Hood	Hood County Jail	Industrial	225	Twin Mountains Formation	322640	974814	Withdrawal of Water
3234601	Hood	City of Granbury	Public Supply	175	Twin Mountains Formation	322643	974704	Withdrawal of Water
3234602	Hood	City of Granbury	Public Supply	225	Twin Mountains Formation	322705	974712	Withdrawal of Water
3234603	Hood	City of Granbury	Public Supply	200	Twin Mountains Formation	322658	974700	Withdrawal of Water
3234604	Hood	City of Granbury	Public Supply	205	Twin Mountains Formation	322655	974656	Withdrawal of Water
3234605	Hood	City of Granbury	Unused	685	Twin Mountains Formation	322650	974704	Withdrawal of Water
3234606	Hood	City of Granbury	Public Supply	175	Twin Mountains Formation	322647	974709	Withdrawal of Water
3234607	Hood	City of Granbury	Public Supply	175	Twin Mountains Formation	322648	974706	Withdrawal of Water

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Table 2.4.12-201 (Sheet 6 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3234608	Hood	City of Granbury	Unused	160	Twin Mountains Formation	322649	974704	Withdrawal of Water
3234609	Hood	City of Granbury	Public Supply	250	Twin Mountains Formation	322627	974551	Withdrawal of Water
3234610	Hood	City of Granbury	Public Supply	256	Twin Mountains Formation	322631	974522	Withdrawal of Water
3234611	Hood	City of Granbury	Public Supply	300	Twin Mountains Formation	322639	974503	Withdrawal of Water
3234612	Hood	City of Granbury	Unused	677	Twin Mountains Formation	322647	974705	Withdrawal of Water
3234613	Hood	City of Granbury	Public Supply	211	Twin Mountains Formation	322653	974650	Withdrawal of Water
3234614	Hood	City of Granbury	Public Supply	225	Twin Mountains Formation	322704	974653	Withdrawal of Water
3234615	Hood	Southwest Water	Public Supply	379	Twin Mountains Formation	322559	974510	Withdrawal of Water
3234616	Hood	Stum's Wholesale	Industrial	185	Twin Mountains Formation	322633	974655	Withdrawal of Water
3234617	Hood	Joe Noah	Industrial	176	Twin Mountains Formation	322605	974647	Withdrawal of Water
3234618	Hood	Ingram Enterprises	Industrial	300	Twin Mountains Formation	322620	974511	Withdrawal of Water
3234619	Hood	Southwest Water	Public Supply	330	Twin Mountains Formation	322614	974514	Withdrawal of Water
3234620	Hood	Thrift Mart Co-op	Public Supply	320	Twin Mountains Formation	322626	974516	Withdrawal of Water
3234621	Hood	City of Granbury	Public Supply	208	Twin Mountains Formation	322622	974600	Withdrawal of Water
3234622	Hood	The Shores Utility	Public Supply	200	Twin Mountains Formation	322558	974644	Withdrawal of Water
3234623	Hood	The Shores Utility	Public Supply	200	Twin Mountains Formation	322552	974640	Withdrawal of Water
3234624	Hood	Southwest Water	Public Supply	386	Twin Mountains Formation	322600	974511	Withdrawal of Water
3234625	Hood	Southwest Water	Unused	370	Twin Mountains Formation	322558	974529	Withdrawal of Water
3234701	Hood	William L. Schomers	Domestic	317	Twin Mountains Formation	322240	975045	Withdrawal of Water
3234702	Hood	City of Granbury	Public Supply	425	Twin Mountains Formation	322445	975021	Withdrawal of Water
3234801	Hood	Ned Davis	Domestic	300	Twin Mountains Formation	322341	974859	Withdrawal of Water
3234803	Hood	Bob Westvold	Domestic	130	Paluxy Sand	322235	974836	Withdrawal of Water
3234804	Hood	Resort Water Services	Public Supply	280	Twin Mountains Formation	322435	974808	Withdrawal of Water

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Table 2.4.12-201 (Sheet 7 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3234805	Hood	Jerry Barrett	Domestic	350	Twin Mountains Formation	322257	974929	Withdrawal of Water
3234806	Hood	Jerry Barrett	Unused	44	Paluxy Sand	322254	974929	Withdrawal of Water
3234807	Hood	Warren Massey	Domestic	365	Twin Mountains Formation	322251	974939	Withdrawal of Water
3234808	Hood	J. Benefield	Domestic	380	Twin Mountains Formation	322236	974926	Withdrawal of Water
3234809	Hood	James Reed	Domestic	354	Twin Mountains Formation	322249	974949	Withdrawal of Water
3234810	Hood	Jesse Martin	Unused	27	Paluxy Sand	322250	974946	Withdrawal of Water
3234811	Hood	Lewis Allen	Domestic	400	Twin Mountains Formation	322312	974857	Withdrawal of Water
3234812	Hood	Forrest Carter	Domestic	375	Twin Mountains Formation	322327	974939	Withdrawal of Water
3234901	Hood	Acton MUD #13	Public Supply	430	Twin Mountains Formation	322414	974633	Withdrawal of Water
3234902	Hood	Southwest Water	Public Supply	320	Twin Mountains Formation	322421	974724	Withdrawal of Water
3234903	Hood	Southwest Water	Public Supply	317	Twin Mountains Formation	322431	974630	Withdrawal of Water
3234904	Hood	Southwest Water	Public Supply	365	Twin Mountains Formation	322424	974627	Withdrawal of Water
3234905	Hood	Tarrant Utility Co.	Unused	265	Twin Mountains Formation	322442	974538	Withdrawal of Water
3234906	Hood	Acton Mun. Util. Dist.	Unused	398	Twin Mountains Formation	322441	974540	Withdrawal of Water
3234907	Hood	Acton Mun. Util. Dist.	Public Supply	535	Twin Mountains Formation	322441	974540	Withdrawal of Water
3234908	Hood	Hood County Water Co.	Public Supply	557	Twin Mountains Formation	322352	974658	Withdrawal of Water
3234909	Hood	Hood County Water Co.	Public Supply	505	Twin Mountains Formation	322353	974702	Withdrawal of Water
3234910	Hood	Hood County Water Co.	Public Supply	378	Twin Mountains Formation	322354	974700	Withdrawal of Water
3234911	Hood	Western Resort Prop.	Public Supply	364	Twin Mountains Formation	322441	974545	Withdrawal of Water
3234912	Hood	Western Resort Prop.	Public Supply	572	Twin Mountains Formation	322411	974648	Withdrawal of Water
3234913	Hood	Rock Harbor Estates	Public Supply	265	Twin Mountains Formation	322444	974639	Withdrawal of Water
3234914	Hood	Scenic View Estates	Public Supply	123	Twin Mountains Formation	322452	974710	Withdrawal of Water
3235101	Hood	Ed Lawrence	Industrial	384	Twin Mountains Formation	322814	974316	Withdrawal of Water

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Table 2.4.12-201 (Sheet 8 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3235102	Hood	A.V. Almy	Domestic	329	Twin Mountains Formation	322902	974346	Withdrawal of Water
3235103	Hood	J.C. Terrell	Domestic	335	Twin Mountains Formation	322757	974325	Withdrawal of Water
3235104	Hood	G.H. Chase	Domestic	292	Twin Mountains Formation	322754	974302	Withdrawal of Water
3235105	Hood	Granbury Water Service	Public Supply	425	Twin Mountains Formation	322735	974300	Withdrawal of Water
3235106	Hood	Lewis Byers	Industrial	380	Twin Mountains Formation	322742	974307	Withdrawal of Water
3235107	Hood	Jean Davis	Irrigation	445	Twin Mountains Formation	322907	974313	Withdrawal of Water
3235108	Hood	Jean Davis	Irrigation	385	Twin Mountains Formation	322908	974333	Withdrawal of Water
3235109	Hood	Jean Davis	Domestic	405	Twin Mountains Formation	322904	974312	Withdrawal of Water
3235110	Hood	Sands Butane Co.	Domestic	380	Twin Mountains Formation	322744	974307	Withdrawal of Water
3235111	Hood	Gran-Tex Land and	Industrial	390	Twin Mountains Formation	322743	974305	Withdrawal of Water
3235112	Hood	Granbury Water Service	Public Supply	425	Twin Mountains Formation	322734	974256	Withdrawal of Water
3235113	Hood	Hood County Water Co.	Public Supply	398	Twin Mountains Formation	322850	974354	Withdrawal of Water
3235114	Hood	Hood County Water Co.	Public Supply	400	Twin Mountains Formation	322850	974422	Withdrawal of Water
3235115	Hood	Hood County Water Co.	Public Supply	420	Twin Mountains Formation	322924	974326	Withdrawal of Water
3235116	Hood	H2M Water Services	Public Supply	408	Twin Mountains Formation	322810	974312	Withdrawal of Water
3235117	Hood	Waples Baptist Church	Public Supply	390	Twin Mountains Formation	322849	974321	Withdrawal of Water
3235118	Hood	Nolan Creek Estates	Public Supply	410	Twin Mountains Formation	322828	974355	Withdrawal of Water
3235119	Hood	Mallard Pointe on Lake	Public Supply	390	Twin Mountains Formation	322742	974459	Withdrawal of Water
3235120	Hood	Mallard Pointe on Lake	Public Supply	370	Twin Mountains Formation	322742	974453	Withdrawal of Water
3235121	Hood	Mallard Pointe on Lake	Public Supply	370	Twin Mountains Formation	322745	974449	Withdrawal of Water
3235201	Hood	Acton Mun. Util. Dist.	Public Supply	540	Twin Mountains Formation	322838	974116	Withdrawal of Water
3235202	Hood	Acton Mun. Util. Dist.	Public Supply	160	Paluxy Sand	322838	974116	Withdrawal of Water
3235203	Hood	Acton Mun. Util. Dist.	Public Supply	90	Paluxy Sand	322807	974155	Withdrawal of Water

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Table 2.4.12-201 (Sheet 9 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3235204	Hood	Acton Mun. Util. Dist.	Public Supply	440	Twin Mountains Formation	322807	974155	Withdrawal of Water
3235205	Hood	Acton Mun. Util. Dist.	Public Supply	640	Twin Mountains Formation	322804	974107	Withdrawal of Water
3235206	Hood	Acton Mun. Util. Dist.	Public Supply	113	Paluxy Sand	322804	974107	Withdrawal of Water
3235401	Hood	Hanco Inc.	Public Supply	387	Twin Mountains Formation	322532	974351	Withdrawal of Water
3235402	Hood	El Brazos Apartments	Public Supply	312	Twin Mountains Formation	322657	974433	Withdrawal of Water
3235403	Hood	Boy Scouts of America	Public Supply	397	Twin Mountains Formation	322558	974456	Withdrawal of Water
3235404	Hood	L.E. Massengale	Domestic	324	Twin Mountains Formation	322614	974424	Withdrawal of Water
3235405	Hood	R.E. Stephens	Industrial	290	Twin Mountains Formation	322654	974428	Withdrawal of Water
3235406	Hood	Hanco Inc.	Public Supply	350	Twin Mountains Formation	322648	974342	Withdrawal of Water
3235407	Hood	Hanco Inc.	Public Supply	415	Twin Mountains Formation	322532	974351	Withdrawal of Water
3235408	Hood	Hood County Water Co.	Public Supply	445	Twin Mountains Formation	322708	974351	Withdrawal of Water
3235409	Hood	Shady Oak Estates	Public Supply	350	Twin Mountains Formation	322626	974418	Withdrawal of Water
3235410	Hood	Charlie & Georges	Public Supply	360	Twin Mountains Formation	322653	974431	Withdrawal of Water
3235411	Hood	Hood County Water Co.	Public Supply	328	Twin Mountains Formation	322508	974318	Withdrawal of Water
3235501	Hood	Acton MUD	Public Supply	395	Twin Mountains Formation	322520	974021	Withdrawal of Water
3235502	Hood	Hanco Inc.	Public Supply	330	Twin Mountains Formation	322538	974216	Withdrawal of Water
3235503	Hood	Acton MUD	Public Supply	379	Twin Mountains Formation	322613	974203	Withdrawal of Water
3235504	Hood	Green Meadows Mobile	Public Supply	435	Twin Mountains Formation	322659	974107	Withdrawal of Water
3235505	Hood	Acton Mun. Util. Dist.	Public Supply	620	Twin Mountains Formation	322645	974004	Withdrawal of Water
3235601	Hood	C.T. Sharp	Domestic	155	Paluxy Sand	322640	973928	Withdrawal of Water
3235602	Hood	Acton Mun. Util. Dist.	Public Supply	520	Twin Mountains Formation	322552	973948	Withdrawal of Water
3235701	Hood	Woddy Oliver	Domestic	250	Twin Mountains Formation	322433	974306	Withdrawal of Water
3235702	Hood	Jackson Heights Mobile	Public Supply	342	Twin Mountains Formation	322432	974407	Withdrawal of Water

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Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3235703	Hood	C.F. Sealey	Public Supply	356	Twin Mountains Formation	322256	974257	Withdrawal of Water
3235704	Hood	Hood Co. Utilities	Public Supply	340	Twin Mountains Formation	322312	974319	Withdrawal of Water
3235705	Hood	Hood County Water Co.	Public Supply	452	Twin Mountains Formation	322249	974406	Withdrawal of Water
3235706	Hood	Canyon Creek Estates	Public Supply	355	Twin Mountains Formation	322341	974429	Withdrawal of Water
3235707	Hood	Canyon Creek Estates	Public Supply	320	Twin Mountains Formation	322336	974435	Withdrawal of Water
3235708	Hood	Canyon Creek Estates	Public Supply	509	Twin Mountains Formation	322334	974418	Withdrawal of Water
3235709	Hood	Boynton Water Supply	Public Supply	365	Twin Mountains Formation	322431	974410	Withdrawal of Water
3235801	Hood	- -Henslee	Stock	0	Paluxy Sand	322357	974138	Spring
3235802	Hood	Texas Power and Light	Industrial	325	Twin Mountains Formation	322409	974205	Withdrawal of Water
3235803	Hood	Texas Power and Light	Industrial	335	Twin Mountains Formation	322415	974155	Withdrawal of Water
3235804	Hood	Hood Co. Utilities	Public Supply	367	Twin Mountains Formation	322317	974146	Withdrawal of Water
3235805	Hood	Hood Co. Utilities	Unused	380	Twin Mountains Formation	322331	974156	Withdrawal of Water
3235806	Hood	Acton Mun. Util. Dist.	Public Supply	600	Twin Mountains Formation	322457	974004	Withdrawal of Water
3235901	Hood	A.J. Purselley	Domestic	457	Twin Mountains Formation	322334	973815	Withdrawal of Water
3235902	Hood	R.A. Massey	Domestic	45	Paluxy Sand	322438	973808	Withdrawal of Water
3235903	Hood	Camp El Tesoro	Unused	348	Twin Mountains Formation	322328	973901	Withdrawal of Water
3235904	Hood	Acton MUD	Public Supply	413	Twin Mountains Formation	322343	973929	Withdrawal of Water
3235905	Hood	Hood Co. Utilities	Public Supply	390	Twin Mountains Formation	322307	973936	Withdrawal of Water
3241101	Hood	P.W. Gage	Domestic	108	Twin Mountains Formation	322107	975916	Withdrawal of Water
3241102	Hood	Stanley Allen	Domestic	140	Twin Mountains Formation	322024	975913	Withdrawal of Water
3241201	Hood	B.E. Wood	Domestic	45	Paluxy Sand	322059	975557	Withdrawal of Water
3241301	Hood	Rufus Vest	Domestic	285	Twin Mountains Formation	322106	975354	Withdrawal of Water
3241402	Hood	Steve Griffith	Irrigation	180	Twin Mountains Formation	321951	975839	Withdrawal of Water

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Table 2.4.12-201 (Sheet 11 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3241501	Hood	R.B. Caraway	Domestic	0	Twin Mountains Formation	321811	975709	Spring
3241601	Hood	J.H. Woods	Domestic	260	Twin Mountains Formation	321831	975303	Withdrawal of Water
3241602	Hood	A. Heathington	Domestic	310	Twin Mountains Formation	321808	975252	Withdrawal of Water
3241801	Hood	F.A. Troutman	Domestic	250	Twin Mountains Formation	321622	975653	Withdrawal of Water
3241802	Hood	J.C. Manley	Domestic	165	Twin Mountains Formation	321720	975649	Withdrawal of Water
3241803	Hood	H.L. Seale Ranch	Domestic	343	Twin Mountains Formation	321510	975607	Withdrawal of Water
3241901	Hood	Paluxy Baptist Church	Domestic	169	Twin Mountains Formation	321616	975428	Withdrawal of Water
3241903	Hood	De Soto Oil Co.	Unused	5082	Aquifer Code Not Applicable	321545	975414	Oil or Gas
3242101	Hood	J.R. Gauntt	Domestic	331	Twin Mountains Formation	322212	975021	Withdrawal of Water
3242202	Hood	Mid-Continent Pet. Co.	Unused	5577	Aquifer Code Not Applicable	322120	974957	Oil or Gas
3242203	Hood	Elsie Holden	Domestic	344	Twin Mountains Formation	322221	974916	Withdrawal of Water
3242301	Hood	- -	Domestic	300	Travis Peak Formation	322204	974637	Withdrawal of Water
3242302	Hood	J.L. Wiggins	Domestic	396	Twin Mountains Formation	322151	974629	Withdrawal of Water
3242303	Hood	A.J. Kiesling	Unused	350	Twin Mountains Formation	322218	974510	Withdrawal of Water
3242401	Hood	J.T. Parker	Domestic	352	Twin Mountains Formation	321830	975052	Withdrawal of Water
3242402	Hood	T.W. Couch	Domestic	335	Twin Mountains Formation	321852	975107	Withdrawal of Water
3242403	Hood	A.L. Hurley	Domestic	355	Twin Mountains Formation	321917	975216	Withdrawal of Water
3242604	Hood	Texas Utilities	Unused	470	Twin Mountains Formation	321910	974655	Observation
3243101	Hood	J.J. Purselley	Domestic	335	Twin Mountains Formation	322222	974452	Withdrawal of Water
3243102	Hood	B.W. Fitzgerald	Unused	4503	Aquifer Code Not Applicable	322133	974449	Oil or Gas
3243103	Hood	V. D. Wheeler	Domestic	360	Twin Mountains Formation	322009	974425	Withdrawal of Water
3243104	Hood	David Wheeler	Irrigation	500	Twin Mountains Formation	322004	974427	Withdrawal of Water
3243105	Hood	McKee Water Services	Public Supply	376	Twin Mountains Formation	322112	974308	Withdrawal of Water

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Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3243201	Hood	H. Zweifel	Stock	185	Twin Mountains Formation	322150	974140	Withdrawal of Water
3243202	Hood	Acton MUD	Public Supply	371	Twin Mountains Formation	322207	974129	Withdrawal of Water
3243203	Hood	Acton Mun. Util. Dist.	Public Supply	393	Twin Mountains Formation	322157	974033	Withdrawal of Water
3243204	Hood	Acton Mun. Util. Dist.	Public Supply	560	Twin Mountains Formation	322158	974045	Withdrawal of Water
3243205	Hood	Acton Mun. Util. Dist.	Unused	572	Twin Mountains Formation	322158	974045	Test Hole
3243206	Hood	Acton MUD	Public Supply	500	Twin Mountains Formation	322047	974037	Withdrawal of Water
3243207	Hood		Not Listed	0	Alluvium	322031	974031	Spring
3243301	Hood	Acton Mun. Util. Dist.	Public Supply	570	Twin Mountains Formation	322159	973928	Withdrawal of Water
3243302	Hood	Acton MUD	Public Supply	530	Twin Mountains Formation	322222	973956	Withdrawal of Water
3243303	Hood	Acton MUD	Public Supply	588	Twin Mountains Formation	322127	973801	Withdrawal of Water
3249201	Hood	H.L. Seale Ranch	Domestic	252	Twin Mountains Formation	321459	975636	Withdrawal of Water
3241902	Somervell	N. B. Sanderson	Domestic	288	Twin Mountains Formation	321518	975314	Withdrawal of Water
3242501	Somervell	Bert Willie	Unused	300	Twin Mountains Formation	321738	974930	Withdrawal of Water
3242502	Somervell	J.C. Ice	Domestic	352	Twin Mountains Formation	321807	974853	Withdrawal of Water
3242503	Somervell	Texas Utilities	Industrial	517	Twin Mountains Formation	321802	974826	Withdrawal of Water
3242504	Somervell	Texas Utilities	Public Supply	400	Twin Mountains Formation	321802	974822	Withdrawal of Water
3242601	Somervell	Texas Utilities	Industrial	466	Twin Mountains Formation	321745	974723	Withdrawal of Water
3242602	Somervell	Texas Utilities	Industrial	490	Twin Mountains Formation	321751	974649	Withdrawal of Water
3242603	Somervell	Texas Utilities	Industrial	471	Twin Mountains Formation	321858	974656	Withdrawal of Water
3242701	Somervell	- - Matheny	Domestic	130	Glen Rose Limestone	321521	975109	Withdrawal of Water
3242801	Somervell	L. P. Jones	Domestic	352	Twin Mountains Formation	321642	974845	Withdrawal of Water
3242802	Somervell	Oak Grove Sub-div.	Public Supply	360	Twin Mountains Formation	321725	974835	Withdrawal of Water
3242803	Somervell	Oak Grove Sub-div.	Public Supply	360	Twin Mountains Formation	321725	974835	Withdrawal of Water

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Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3242804	Somervell	Scruggs Mobile Home Pk	Public Supply	420	Twin Mountains Formation	321656	974832	Withdrawal of Water
3242901	Somervell	G. A. Jackson	Stock	350	Twin Mountains Formation	321714	974522	Withdrawal of Water
3242902	Somervell	Texas Utilities	Unused	318	Twin Mountains Formation	321709	974513	Withdrawal of Water
3242903	Somervell	Texas Utilities	Unused	479	Twin Mountains Formation	321651	974623	Withdrawal of Water
3242904	Somervell	City of Glen Rose	Public Supply	500	Twin Mountains Formation	321545	974512	Withdrawal of Water
3242905	Somervell	Texas Ampitheater	Public Supply	340	Twin Mountains Formation	321546	974510	Withdrawal of Water
3243401	Somervell	D. Trembly	Domestic	330	Hensell Sand Member of Travis Peak Formation	321830	974409	Withdrawal of Water
3243402	Somervell	B. B. Halbert	Stock	200	Hensell Sand Member of Travis Peak Formation	321857	974339	Withdrawal of Water
3243403	Somervell	F. E. Miller	Domestic	140	Hensell Sand Member of Travis Peak Formation	321848	974238	Withdrawal of Water
3243404	Somervell	I. W. Keller	Stock	200	Hensell Sand Member of Travis Peak Formation	321838	974332	Withdrawal of Water
3243405	Somervell	I. W. Keller	Stock	0	Alluvium	321815	974248	Spring
3243406	Somervell	J. D. Hardy	Unused	212	Twin Mountains Formation	321918	974241	Withdrawal of Water
3243407	Somervell	Ri-Mac Development	Public Supply	383	Twin Mountains Formation	321813	974425	Withdrawal of Water
3243408	Somervell	Harston Gravel Co.	Industrial	340	Twin Mountains Formation	321824	974240	Withdrawal of Water
3243409	Somervell	Harston Gravel Co.	Industrial	450	Twin Mountains Formation	321824	974240	Withdrawal of Water
3243410	Somervell	Alton May	Public Supply	420	Twin Mountains Formation	321744	974305	Withdrawal of Water
3243411	Somervell	Harston Gravel Co.	Industrial	260	Twin Mountains Formation	321828	974251	Withdrawal of Water
3243412	Somervell	Happy Hills Home	Public Supply	378	Twin Mountains Formation	321738	974414	Withdrawal of Water
3243413	Somervell	Happy Hills Home	Public Supply	400	Twin Mountains Formation	321738	974414	Withdrawal of Water
3243414	Somervell	Happy Hills Home	Public Supply	517	Twin Mountains Formation	321738	974414	Withdrawal of Water

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Table 2.4.12-201 (Sheet 14 of 18) Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3243415	Somervell	H2M Water Services	Public Supply	383	Twin Mountains Formation	321831	974432	Withdrawal of Water
3243501	Somervell	Arrowhead Camp	Public Supply	270	Twin Mountains Formation	321828	974219	Withdrawal of Water
3243601	Somervell	Capital Silica Co.	Industrial	285	Hensell Sand Member of Travis Peak Formation	321747	973736	Withdrawal of Water
3243701	Somervell	W. H. Howth	Domestic	230	Travis Peak Formation	321633	974356	Withdrawal of Water
3243702	Somervell	Squaw Creek Cemetary	Irrigation	359	Travis Peak Formation	321547	974310	Withdrawal of Water
3243703	Somervell	W. B. Stewart	Domestic	374	Glen Rose Limestone and Twin Mountains Formation	321605	974259	Withdrawal of Water
3243704	Somervell	Mark Dodson	Irrigation	390	Twin Mountains Formation	321522	974307	Withdrawal of Water
3243705	Somervell	Tres Rios Estates	Public Supply	360	Twin Mountains Formation	321513	974305	Withdrawal of Water
3243801	Somervell	George Day	Stock	260	Twin Mountains Formation	321655	974115	Withdrawal of Water
3243802	Somervell	Shackelford Est.	Stock	256	Twin Mountains Formation	321631	974012	Withdrawal of Water
3243803	Somervell	F. Williams	Stock	260	Twin Mountains Formation	321601	974007	Withdrawal of Water
3243804	Somervell	J. M. West	Unused	260	Twin Mountains Formation	321544	974050	Withdrawal of Water
3243805	Somervell	E. J. Doughty	Domestic	464	Twin Mountains Formation	321520	974136	Withdrawal of Water
3243806	Somervell	Mrs.W. H. White	Not Listed	0	Glen Rose Limestone	321536	974203	Spring
3243807	Somervell	K-B Oil Co.---	Unused	4213	Aquifer Code Not Applicable	321634	974210	Oil or Gas
3243808	Somervell	J. H. Shook	Domestic	200	Glen Rose Limestone	321544	974222	Withdrawal of Water
3243809	Somervell	Derbie Schackleford	Unused	253	Twin Mountains Formation	321631	974055	Withdrawal of Water
3243810	Somervell	M & W Ranch	Irrigation	420	Twin Mountains Formation	321514	974222	Withdrawal of Water
3243811	Somervell	Tres Rios	Domestic	380	Twin Mountains Formation	321602	974207	Withdrawal of Water
3243812	Somervell	Oak River Ranch	Public Supply	500	Twin Mountains Formation	321536	974023	Withdrawal of Water
3243813	Somervell	John Pugh	Domestic	302	Twin Mountains Formation	321613	974021	Withdrawal of Water

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Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3243901	Somervell	Texas Cedar Oil Co.	Industrial	380	Hensell Sand Member of Travis Peak Formation	321637	973906	Withdrawal of Water
3243902	Somervell	Georges Creek Church	Public Supply	147	Paulxy Sand	321729	973806	Withdrawal of Water
3243903	Somervell	Stevens Ranch on Brazo	Unused	470	Twin Mountains Formation	321545	973928	Withdrawal of Water
3243904	Somervell	Stevens Ranch on Brazo	Public Supply	645	Twin Mountains Formation	321547	973929	Withdrawal of Water
3249301	Somervell	J. P. Morrow	Irrigation	298	Twin Mountains Formation	321442	975326	Withdrawal of Water
3249601	Somervell	J. W. Tottenham	Stock	345	Hensell Sand Member of Travis Peak Formation	321203	975355	Withdrawal of Water
3249901	Somervell	C. A. Rogers	Stock	281	Paulxy Sand	320959	975347	Withdrawal of Water
3250101	Somervell	D. H. Smith	Stock	273	Twin Mountains Formation	321446	975102	Withdrawal of Water
3250102	Somervell	Gene Ratliff	Domestic	275	Twin Mountains Formation	321417	975004	Withdrawal of Water
3250103	Somervell	Roy Kenedy and E. H.	Irrigation	300	Travis Peak Formation	321456	975006	Withdrawal of Water
3250201	Somervell	L. H. Daniels	Stock	176	Glen Rose Limestone	321421	974941	Withdrawal of Water
3250202	Somervell	W. A. Wood	Unused	297	Twin Mountains Formation	321425	974942	Withdrawal of Water
3250203	Somervell	Travis Wooley	Unused	284	Travis Peak Formation	321418	974927	Withdrawal of Water
3250204	Somervell	J. O. Pruitt	Stock	135	Twin Mountains Formation	321437	974834	Withdrawal of Water
3250205	Somervell	C. C. Moss	Domestic	143	Twin Mountains Formation	321350	974842	Withdrawal of Water
3250206	Somervell	J. O. Pruitt	Domestic	120	Twin Mountains Formation	321351	974813	Withdrawal of Water
3250207	Somervell	W. M. Spoonmore	Stock	125	Twin Mountains Formation	321401	974743	Withdrawal of Water
3250208	Somervell	Tx Parks and Wildlife	Public Supply	354	Twin Mountains Formation	321448	974858	Withdrawal of Water
3250209	Somervell	American Legion	Public Supply	170	Glen Rose Limestone	321347	974747	Withdrawal of Water
3250301	Somervell	W. M. Spoonmore	Stock	317	Twin Mountains Formation	321404	974721	Withdrawal of Water

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Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3250302	Somervell	W. M. Spoonmore	Domestic	567	Twin Mountains Formation	321434	974722	Withdrawal of Water
3250303	Somervell	City of Glen Rose	Public Supply	325	Hosston Formation	321410	974524	Withdrawal of Water
3250304	Somervell	City of Glen Rose	Public Supply	352	Travis Peak Formation	321401	974508	Withdrawal of Water
3250305	Somervell	T. K. Blalock	Domestic	120	Glen Rose Limestone	321348	974510	Withdrawal of Water
3250306	Somervell	Mrs. Nix	Not Listed	186	Glen Rose Limestone	321406	974541	Withdrawal of Water
3250307	Somervell	J. B. Young	Not Listed	177	Glen Rose Limestone	321359	974525	Withdrawal of Water
3250308	Somervell	T. W. Garner	Domestic	140	Glen Rose Limestone	321342	974513	Withdrawal of Water
3250309	Somervell	City of Glen Rose	Public Supply	472	Hosston Formation	321421	974558	Withdrawal of Water
3250310	Somervell	Glen Rose Public	Irrigation	370	Twin Mountains Formation	321350	974603	Withdrawal of Water
3250311	Somervell	Kirk Estates	Public Supply	280	Glen Rose Limestone	321320	974639	Withdrawal of Water
3250312	Somervell	Kirk Estates	Public Supply	280	Glen Rose Limestone	321320	974637	Withdrawal of Water
3250313	Somervell	Kirk Estates	Public Supply	260	Glen Rose Limestone	321347	974626	Withdrawal of Water
3250314	Somervell	Paluxy Estates	Public Supply	280	Glen Rose Limestone	321329	974517	Withdrawal of Water
3250315	Somervell	Paluxy Estates	Public Supply	400	Glen Rose Limestone	321318	974517	Withdrawal of Water
3250316	Somervell	Somervell Co. Courthos	Public Supply	200	Glen Rose Limestone	321405	974521	Withdrawal of Water
3250317	Somervell	Sunset Park	Public Supply	320	Twin Mountains Formation	321406	974657	Withdrawal of Water
3250401	Somervell	W. A. Schmidt	Unused	328	Twin Mountains Formation	321201	975026	Withdrawal of Water
3250402	Somervell	Otis Shipman	Domestic	372	Twin Mountains Formation	321121	975208	Withdrawal of Water
3250501	Somervell	J. E. Jackson	Domestic	297	Twin Mountains Formation	321152	974833	Withdrawal of Water
3250502	Somervell	Whitaker and Whitaker	Unused	2421	Aquifer Code Not Applicable	321116	974816	Oil or Gas
3250503	Somervell	Kelley Lewellen	Domestic	347	Twin Mountains Formation	321154	974822	Withdrawal of Water
3250504	Somervell	Cedar Ridge	Public Supply	530	Twin Mountains Formation	321153	974826	Withdrawal of Water
3250505	Somervell	Fossil Rim Wildlife	Public Supply	500	Twin Mountains Formation	321028	974739	Withdrawal of Water

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Table 2.4.12-201 (Sheet 17 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3250506	Somervell	Fossil Rim Wildlife	Public Supply	76	Paulxy Sand	321036	974803	Withdrawal of Water
3250601	Somervell	- -Shelton	Domestic	110	Paulxy Sand	321149	974503	Withdrawal of Water
3250701	Somervell	Cedar Valley Ranch	Domestic	510	Hensell Sand Member of Travis Peak Formation	320959	975111	Withdrawal of Water
3250801	Somervell	C. D. Montgomery	Domestic	225	Paulxy Sand	320755	974926	Withdrawal of Water
3250802	Somervell	Fossil Rim Wildlife	Domestic	321	Paulxy Sand	320924	974741	Withdrawal of Water
3250803	Somervell	Fossil Rim Wildlife	Public Supply	714	Hosston Formation	320939	974743	Withdrawal of Water
3250804	Somervell	Fossil Rim Wildlife	Public Supply	560	Hosston Formation	320948	974834	Withdrawal of Water
3250805	Somervell	Fossil Rim Wildlife	Public Supply	820	Hosston Formation	320938	974741	Withdrawal of Water
3250901	Somervell	Benedum Trees Oil Co.	Unused	3625	Aquifer Code Not Applicable	320912	974643	Oil or Gas
3251101	Somervell	Camp Tres Rios	Public Supply	277	Hensell Sand Member of Travis Peak Formation	321452	974312	Withdrawal of Water
3251102	Somervell	M. E. Davis	Unused	6505	Aquifer Code Not Applicable	321254	974435	Oil or Gas
3251103	Somervell	- -Bartlett	Not Listed	128	Glen Rose Limestone	321434	974447	Withdrawal of Water
3251104	Somervell	Bill Walker	Domestic	376	Twin Mountains Formation	321231	974455	Withdrawal of Water
3251105	Somervell	City of Glen Rose	Public Supply	484	Twin Mountains Formation	321341	974449	Withdrawal of Water
3251106	Somervell	Glen Lake Methodist	Irrigation	348	Twin Mountains Formation	321418	974435	Withdrawal of Water
3251107	Somervell		Not Listed	0	Aquifer Not Listed	321438	974429	Spring
3251108	Somervell	City of Glen Rose	Public Supply	410	Hensell Sand Member of Travis Peak Formation	321453	974357	Withdrawal of Water
3251201	Somervell	W. L. Lilly	Stock	187	Hensell Sand Member of Travis Peak Formation	321428	974215	Withdrawal of Water
3251202	Somervell	C. L. Oldham	Domestic	240	Hensell Sand Member of Travis Peak Formation	321440	974128	Withdrawal of Water

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Table 2.4.12-201 (Sheet 18 of 18)
Hood and Somervell County Water Well Information

Well Number	County	Owner	Primary Use	Well Depth	Aquifer	Latitude	Longitude	Well Type
3251203	Somervell	J. E. Turner	Unused	221	Hensell Sand Member of Travis Peak Formation	321408	974059	Withdrawal of Water
3251204	Somervell	Harry Dennis	Stock	125	Glen Rose Limestone	321353	974049	Withdrawal of Water
3251205	Somervell	V. M. Reeves	Domestic	425	Travis Peak Formation	321330	974119	Withdrawal of Water
3251206	Somervell	Sandlin Est.	Stock	240	Twin Mountains Formation	321259	974107	Withdrawal of Water
3251207	Somervell	G. T. Stevens	Domestic	431	Hosston Formation	321241	974145	Withdrawal of Water
3251208	Somervell	M & W Ranch	Irrigation	410	Twin Mountains Formation	321456	974209	Withdrawal of Water
3251209	Somervell	M & W Ranch	Irrigation	410	Twin Mountains Formation	321447	974229	Withdrawal of Water
3251301	Somervell	Clark Hedrick	Unused	91	Paulxy Sand	321435	973922	Withdrawal of Water
3251302	Somervell	T. T. Mullins	Stock	211	Twin Mountains Formation	321246	973919	Withdrawal of Water
3251501	Somervell	H. C. Polley	Unused	370	Hensell Sand Member of Travis Peak Formation	321104	974129	Withdrawal of Water
3251502	Somervell	Tarrant Baptist	Public Supply	475	Twin Mountains Formation	321212	974144	Withdrawal of Water
3251503	Somervell	Tarrant Baptist	Public Supply	550	Twin Mountains Formation	321211	974143	Withdrawal of Water
3251601	Somervell	A. E. Smith	Unused	375	Hensell Sand Member of Travis Peak Formation	321220	973739	Withdrawal of Water
3251701	Somervell	D. A. Odom	Unused	119	Paulxy Sand	320906	974456	Withdrawal of Water
3258101	Somervell	W. F. Long	Stock	283	Paulxy Sand	320713	975220	Withdrawal of Water

Source: Reference 2.4-258

CP COL 2.4(1)

Table 2.4.12-202
2003 Groundwater Withdrawal from the Trinity Aquifer by Use Category- Hood and Somervell Counties, Texas

					Self Supplied				
County	Municipal	Manufacturing	Steam Electric	Irrigation	Mining	Livestock	County Total Withdrawal	Trinity Total Withdrawal	Percentage of Total Withdrawal
Hood	5,195	15	43	0	167	309	5,729	172,098	3.33%
Somervell	941	4	28	0	715	38	1,726	172,098	1.00%
Total	6,136	19	71	0	882	347	7,455	172,098	4.33%

Note: All values are in acre feet (ac-ft).

Source: Reference 2.4-257

CP COL 2.4(1)

Table 2.4.12-203
CPNPP Water Well Information

CPNPP Well ID	State Well Number	Location	Primary Use	Well Depth (ft)	Latitude	Longitude	Well Type
1	3242903	Ball Bark Road	Not Used	479	321651	974623	Observation
2	3242902	Training Center	Not Used	318	321707	974515	Observation
3	3242901	Training Center	Public Supply	350	321707	974516	Withdrawal of Water
4	3242601	Batch Plant	Public Supply	466	321748	974733	Withdrawal of Water
5	3242602	Met Tower	Public Supply	490	321750	974650	Withdrawal of Water
6	N/A	Plant Entrance	Not Used	>280(1)	321749	974859	Observation
7	3242503	NOSF - North	Public Supply	517	321760	974828	Withdrawal of Water
8	3242504	NOSF - South	Public Supply	400	321757	974826	Withdrawal of Water
9	3242603	Squaw Creek Park	Public Supply	471	321905	974659	Withdrawal of Water
10	3242604	Squaw Creeak Park	Not Used	470	321905	974660	Observation
11	N/A	Squaw Creek Park Office	Public Supply	Unknown(2)	321946	974648	Withdrawal of Water
12	N/A	Rifle Training Facility	Public Supply	485	321905	974659	Withdrawal of Water

Notes:

Onsite water wells are owned by Luminant and completed in the Twin Mountains (Trinity) Aquifer

(1) Total depth of well is unknown due to obstruction. Static water level has been measured at approximately 280 ft below top of casing.

(2) Inactive public supply well, total depth of well is unknown.

NOSF Nuclear Operations Support Facility

N/A Not Assigned

CP COL 2.4(1)

Table 2.4.12-204
2006 CPNPP Monthly Groundwater Use

Month (2006)	Self Supplied (Gallons)
January	835,600
February	759,800
March	1,050,700
April	904,400
May	688,300
June	762,600
July	697,500
August	679,000
September	628,500
October	930,000
November	568,800
December	587,500
Total	9,092,700

Source: Reference 2.4-217

CP COL 2.4(1)

Table 2.4.12-205
Projected Water Demands for 2010-2060 in Brazos Region G

Water Use Category	Projections					
	2010	2020	2030	2040	2050	2060
Municipal	347,389	397,090	444,820	491,312	542,172	595,482
Manufacturing	19,787	23,201	25,077	26,962	30,191	31,942
Steam-Electric	147,734	158,789	171,489	191,968	219,340	242,344
Mining	36,664	37,591	38,037	27,251	20,744	21,243
Irrigation	232,541	227,697	222,691	217,859	213,055	208,386
Livestock	51,576	51,576	51,576	51,576	51,576	51,576
Total for Region	835,691	895,944	953,690	1,006,928	1,077,078	1,150,973

Note: Demands are in ac-ft/yr

Source: Reference 2.4-208

CP COL 2.4(1)

Table 2.4.12-206
Projected Water Demands for 2010-2060 in Hood and Somervell Counties

	Year						
County	2010	2020	2030	2040	2050	2060	Use
Hood	9,135	10,666	12,077	13,616	15,557	17,897	Municipal
Somervell	1,071	1,145	1,202	1,229	1,238	1,245	
Hood	25	28	30	32	34	37	Manufacturing
Somervell	6	7	8	9	10	11	
Hood	6,594	8,098	9,467	11,137	13,172	15,653	Steam Electric
Somervell	23,200	23,200	23,200	23,200	23,200	23,200	
Hood	162	161	160	159	158	157	Mining
Somervell	304	287	278	270	263	257	
Hood	3,179	3,120	3,062	3,005	2,948	2,893	Irrigation
Somervell	474	471	468	467	464	461	
Hood	623	623	623	623	623	623	Livestock
Somervell	166	166	166	166	166	166	
Hood and Somervell Total	44939.00	47,972	50,741	53,913	57,833	62,600	All Uses

Note: Demands are in ac-ft/yr

Source: Reference 2.4-208

CP COL 2.4(1)

Table 2.4.12-207
2030 and 2060 Water Surplus and Shortage Estimates for Hood and Somervell Counties

Water User Group	County	Surplus/(Shortage) in ac-ft		Comment
		2030	2060	
Acton MUD	Hood	2,347	484	Projected surplus
City of Granbury	Hood	4,888	3,252	Projected surplus
Oak Trail Shores Subdivision	Hood	(114)	(101)	Projected shortage
City of Tolar	Hood	58	62	Projected surplus
County-Other	Hood	(1,195)	(3,543)	Projected shortage
Manufacturing	Hood	(8)	(15)	Projected shortage
Steam-Electric	Hood	33,980	27,794	Projected surplus
Mining	Hood	(25)	(24)	Projected shortage
Irrigation	Hood	10,346	10,628	Projected surplus
Livestock	Hood	0	0	No projected surplus/shortage
City of Glen Rose	Somervell	38	37	Projected surplus
County-Other	Somervell	(231)	(260)	Projected shortage
Manufacturing	Somervell	(4)	(7)	Projected shortage
Steam-Electric	Somervell	25,570	25,510	Projected surplus
Mining	Somervell	(94)	(85)	Projected shortage
Irrigation	Somervell	945	953	Projected surplus
Livestock	Somervell	0	0	Supply equals demand

Source: Reference 2.4-208

CP COL 2.4(1)

Table 2.4.12-208 (Sheet 1 of 2)
Monitoring Well Installation Data

Monitoring Point	Reference Elevation (ft msl)	Ground Elevation (ft msl)	Well Depth (ft bre)	Screen Length (ft)	Top of Screen (ft msl)	Bottom of Screen[(a)] (ft msl)	Boring Depth (ft bgs)
MW-1200b	851.44	848.91	57.92	20	#REF!	796.44	55.40
MW-1200c	851.32	848.97	97.39	20	776.32	756.32	95.40
MW-1201a	866.02	863.19	21.78	10	857.02	847.02	21.00
MW-1201b	865.91	863.15	57.97	20	831.11	811.11	55.00
MW-1201c	865.76	863.08	87.89	20	801.16	781.16	85.00
MW-1202b	855.62	853.57	67.41	20	810.62	790.62	65.40
MW-1202c	856.17	853.86	102.64	20	776.17	756.17	100.00
MW-1203a	862.18	862.44	16.69	5	851.18	846.18	16.40
MW-1203b	861.88	862.08	50.51	20	831.87	811.87	50.40
MW-1203c	862.16	862.42	75.67	20	809.16	789.16	75.40
MW-1204a	844.31	841.87	27.77	10	829.71	819.71	25.00
MW-1204b	845.35	841.88	57.18	25	815.35	790.35	55.40
MW-1204c	844.68	842.18	93.06	20	774.68	754.68	90.00
MW-1205a	860.07	857.61	15.71	5	852.07	847.07	13.35
MW-1205b	860.25	857.97	62.71	20	820.25	800.25	60.40
MW-1205c	859.73	857.45	93.03	20	789.73	769.73	90.40
MW-1206a	835.37	833.13	27.65	10	820.37	810.37	25.40
MW-1206b	835.40	833.20	52.7	20	805.40	785.40	50.40
MW-1206c	836.05	833.08	88.95	20	771.05	751.05	85.40
MW-1207a	851.30	848.95	17.69	5	841.70	836.70	15.00
MW-1207b	851.00	848.40	48.44	20	826.00	806.00	45.00
MW-1207c	851.16	848.57	73.25	20	801.16	781.16	70.00
MW-1208a	820.08	817.43	47.6	20	795.48	775.48	45.00
MW-1209a	811.88	809.21	42.93	20	791.88	771.88	40.40
MW-1209b	811.69	808.66	68.59	20	766.69	746.69	65.40
MW-1209c	811.41	808.45	103.32	20	731.41	711.41	100.40
MW-1210b	830.64	827.97	48.18	20	805.64	785.64	45.00
MW-1210c	830.58	827.92	82.73	20	770.58	750.58	80.00
MW-1211a	813.03	810.38	52.93	20	783.03	763.03	50.00
MW-1211b	813.24	810.57	77.23	25	763.24	738.24	75.00
MW-1212a	822.59	820.04	38.24	15	802.99	787.99	35.00
MW-1212b	822.96	820.27	58.23	15	782.96	767.96	55.00
MW-1212c	822.57	819.93	88.25	20	757.57	737.57	85.00
MW-1213b	848.63	845.92	67.97	20	804.03	784.03	65.00
MW-1213c	848.31	845.55	92.92	20	778.71	758.71	90.00

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Table 2.4.12-208 (Sheet 2 of 2) Monitoring Well Installation Data

Monitoring Point	Reference Elevation (ft msl)	Ground Elevation (ft msl)	Well Depth (ft bre)	Screen Length (ft)	Top of Screen (ft msl)	Bottom of Screen[(a)] (ft msl)	Boring Depth (ft bgs)
MW-1214a	824.16	821.36	47.78	15	794.56	779.56	45.00
MW-1215a	850.63	847.97	18.69	5	839.53	834.53	16.50
MW-1215b	851.05	848.47	42.89	20	831.05	811.05	40.00
MW-1215c	850.34	847.77	73.33	20	800.34	780.34	70.00
MW-1216a	846.39	843.74	20.63	5	833.79	828.79	18.00
MW-1216b	846.92	844.36	48.2	20	821.92	801.92	45.00
MW-1216c	846.65	844.04	68.39	20	801.65	781.65	65.00
MW-1217a	846.98	844.35	17.75	5	837.38	832.38	15.00
MW-1217b	847.38	844.83	48.21	20	822.38	802.38	45.40
MW-1217c	846.89	844.30	72.99	20	796.89	776.89	70.00
MW-1218a	838.06	835.48	18.05	5	828.06	823.06	15.60
MW-1219a	838.72	836.35	55.74	25	811.12	786.12	53.00
RW-1	818.69	816.19	64.23	30	788.19	758.19	63.00
OW-1	819.07	816.57	60.10	25	786.07	761.07	58.00
OW-2	818.88	816.33	54.21	20	787.38	767.38	52.00
OW-3	818.30	815.80	66.5	30	783.80	753.80	65.00

a) Bottom of screen includes 0.45 foot (5.4 inches) for bottom cap and threads.
Bottom of Screen Elevation = Reference Elevation - Well Depth + 0.45 ft

bre - below reference elevation (top of well casing)
bgs - below ground surface
amsl - above mean sea level
ft - feet

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Table 2.4.12-209 (Sheet 1 of 3)
Groundwater and Surface Water Elevation Measurements

	2006		2007										
Monitoring Point	11/29	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1200b	Dry	Dry	Dry	Dry	794.34	794.80	795.56	796.08	796.55	796.87	797.22	797.47	797.66
MW-1200c	Dry	Dry	Dry	Dry	Dry	Dry	Dry	754.00	754.07	754.06	754.06	754.06	754.04
MW-1201a	845.34	849.60	850.58	849.89	854.22	855.66	856.23	857.50	858.64	857.57	856.86	856.01	855.42
MW-1201b	813.31	830.41	830.63	830.77	830.93	831.12	830.70	830.95	830.95	830.32	830.75	830.9	830.35
MW-1201c	778.13	778.14	778.14	778.58	779.11	779.54	780.23	780.75	781.37	781.85	782.38	782.96	783.45
MW-1202b	788.69	788.74	789.16	789.74	790.36	790.84	791.62	792.27	792.97	793.56	795.21	794.84	795.52
MW-1202c	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
MW-1203a	846.36	848.08	849.03	849.63	851.43	854.84	855.01	855.18	857.18	856.26	854.64	853.12	852.95
MW-1203b	Dry	813.23	816.09	819.29	822.47	825.16	828.23	830.10	832.20	833.64	834.43	835.11	835.57
MW-1203c	Dry	Dry	Dry	Dry	788.35	788.96	789.94	790.71	791.65	792.45	793.32	794.19	794.96
MW-1204a	819.96	822.86	823.35	823.58	823.41	824.15	824.17	825.01	825.04	824.96	824.69	824.38	824.17
MW-1204b	789.68	789.74	790.07	790.63	791.16	791.65	792.54	793.25	794.20	794.93	795.65	796.57	797.23
MW-1204c	Dry	752.33	752.44	752.63	752.75	752.84	753.08	753.30	753.68	754.07	754.33	754.54	754.74
MW-1205a	845.03	845.23	845.22	845.15	845.09	845.07	845.52	847.53	850.13	850.09	850.16	849.54	848.40
MW-1205b	Dry	Dry	Dry	798.24	798.58	798.84	799.26	799.57	799.98	800.28	800.6	800.95	801.25
MW-1205c	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
MW-1206a	808.40	808.49	808.56	808.57	808.58	808.58	808.56	808.59	815.07	814.80	814.61	814.46	814.34
MW-1206b	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	783.16	783.32	783.47	783.58
MW-1206c	Dry	747.16	747.15	747.15	747.15	Dry	747.97	748.23	748.53	748.80	749.1	749.41	749.70
MW-1207a	835.00	837.24	841.20	840.08	840.34	840.99	840.33	840.34	840.54	839.89	839.95	839.75	839.61
MW-1207b	809.15	828.68	830.16	829.17	829.35	831.55	828.29	829.45	830.48	828.01	827.66	826.95	826.49
MW-1207c	Dry	779.27	780.53	781.91	783.23	784.34	786.08	787.44	788.89	790.05	791.34	792.62	793.74
MW-1208a	781.82	780.85	781.89	781.93	781.92	781.97	781.94	783.48	785.35	785.56	784.95	784.34	783.88

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Table 2.4.12-209 (Sheet 2 of 3) Groundwater and Surface Water Elevation Measurements

	2006						2007						
Monitoring Point	11/29	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1209a	Dry	Dry	769.39	770.47	771.62	772.51	774.12	783.28	785.45	785.58	784.93	784.3	783.79
MW-1209b	750.61	773.18	774.68	775.16	775.36	775.37	775.19	775.14	775.09	774.97	775.13	775.17	775.11
MW-1209c	Dry	709.85	711.91	714.05	716.16	717.89	720.64	722.70	725.05	726.92	729.24	731.96	734.24
MW-1210b	Dry	Dry	Dry	783.38	784.05	784.50	785.08	785.44	785.74	785.95	786.09	786.19	786.25
MW-1210c	Dry	748.31	748.31	748.33	748.33	748.33	748.34	748.34	748.36	748.38	748.38	748.37	748.37
MW-1211a	775.33	775.09	775.36	775.25	775.28	775.27	775.17	775.07	775.06	775.03	775.12	775.21	775.16
MW-1211b	775.31	774.06	775.35	775.23	775.25	775.24	775.14	775.05	775.03	775.02	775.1	775.19	775.13
MW-1212a	785.79	787.11	787.34	787.55	787.48	787.75	787.29	787.89	788.49	787.33	787.27	787.21	786.86
MW-1212b	785.22	785.04	785.27	784.85	784.54	784.94	785.09	784.50	784.55	784.08	784.75	785.33	783.73
MW-1212c	735.07	735.65	736.08	736.55	736.99	737.34	737.88	738.29	738.78	739.18	739.64	740.16	740.59
MW-1213b	Dry	Dry	Dry	781.40	782.27	783.02	784.21	785.22	786.42	787.44	788.52	789.61	790.58
MW-1213c	756.60	756.36	756.37	756.41	756.41	756.45	756.48	756.51	756.54	756.56	756.59	756.63	756.66
MW-1214a	777.79	777.95	779.90	780.72	779.32	782.06	783.37	784.14	783.81	782.51	780.37	778.47	777.80
MW-1215a	834.26	833.79	835.25	835.93	836.21	837.27	837.26	839.70	841.18	841.41	841.89	841.81	841.42
MW-1215b	808.52	831.35	831.27	831.64	831.60	832.10	831.80	832.91	833.74	833.55	833.54	833.84	833.12
MW-1215c	Dry	Dry	Dry	Dry	Dry	777.46	777.99	778.40	778.89	779.28	779.69	780.14	780.52
MW-1216a	827.19	827.79	828.10	828.57	828.35	828.59	828.99	829.62	830.69	830.82	830.47	830.18	829.87
MW-1216b	Dry	800.52	802.43	804.16	805.51	806.37	807.42	808.10	808.83	809.62	810.71	812.11	813.73
MW-1216c	Dry	Dry	Dry	Dry	Dry	Dry	778.73	778.96	779.20	779.37	779.6	779.82	780.00
MW-1217a	830.28	829.52	829.45	829.45	829.45	829.45	829.45	829.44	830.31	829.70	829.57	829.54	829.54
MW-1217b	800.55	810.94	820.76	824.72	825.06	823.82	820.08	820.38	821.13	822.28	823.83	825.64	827.00
MW-1217c	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	774.04	774.36	774.58	774.75
MW-1218a	823.41	824.06	827.35	826.24	825.62	830.78	830.97	831.32	831.23	828.84	826.36	823.96	823.53

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Table 2.4.12-209 (Sheet 3 of 3)
Groundwater and Surface Water Elevation Measurements

	2006							2007					
Monitoring Point	11/29	12/27	1/23	2/20	3/19	4/10	5/16	6/13	7/16	8/13	9/13	10/16	11/15
MW-1219a	788.91	788.99	789.22	789.47	789.52	790.96	791.58	793.14	794.04	793.50	792.25	790.66	789.73
RW-1	--	--	--	--	775.18	775.17	775.07	774.97	774.97	774.94	775.03	775.10	775.05
OW-1	--	--	--	--	775.23	775.21	775.12	775.01	775.01	774.97	775.07	775.16	775.10
OW-2	--	--	--	--	775.18	775.16	775.07	774.98	774.97	774.94	775.03	775.13	775.06
OW-3	--	--	--	--	775.60	775.59	775.50	775.40	775.39	775.37	775.46	775.56	775.48
Brazos River Glen Rose Station (USGS 08091000[a])	569.37	569.34	569.68	569.37	569.40	572.33	574.01	573.03	574.41	571.54	572.51	571.35	570.58
SCR (USGS 08091730[a])	775.40	775.23	775.42	775.19	--	775.36	775.39	775.31	775.33	775.40	775.46	775.48	775.38
Lake Granbury (USGS 08090900[a])	691.14	691.53	692.15	692.32	692.37	692.37	692.54	692.48	692.30	692.38	692.29	692.44	691.90

a) Provisional Data

NOTES:
Elevations provided are in ft msl
-- No Data Available

Source: Water Data for Texas. U.S. Geological Survey, National Water Information System. USGS Surface Water Data for the Nation. http://waterdata.usgs.gov/tx/nwis/. Accessed June 2007.

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Table 2.4.12-210 (Sheet 1 of 4)
Characteristics of Soil Areas at the CPNPP Site

Soil Name	Description	Slope Range (percent)
Aledo Series	The Aledo series consists of shallow, calcareous, gently sloping to rolling soils on uplands. In a representative profile, the surface layer is dark grayish-brown gravelly clay loam, about 4 inches thick. Below the surface and to a depth of 16 inches is grayish-brown very gravelly clay loam that rests abruptly on coarsely fractured limestone.	---
Bolar Series	The Bolar series consists of moderately deep, well drained soils on uplands. The soil formed in interbedded limestone, marl and marly clay. The surface layer is dark brown clay loam 16 inches thick. From 16 to 32 inches is brown clay loam. It is yellowish brown very stony clay loam from 32 to 36 inches. Below is fractured limestone bedrock. Interbedded with marly clay. The soil is calcareous throughout.	1 to 5
Bosque Series	The Bosque series consists of very deep, well drained moderately permeable nearly level soils of the bottomlands. The soil formed in calcareous loamy sediments. In a representative profile, the surface layer is dark grayish brown loam 20 inches thick. The next layer is clay loam 30 inches thick, that is dark brown in the upper part and brown in the lower part. The substratum, below 50 inches, is dark grayish brown clay.	---
Bunyan Series	The Bunyan series consists of deep, well drained, nearly level soils of the bottomlands. The soil formed in stratified loamy alluvium. In a representative profile, the surface layer is light brownish gray fine sandy loam about 10 inches thick. Below the surface layer and to a depth of 16 inches is very dark grayish brown clay loam. The next layer is grayish brown to pale brown sandy clay loam about 30 inches thick. The next layer is gray clay loam that extends to 62 inches depth.	-
Chaney Series	The Chaney series consists of very deep, moderately well drained nearly level to moderately sloping soils of uplands. The soil formed in clayey deposits. In a representative profile, the surface layer is loamy sand 14 inches thick. Dark grayish brown in the upper part and light gray in the lower part. The subsoil is dark red and red mottled sandy clay 20 inches thick. The next layer is sandy clay loam 18 inches thick; brownish yellow in the upper part and light brownish gray in the lower part. Below 52 inches is olive gray shale that has clay texture.	1 to 5

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Table 2.4.12-210 (Sheet 2 of 4)
Characteristics of Soil Areas at the CPNPP Site

Soil Name	Description	Slope Range (percent)
Duffau Series	The Duffau series consists of very deep, well drained, nearly level to sloping soils of uplands. The soil formed in sandy and loamy deposits. In a representative profile, the surface layer is fine sandy loam 10 inches thick and is dark grayish brown in the upper part and pale brown in the lower part. The subsoil is yellowish red sandy clay loam 60 inches thick. The substrata, below 70 inches depth, is reddish yellow sandy clay loam.	1 to 5
Frio Series	The Frio series consists of very deep, well drained, nearly level soils of the bottomlands. The soil formed in calcareous alluvium. In a representative profile, the surface layer is very dark grayish brown silty clay loam about 8 inches thick. Below the surface layer and to a depth of 40 inches is very dark grayish brown silty clay loam and clay loam. The next layer extends to 80 inches and is dark grayish brown silty clay, with soft masses of calcium carbonate.	---
Hassee Series	The Hassee series consists of very deep, moderately well drained, nearly level to very gently sloping soils of uplands. The soil formed in clayey sediments. In a representative profile, the surface layer is fine sandy loam 11 inches thick and is brown in the lower part and dark grayish brown in the lower part. The subsoil to 36 inches is clay that is grayish brown. Below 36 inches is grayish brown, and light brownish gray clay loam.	1 to 3
Krum Series	The Krum series consists of very deep, well drained, nearly level to moderately sloping soils of uplands. The soil formed in calcareous clayey sediments. In a representative profile the surface layer is dark grayish brown to very dark grayish brown silty clay about 26 inches thick. The next lower layer is a brown silty clay about 18 inches thick. The underlying sediments are reddish yellow silty clay.	1 to 3
Nimrod Series	The Nimrod series consists of very deep, moderately well drained, nearly level to sloping sandy soils of uplands. The soil formed in sandy and loamy deposits. In a representative profile, the surface layer is grayish brown fine sand 4 inches thick. From 4 to 27 inches is very pale brown fine sand. The next layer is mottled light gray. Reddish yellow and yellowish brown sandy clay loam 13 inches thick. The next layer is light gray sandy clay loam 28 inches thick. Below 68 inches is red and light gray sandy loam.	1 to 5

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Table 2.4.12-210 (Sheet 3 of 4)
Characteristics of Soil Areas at the CPNPP Site

Soil Name	Description	Slope Range (percent)
Perdenales Series	The Pedernales series consists of very deep, well drained, nearly level to moderately sloping soils of uplands. This soil formed in loamy calcareous materials. In a representative profile, the surface layer is a reddish brown fine sandy loam about 11 inches thick. The subsoil is red sandy clay from 11 to 37 inches and yellowish red sandy clay loam from 37-43 inches. Below 43 inches is light reddish brown sandy clay loam.	1 to 5
Purves Series	The Purves series consists of shallow, well drained, moderately slowly permeable, gently sloping to moderately sloping upland soils. The soil formed in interbedded limestone and calcareous marls. In a representative profile, the surface layer is very dark grayish brown clay about 8 inches thick. The next layer is brown clay about 4 inches thick. The next lower layer is brown very gravelly clay about 2 inches thick. The substrata below 14 inches is limestone bedrock.	1 to 5
Sunev Series	The Sunev series consists of very deep, well drained, moderately permeable nearly level to sloping limy soils of uplands. The soil formed in loamy alluvial sediments. In a representative profile, the surface layer is dark grayish brown loam about 12 inches thick. The next layer is brown loam about 9 inches thick. The lower layer is very pale brown loam extending to 72 inches.	3 to 5
Tarrant Series	The Tarrant series consists of very shallow and shallow, well drained, moderately slowly permeable nearly level to steep soils on uplands. The soil formed in residuum over limestone bedrock. In a representative profile, the soil is a very dark grayish brown and dark brown calcareous stony clay about 13 inches thick. The substratum from 13 to 30 inches is fractured platy limestone bedrock.	---
Thurber Series	The Thurber series consists of very deep, moderately well drained, nearly level to gently sloping soils of uplands. The soil formed in clayey sediments. In a representative profile, the surface layer is dark grayish brown clay loam 8 inches thick. The subsoil is brown clay from 8 to 38 inches and from 38 to 93 inches is brown clay in the upper part and yellowish brown clay loam in the lower part.	1 to 3

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Table 2.4.12-210 (Sheet 4 of 4)
Characteristics of Soil Areas at the CPNPP Site

Soil Name	Description	Slope Range (percent)
Venus Series	The Venus series consists of very deep, well drained, nearly level to strongly sloping calcareous soils of uplands. The soil formed in calcareous loamy Sediments. In a representative profile, the surface layer is dark grayish brown loam about 14 inches thick. Below the surface layer and to a depth of 50 inches is loam that is grayish brown in the upper part and very pale brown in the lower part. Below 50 inches is very pale brown fine sandy loam.	1 to 3
Windthorst Series	The Windthorst series consists of very deep, gently sloping to strongly sloping soils on uplands. The soil formed in stratified clayey and loamy materials. In a representative profile the surface layer is fine sandy loam 10 in. Thick and is grayish brown in the upper part and light yellowish brown below. The subsoil is sandy clay 28 in. Thick and is red in the upper part and yellowish red below. The next layer is mottled sandy clay loam 12 inches thick. Below 50 inches is light gray sandy clay loam that grades to weakly cemented packsand.	1 to 8

Note:

--- No slope range specified

Source: Reference 2.4-260

Table 2.4.12-211
Groundwater Velocities and Travel Times

	Path 1	Path 2
Release Elevation (E_h)(ft msl)	820.00	820.00
Discharge Elevation (E_l)(ft msl)	770.00	770.00
Distance to SCR (L)(ft)	600	350
Hydraulic Gradient (Eh-El)/L	0.0833	0.1429
Velocity (V) (ft/day)	4.13	1.01
Travel Time (T) (days)	145	346

Path 1 is from Unit 3 east to SCR; Path 2 is from Unit 4 north to SCR

Equation for Velocity: $v = (Kh\ (E_h-E_l)/L)/\eta$

Equation for Travel Time: T = L/V

Path 1 fill Kh is 3.50×10^{-3} cm/sec (9.92 ft/day) from RW-1 recovery test.

Path 2 fill Kh is 5.00×10^{-4} cm/sec (1.42 ft/day) from MW-1219a slug test.

Assumptions:

1. Engineered fill is conservatively assumed as having negligible transport time.
2. Engineered fill is assumed to be fully saturated to level of the perimeter trench drains.
3. Release elevation is assumed to be the elevation of trench drain transposed to the edge of the existing fill at the pathway release point (E_h at 820 ft msl).
4. Discharge elevation is assumed to be the elevation of the SCR minimum operating pool (E_l at 770 ft msl).
5. Pathway distance is assumed to be the shortest distance from the pathway release point to the shoreline of SCR.
6. Existing fill (large rubble, sand, and gravel) is assumed to have 20% effective porosity (η=0.20).

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Table 2.4.12-212
Domestic Wells Within 2-Mile Radius of CPNPP

Well Number	County	Owner	Primary Use	Well Depth (ft)	Aquifer	Latitude	Longitude	Well Type
3242501	Somervell	Bert Willie	Unused	300	Twin Mountains Formation	321738	974930	Withdrawal of Water
3242502	Somervell	J.C. Ice	Domestic	352	Twin Mountains Formation	321807	974853	Withdrawal of Water
3242503	Somervell	Texas Utilities	Industrial	517	Twin Mountains Formation	321802	974826	Withdrawal of Water
3242504	Somervell	Texas Utilities	Public Supply	400	Twin Mountains Formation	321802	974822	Withdrawal of Water
3242601	Somervell	Texas Utilities	Industrial	466	Twin Mountains Formation	321745	974723	Withdrawal of Water
3242602	Somervell	Texas Utilities	Industrial	490	Twin Mountains Formation	321751	974649	Withdrawal of Water
3242603	Somervell	Texas Utilities	Industrial	471	Twin Mountains Formation	321858	974656	Withdrawal of Water
3242604	Hood	Texas Utilities	Not Used	470	Twin Mountains Formation	321910	974655	Observation
3242801	Somervell	L. P. Jones	Domestic	352	Twin Mountains Formation	321642	974845	Withdrawal of Water
3242802	Somervell	Oak Grove Sub-div.	Public Supply	360	Twin Mountains Formation	321725	974835	Withdrawal of Water
3242803	Somervell	Oak Grove Sub-div.	Public Supply	360	Twin Mountains Formation	321725	974835	Withdrawal of Water
3242804	Somervell	Scruggs Mobile Home Pk	Public Supply	420	Twin Mountains Formation	321656	974832	Withdrawal of Water
3242903	Somervell	Texas Utilities	Not Used	479	Twin Mountains Formation	321651	974623	Withdrawal of Water

Source: Reference 2.4-258

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Table 2.4.13-201
Distribution Coefficients (K_d)

Isotope	MW-1201	MW-1208	MW-1219
Co-60 (cm3/g)	3071 ± 309	8615 ± 991	5501 ± 551
Cs-137 (cm3/g)	5586 ± 786	7390 ± 1022	7978 ± 1113
Fe-55 (cm3/g)	1901 ± 269	4113 ± 582	3973 ± 562
I-129 (cm3/g)	33.6 ± 5.0	0.08 ± 0.01	6.35 ± 0.94
Ni-63 (cm3/g)	736 ± 85	875 ± 100	881 ± 98
Pu-242 (cm3/g)	1340 ± 189	1773 ± 251	2526 ± 357
Sr-90 (cm3/g)	578 ± 81	101 ± 14	134 ± 19
Tc-99 (cm3/g)	0.0	0.08 ± 0.01	0.11 ± 0.01
U-235 (cm3/g)	21.6 ± 3.1	26.2 ± 3.7	46.2 ± 6.5

Table 2.4.13-202 (Sheet 1 of 2)
Source Term Activity after 0.4 Years (145 Days) Decay

Radioisotope	ECL Limit		Activity Concentration		145 Days Decay Activity[(a)]
	(μCi/ml)	(μCi/gal)	(μCi/ml)	(μCi/gal)	(μCi)
H-3	1.00E-03	3.79+E00	7.30E-01	2.76E+03	1.46E+08
Cr-51	5.00E-04	1.89+E00	8.00E-07	3.03E-03	1.60E+02
Mn-54	3.00E-05	1.14E-01	3.00E-05	1.14E-01	6.02E+03
Fe-55	1.00E-04	3.79E-01	3.00E-04	1.14E+00	6.02E+04
Fe-59	1.00E-05	3.79E-02	2.70E-06	1.02E-02	5.39E+02
Co-58	2.00E-05	7.57E-02	1.60E-04	6.06E-01	3.20E+04
Co-60	3.00E-06	1.14E-02	4.50E-04	1.70E+00	8.98E+04
Sr-89	8.00E-06	3.03E-02	1.44E-06	5.45E-03	2.88E+02
Sr-90	5.00E-07	1.89E-03	2.64E-06	9.99E-03	5.27E+02
Y-91	8.00E-06	3.03E-02	4.20E-07	1.59E-03	8.40E+01
Zr-95	2.00E-05	7.57E-02	4.68E-07	1.77E-03	9.35E+01
Nb-95	3.00E-05	1.14E-01	1.80E-07	6.81E-04	3.60E+01
Ru-103	3.00E-05	1.14E-01	7.92E-08	3.00E-04	1.58E+01
Ru-106	3.00E-06	1.14E-02	1.14E-06	4.32E-03	2.28E+02
Te-129m	7.00E-06	2.65E-02	1.44E-06	5.45E-03	2.88E+02
I-131	1.00E-06	3.79E-03	4.68E-09	1.77E-05	9.35E-01
Cs-134	9.00E-07	3.41E-03	9.24E-02	3.50E+02	1.85E+07
Cs-136	6.00E-06	2.27E-02	9.00E-07	3.41E-03	1.80E+02
Cs-137	1.00E-06	3.79E-03	9.96E-02	3.77E+02	1.99E+07
Ce-141	3.00E-05	1.14E-01	5.76E-08	2.18E-04	1.15E+01

Table 2.4.13-202 (Sheet 2 of 2)
Source Term Activity after 0.4 Years (145 Days) Decay

Radioisotope	ECL Limit		Activity Concentration		145 Days Decay Activity[(a)]
	(µCi/ml)	(µCi/gal)	(µCi/ml)	(µCi/gal)	(µCi)
Ce-144	3.00E-06	1.14E-02	3.00E-06	1.14E-02	6.02E+02

Note:

(a) Based upon 52,800 gallons in BAT release.

Table 2.4.13-203
Cases Considered in Tank Failure Analysis for Units 3 and 4

Analysis Category	Unit 3	Unit 4
Retardation and retention in engineered fill	None	None
Transported as a slug via groundwater in engineered fill	No groundwater diffusion considered	No groundwater diffusion considered
Transported with the groundwater velocity	145 days	346 days
Radionuclide decay time (days)	145 days	346 days
Dilution volume of total available groundwater	25% (2.5E06 gal)	N/A[(a)]
Dilution volume of SCR for CW half-flow condition	1E06 gpm (Subsection 2.4.13.5.5)	N/A[(a)]
Dilution volume of SCR for CW full-flow condition	2E06 gpm (Subsection 2.4.13.5.5)	N/A[(a)]
Dilution volume of SCR for no-flow condition	1.36E10 gal (Subsection 2.4.13.5.6)	1.36E10 gal

Note:

(a) N/A - Not Applicable - Unit 3 is bounding condition.

Table 2.4.13-204 (Sheet 1 of 2)
Dilution Effect of Various Quantities of Existing Fill Groundwater

Radioisotope	Activity Concentration	Existing Fill Groundwater Dilution Percent Credited		
		100%	50%	25%
	(μCi)	(μCi/gal)	(μCi/gal)	(μCi/gal)
H-3	1.46E+08	1.46E+01	2.92E+01	5.85E+01
Cr-51	1.60E+02	1.60E-05	3.21E-05	6.41E-05
Mn-54	6.02E+03	6.03E-04	1.21E-03	2.41E-03
Fe-55	6.02E+04	6.03E-03	1.21E-02	2.41E-02
Fe-59	5.39E+02	5.40E-05	1.08E-04	2.16E-04
Co-58	3.20E+04	3.21E-03	6.41E-03	1.28E-02
Co-60	8.98E+04	8.99E-03	1.80E-02	3.60E-02
Sr-89	2.88E+02	2.88E-05	5.77E-05	1.15E-04
Sr-90	5.27E+02	5.29E-05	1.06E-04	2.11E-04
Y-91	8.40E+01	8.41E-06	1.68E-05	3.36E-05
Zr-95	9.35E+01	9.36E-06	1.87E-05	3.75E-05
Nb-95	3.60E+01	3.60E-06	7.21E-06	1.44E-05
Ru-103	1.58E+01	1.59E-06	3.17E-06	6.35E-06
Ru-106	2.28E+02	2.29E-05	4.57E-05	9.14E-05
Te-129m	2.88E+02	2.88E-05	5.77E-05	1.15E-04
I-131	9.35E-01	9.36E-08	1.87E-07	3.75E-07
Cs-134	1.85E+07	1.85E+00	3.70E+00	7.40E+00
Cs-136	1.80E+02	1.80E-05	3.61E-05	7.22E-05
Cs-137	1.99E+07	1.99E+00	3.99E+00	7.98E+00

**Table 2.4.13-204 (Sheet 2 of 2)
Dilution Effect of Various Quantities of Existing Fill Groundwater**

Radioisotope	Activity Concentration	Existing Fill Groundwater Dilution Percent Credited		
		100%	50%	25%
	(µCi)	(µCi/gal)	(µCi/gal)	(µCi/gal)
Ce-141	1.15E+01	1.15E-06	2.31E-06	4.61E-06
Ce-144	6.02E+02	6.03E-05	1.21E-04	2.41E-04

Note:

Activity Concentration after dilution = Activity (µCi) / 9.98E06 gal x percent credited.

Table 2.4.13-205 (Sheet 1 of 2)
Dilution Effect of 25 Percent of Existing Fill Groundwater

Radioisotope	Activity (μCi)	Activity Concentration (μCi/gal)[1]	25% Groundwater Dilution (μCi/gal)[2]	Flow into SCR (μCi/min)[3]
H-3	1.46E+08	2.76E+03	5.85E+01	8.75E+03
Cr-51	1.60E+02	3.03E-03	6.41E-05	9.60E-03
Mn-54	6.02E+03	1.14E-01	2.41E-03	3.61E-01
Fe-55	6.02E+04	1.14E+00	2.41E-02	3.61E+00
Fe-59	5.39E+02	1.02E-02	2.16E-04	3.23E-02
Co-58	3.20E+04	6.06E-01	1.28E-02	1.92E+00
Co-60	8.98E+04	1.70E+00	3.60E-02	5.39E+00
Sr-89	2.88E+02	5.45E-03	1.15E-04	1.73E-02
Sr-90	5.27E+02	9.99E-03	2.11E-04	3.16E-02
Y-91	8.40E+01	1.59E-03	3.36E-05	5.04E-03
Zr-95	9.35E+01	1.77E-03	3.75E-05	5.61E-03
Nb-95	3.60E+01	6.81E-04	1.44E-05	2.16E-03
Ru-103	1.58E+01	3.00E-04	6.35E-06	9.50E-04
Ru-106	2.28E+02	4.32E-03	9.14E-05	1.37E-02
Te-129m	2.88E+02	5.45E-03	1.15E-04	1.73E-02
I-131	9.35E-01	1.77E-05	3.75E-07	5.61E-05
Cs-134	1.85E+07	3.50E+02	7.40E+00	1.11E+03
Cs-136	1.80E+02	3.41E-03	7.22E-05	1.08E-02
Cs-137	1.99E+07	3.77E+02	7.98E+00	1.19E+03
Ce-141	1.15E+01	2.18E-04	4.61E-06	6.91E-04

Table 2.4.13-205 (Sheet 2 of 2)
Dilution Effect of 25 Percent of Existing Fill Groundwater

Radioisotope	Activity (μCi)	Activity Concentration (μCi/gal)[1]	25% Groundwater Dilution (μCi/gal)[2]	Flow into SCR (μCi/min)[3]
Ce-144	6.02E+02	1.14E-02	2.41E-04	3.61E-02

Notes:

1. After 145 days of decay and reduced by 52,800 gal.
2. Based upon dilution with 2.5E06 gal of existing fill groundwater.
3. Based upon 25 percent existing fill dilution, and = μCi/gal x 149.7 gpm groundwater flow.

Table 2.4.13-206 (Sheet 1 of 2)
Mixing and Dilution Effect of Circulating Water

Radioisotope	Flow into SCR (μCi/min)[1]	Dilution Effect of Full-flow CW (μCi/gal)[2]	Dilution Effect of Half-flow CW (μCi/gal)[3]
H-3	8.75E+03	4.38E-03	8.75E-03
Cr-51	9.60E-03	4.80E-09	9.60E-09
Mn-54	3.61E-01	1.81E-07	3.61E-07
Fe-55	3.61E+00	1.81E-06	3.61E-06
Fe-59	3.23E-02	1.62E-08	3.23E-08
Co-58	1.92E+00	9.60E-07	1.92EE-06
Co-60	5.39E+00	2.69E-06	5.39E-06
Sr-89	1.73E-02	8.63E-09	1.73E-08
Sr-90	3.16E-02	1.58E-08	3.16E-08
Y-91	5.04E-03	2.52E-09	5.04E-09
Zr-95	5.61E-03	2.80E-09	5.61E-09
Nb-95	2.16E-03	1.08E-09	2.16E-09
Ru-103	9.50E-04	4.75E-10	9.50E-10
Ru-106	1.37E-02	6.84E-09	1.37E-08
Te-129m	1.73E-02	8.63E-09	1.73E-08
I-131	5.61E-05	2.80E-11	5.61E-11
Cs-134	1.11E+03	5.54E-04	1.11E-03
Cs-136	1.08E-02	5.40E-09	1.08E-08
Cs-137	1.19E+03	5.97E-04	1.19E-03
Ce-141	6.91E-04	3.45E-10	6.91E-10

**Table 2.4.13-206 (Sheet 2 of 2)
Mixing and Dilution Effect of Circulating Water**

Radioisotope	Flow into SCR (µCi/min)[1]	Dilution Effect of Full-flow CW (µCi/gal)[2]	Dilution Effect of Half-flow CW (µCi/gal)[3]
Ce-144	3.61E-02	1.81E-08	3.61E-08

Notes:

1. Based upon 25 percent existing fill dilution, and = µCi/gal x 149.7 gpm groundwater flow.
2. Based upon 149.7 gpm infiltration flow, and = infiltration flow into SCR (µCi/min) / 2E06 gpm.
3. Based upon 149.7 gpm infiltration flow, and = infiltration flow into SCR (µCi/min) / 1E06 gpm.

Table 2.4.13-207 (Sheet 1 of 2)
Ratio of Source Term Concentration to ECL for Full-flow CW

Radioisotope	ECL Limit (μCi/gal)	Dilution Effect of CW (μCi/gal)[(a)]	Ratio of Activity Concentration to ECL
H-3	3.79E+00	4.38E-03	1.15E-03
Cr-51	1.89E+00	4.80E-09	2.54E-09
Mn-54	1.14E-01	1.81E-07	1.58E-06
Fe-55	3.79E-01	1.81E-06	4.76E-06
Fe-59	3.79E-02	1.62E-08	4.26E-07
Co-58	7.57E-02	9.60E-07	1.27E-05
Co-60	1.14E-02	2.69E-06	2.36E-04
Sr-89	3.03E-02	8.63E-09	2.85E-07
Sr-90	1.89E-03	1.58E-08	8.37E-06
Y-91	3.03E-02	2.52E-09	8.31E-08
Zr-95	7.57E-02	2.80E-09	3.70E-08
Nb-95	1.14E-01	1.08E-09	9.46E-09
Ru-103	1.14E-01	4.75E-10	4.17E-09
Ru-106	1.14E-02	6.84E-09	6.00E-07
Te-129m	2.65E-02	8.63E-09	3.26E-07
I-131	3.79E-03	2.80E-11	7.40E-09
Cs-134	3.41E-03	5.54E-04	1.62E-01
Cs-136	2.27E-02	5.40E-04	2.38E-07
Cs-137	3.79E-03	5.97E-04	1.58E-01
Ce-141	1.14E-01	3.45E-10	3.03E-09

Table 2.4.13-207 (Sheet 2 of 2)
Ratio of Source Term Concentration to ECL for Full-flow CW

Radioisotope	ECL Limit (μCi/gal)	Dilution Effect of CW (μCi/gal)[(a)]	Ratio of Activity Concentration to ECL
Ce-144	1.14E-02	1.81E-08	1.58E-06
	Σ [Source Term Activity / ECL] =		3.21E-01

Note:

(a) At the infiltration rate of 149.7 gpm, and = infiltration flow into SCR (μCi/min) / 2E06 gpm.

Table 2.4.13-208 (Sheet 1 of 2)
Ratio of Source Term Concentration to ECL for Half-flow CW

Radioisotope	ECL Limit (µCi/gal)	Dilution Effect of CW (µCi/gal)(a)	Ratio of Activity Concentration to ECL
H-3	3.79E+00	8.75E-03	2.31E-03
Cr-51	1.89E+00	9.60E-09	5.08E-09
Mn-54	1.14E-01	3.61E-07	3.17E-06
Fe-55	3.79E-01	3.61E-06	9.53E-06
Fe-59	3.79E-02	3.23E-08	8.53E-07
Co-58	7.57E-02	1.92E-06	2.54E-05
Co-60	1.14E-02	5.39E-06	4.72E-04
Sr-89	3.03E-02	1.73E-08	5.70E-07
Sr-90	1.89E-03	3.16E-08	1.67E-05
Y-91	3.03E-02	5.04E-09	1.66E-07
Zr-95	7.57E-02	5.61E-09	7.41E-08
Nb-95	1.14E-01	2.16E-09	1.89E-08
Ru-103	1.14E-01	9.50E-10	8.34E-09
Ru-106	1.14E-02	1.37E-08	1.20E-06
Te-129m	2.65E-02	1.73E-08	6.52E-07
I-131	3.79E-03	5.61E-11	1.48E-08
Cs-134	3.41E-03	1.11E-03	3.25E-01
Cs-136	2.27E-02	1.08E-08	4.76E-07
Cs-137	3.79E-03	1.19E-03	3.15E-01
Ce-141	1.14E-01	6.91E-10	6.06E-09

Table 2.4.13-208 (Sheet 2 of 2)
Ratio of Source Term Concentration to ECL for Half-flow CW

Radioisotope	ECL Limit (μCi/gal)	Dilution Effect of CW (μCi/gal)[(a)]	Ratio of Activity Concentration to ECL
Ce-144	1.14E-02	3.61E-08	3.17E-06
	Σ [Source Term Activity / ECL] =		6.43E-01

Note:

(a) At the infiltration rate of 149.7 gpm, and = infiltration flow into SCR (μCi/min) / 1E06 gpm.

**Table 2.4.13-209 (Sheet 1 of 2)
Ratio of Source Term Concentration to ECL for No-flow Conditions**

Radioisotope	Activity Concentration (µCi)[1]	SCR Dilution Effect (µCi/gal)[2]	Ratio of Activity Concentration to ECL
H-3	1.46E+08	1.07E-02	2.83E-03
Cr-51	1.60E+02	1.18E-08	6.22E-09
Mn-54	6.02E+03	4.42E-07	3.88E-06
Fe-55	6.02E+04	4.42E-06	1.17E-05
Fe-59	5.39E+02	3.96E-08	1.04E-06
Co-58	3.20E+04	2.35E-06	3.11E-05
Co-60	8.98E+04	6.60E-06	5.79E-04
Sr-89	2.88E+02	2.11E-08	6.98E-07
Sr-90	5.27E+02	3.88E-08	2.05E-05
Y-91	8.40E+01	6.17E-09	2.04E-07
Zr-95	9.35E+01	6.87E-09	9.07E-08
Nb-95	3.60E+01	2.64E-09	2.32E-08
Ru-103	1.58E+01	1.16E-09	1.02E-08
Ru-106	2.28E+02	1.68E-08	1.47E-06
Te-129m	2.88E+02	2.11E-08	7.98E-07
I-131	9.35E-01	6.87E-11	1.81E-08
Cs-134	1.85E+07	1.36E-03	3.98E-01
Cs-136	1.80E+02	1.32E-08	5.83E-07
Cs-137	1.99E+07	1.46E-03	3.86E-01
Ce-141	1.15E+01	8.46E-10	7.42E-09

Table 2.4.13-209 (Sheet 2 of 2)
Ratio of Source Term Concentration to ECL for No-flow Conditions

Radioisotope	Activity Concentration (µCi)[1]	SCR Dilution Effect (µCi/gal)[2]	Ratio of Activity Concentration to ECL
Ce-144	6.02E+02	4.42E-08	3.88E-06
	Σ [Source Term Activity / ECL] =		7.87E-01

Notes:

1. At 145 days of decay.

2. Volume determined from east of existing fill south to Roto-cone and = Activity (µCi) / 1.36E10 gal.

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

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

CP COL 2.5(1) Replace the content of DCD Section 2.5 with the following.

This section provides information on the geology, seismology, and geotechnical characteristics of the Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 site focusing on those data developed since the publication of the Final Safety Analysis Report (FSAR, Reference 2.5-201) for licensing CPNPP Units 1 and 2 at the CPNPP site. The section follows the standard format and content specifications of Regulatory Guide (RG) 1.206 (U.S. Nuclear Regulatory Commission [NRC], June 20, 2007). The section complies with RG 1.208, which provides guidance for the level of investigation recommended at different distances from a proposed site for a nuclear facility.

Subsection 2.5.1 describes basic geological and seismological data compiled through literature reviews as well as regional and site-specific studies. Much of the site-specific data were gathered as part of the geotechnical exploration program described in detail within Subsection 2.5.4, which also provided information for Subsection 2.5.2 as well as Subsection 2.5.3.

Subsection 2.5.2 describes the vibratory ground motion at the site, including an updated seismicity catalog, description of seismic sources, and development of the Ground Motion Response Spectra (GMRS) and site-specific Safe Shutdown Earthquake (SSE). The site characteristics to develop the GMRS and SSE were compiled from data acquired as part of the investigations described in Subsections 2.5.1 and 2.5.4.

Subsection 2.5.3 describes the potential for surface faulting in the site area and presents the results of both regional and site-specific studies discussed in Subsections 2.5.1 and 2.5.4.

Subsections 2.5.4 and 2.5.5 describe the stability of surface materials and slopes at the site. Included in Subsection 2.5.4 is a detailed discussion of site-specific investigations that provide supporting data for Subsections 2.5.1, 2.5.2, and 2.5.3.

In summary, the geologic, seismologic, and geotechnical conditions for the CPNPP Units 3 and 4 site are well characterized and are consistent with the conditions noted for the existing Units 1 and 2. Detailed discussions are provided in the following subsections.

2.5.1 Basic Geologic and Seismic Information

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.1 with the following.

This subsection presents information on the geological, seismological, and geotechnical engineering properties of CPNPP Units 3 and 4.

CP COL 2.5(1) RG 1.208 provides guidance for the recommended level of investigation at different distances from a proposed site for a nuclear facility.

- The site region is that area within 200 miles (mi) of the site.
- The site vicinity is that area within 25 mi of the site.
- The site area is that area within 5 mi of the site.
- The site location is that area within 0.6 mi of the site.

These terms–site region, site vicinity, site area, and site–are used in Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4 and 2.5.5 to describe these specific areas of investigation and are not applicable to other subsections of the FSAR.

The geological and seismological information presented in this subsection was developed from a review of previous reports prepared for CPNPP Units 1 and 2, published geologic literature, interpretation of aerial photography, subsurface investigations, geological mapping, and aerial reconnaissance conducted to support this CPNPP Units 3 and 4 application. Previous site-specific reports reviewed include the CPNPP Units 1 and 2 FSAR (Reference 2.5-201). A review of published geologic literature was used to supplement and update the existing geological and seismological information.

This subsection of the CPNPP Units 3 and 4 FSAR is intended to demonstrate compliance with the requirements of 10 CFR 100 “Reactor Site Criteria,” Section 100.23(c). The results of detailed, site-specific investigations to define the geologic and geotechnical conditions are presented in the following subsections. Results of these investigations are used to demonstrate the subsurface conditions for site response as well as static and dynamic geotechnical performance. The presented analysis and conclusions were developed by the following:

- William Lettis & Associates Inc. - Overall technical responsibility for Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4 and 2.5.5.
- Fugro West Inc. - Geotechnical analysis and laboratory testing.
- Risk Engineering Inc. - Site Response and Probabilistic Seismic Hazard Analysis (PSHA).

2.5.1.1 Regional Geology

This subsection discusses the physiography, geologic history, stratigraphy, and tectonic setting within a 200-mi radius of CPNPP Units 3 and 4. The Physiographic Map of Texas (Figure 2.5.1-201) and Regional Geology Map (Figure 2.5.1-202a) show the physiography, geology, stratigraphy, and tectonic setting of the region surrounding the CPNPP site. The 25-mi geologic map is presented in Subsection 2.5.2. Summaries of the aspects of the regional geology are presented to provide the framework for evaluation of the geologic and seismologic hazards presented in the succeeding subsections.

The nomenclature adopted for describing the physiography, tectonic provinces, stratigraphy, and geologic regions is based on the current literature referenced herein and may differ slightly from the nomenclature presented in the CPNPP UFSAR for Units 1 and 2 (Reference 2.5-201).

2.5.1.1.1 Regional Physiography and Geomorphology

The discussion of the site region physiography is consistent with the terminology and demarcation of physiographic provinces presented on the Physiographic Map of Texas (Reference 2.5-202; Figure 2.5.1-201). CPNPP Units 3 and 4 are located in the Grand Prairie and North Central Plains physiographic provinces. However, several other physiographic provinces and subprovinces are encompassed within the site region. The Blackland Prairies and Interior Coastal Plains subprovinces of the Coastal Plain physiographic province are located southeast of the site. The central and eastern parts of the Edwards Plateau province and the Central Texas Uplift province are located southwest of the CPNPP Units 3 and 4 site. The North Central Plains physiographic province is located northeast of the site.

2.5.1.1.1.1 Grand Prairie Physiographic Province

The site vicinity and CPNPP Units 3 and 4 are located mostly within the southern portion of the Grand Prairie physiographic province; the northwestern-most portion of the site vicinity is located within the North Central Plains physiographic province. The Grand Prairie physiographic province is underlain by gently east- to southeast-dipping Early Cretaceous carbonate (limestone) and clastic units. Surface elevations range from about 450 to 1250 feet (ft) mean sea level (msl).

Limestones and shales underlie the eastern portions of the province. Erosion and weathering of these units has resulted in a dissected plateau with tablelands formed by resistant caprock, and less resistant shales cut by well-developed drainages that form a stair-step topography. This portion of the Grand Prairie is characterized by thin rocky soils developed from weathering of the calcareous and shaley bedrock.

More clastic rich, Early Cretaceous stratigraphic units, resulting from the westerly pinch-out of the carbonate section, underlie the western portion of the Grand

Prairie. These units are primarily sandstones, the more resistant of which form topography characterized by low hills.

2.5.1.1.1.2 North Central Plains Province

The North Central Plains physiographic province is located west and northwest of the CPNPP Units 3 and 4 site and the Grand Prairie physiographic province. The North Central Plains province is underlain by sandstones, shales, and limestones of upper Paleozoic age deposited on the Laurentian Margin. The exposed upper portions of the section record synorogenic sedimentation associated with Ouachita (Mississippian to Pennsylvanian Epochs) tectonism to the southeast.

The stratigraphic units of the North Central Plains dip to the west and are age-constraint-faulted. The more resistant lithologies result in a topographic expression characterized by low, north–south oriented cuestas, or ridges. Surface elevations range from 900 to 3000 ft msl.

2.5.1.1.1.3 Coastal Plain Province

To the southeast of the CPNPP site, the site region encompasses the Blackland Prairies and Interior Plains subprovinces of the Coastal Plain physiographic province. The Blackland Prairies forms the innermost margin of the Coastal Plain province and is named for the deep, rich, dark-colored soils that have developed from weathering of Late Cretaceous chalks and marls that underlie the province. The Blackland Prairies is a gently undulating surface, much of which is cultivated because of its rich, productive soil. The stratigraphic units dip gently south and east, and surface elevations range from about 450 to 1000 ft.

The Interior Coastal Plains province is located southeast of the Blackland Prairies. The rocks of the Interior Coastal Plains province post-date those of the Blackland Prairies and comprise both resistant and unconsolidated sands and clays that dip gently to the southeast towards the Gulf of Mexico. Differential weathering resistance of these units form parallel cuestas and intervening valleys, with surface elevations ranging from 300 to about 800 ft above sea level.

2.5.1.1.1.4 Edwards Plateau Province

The Edwards Plateau is located south and southwest of the CPNPP Units 3 and 4 site location. The eastern and southeastern boundary of the Edwards Plateau, which separates it from the Blackland subprovince of the Coastal Plain, is marked by a zone of normal faults that form the Balcones Fault Zone. Down-to-the-southeast (Coastal Plain side down) movement along this zone forms a boundary escarpment on the eastern and southern side of the plateau. This differential movement has resulted in the juxtaposition of Early Cretaceous Age limestones of the Edwards Plateau against the Late Cretaceous Age chalks and marls of the Blackland Prairies.

Most of the Edwards Plateau consists of a broad plateau of smooth plains and tablelands, with a stair-step topographic expression due to the erosion of interbedded hard carbonates and softer shales. However, this plateau is dissected near the Balcones Escarpment to form the Hill Country, where elevations range from about 450 to 3000 ft msl. In many locations, karstic dissolution of the more calcareous units is expressed topographically as sinkholes and caverns.

2.5.1.1.1.5 Central Texas (Llano) Uplift Province

The Central Texas (Llano) Uplift, although termed an uplift, is a large, enclosed, topographic basin located in the northern portion of the Edwards Plateau area. The basin is floored by metasedimentary and metaigneous crystalline rocks and is rimmed by lower Paleozoic sedimentary strata. These rocks represent the Laurentian Margin cratonic basement and platform shelf cover sequence that has been uplifted to shallow levels and exposed by erosion.

The floor of the basin consists of rolling topography with occasional hills 400 to 600 ft in relief that form from the more erosion-resistant granitic rocks. Surface elevations in this province range from 800 to 2000 ft msl. The concentric ridge around the outer edge of the basin comprises resistant rocks of Lower Paleozoic age. A second concentric outer rim is formed by erosion-resistant limestones of the Edwards Plateau.

2.5.1.1.2 Regional Stratigraphy and Geologic History

The site region (200-mi radius) for CPNPP Units 3 and 4 encompasses an area that is transected by the Laurentian cratonic edge, which formed by the breakup of the Rodinian continental mass in the Late Proterozoic Era and Early Cambrian Period. This breakup was accommodated in the site region by a pronounced change in orientation of the Laurentian Margin at this location from a northerly trend to a more east–west orientation. This change in orientation is now expressed in map patterns of the physiography and geologic units (Figures 2.5.1-201, Figure 2.5.1-202a and Figure 2.5.1-202b), in addition to expression by the regional gravity and magnetic fields (Figures 2.5.1-205 and 2.5.1-206). This change in the orientation of the Laurentian Margin has been interpreted as resulting from a triple point from which rifting was accomplished along the south-trending arm and the east–west-trending arm. The northwest-trending arm of the triple point became a failed rift that now forms the Southern Oklahoma Aulacogen (Reference 2.5-203). Subsequent interpretations have associated the features described above, with an oceanic transform in which the east-west-trending arm represents a transform margin and the northwest-trending arm a “leaky transform” (Reference 2.5-204). Regardless of origin, this basic structure forms a template that has affected the subsequent tectonic, stratigraphic, and structural development and associated geophysical expression for the region.

The controlling effects of the Laurentian cratonic margin on regional geologic expression include the structural trend of the Ouachita orogen and subsequent rifting that formed the Gulf of Mexico.

Subsections 2.5.1.1.2.1, 2.5.1.1.2.2, and 2.5.1.1.2.3 describe the geologic history and regional stratigraphy of the Laurentian Margin-cover sequence, the Ouachita tectonic phase, and the Gulf of Mexico, respectively (Figures 2.5.1-202 and 2.5.1-203). Figure 2.5.1-204 shows a regional geologic cross-section illustrating the structural relationships discussed below.

2.5.1.1.2.1 Laurentian Margin Basement—Cover Sequence

Several deep borings have sampled crystalline basement in the site region (Reference 2.5-205). This basement is also exposed in a structural dome that forms an erosional window through the overlying sedimentary cover sequence in the Central Texas (Llano) Uplift. The basement exposed at this location consists of Middle Proterozoic (1232 to 1303 million years ago, Ma; Reference 2.5-206) polydeformed and structurally imbricated metaigneous and metasedimentary rocks. The occurrence of rocks with oceanic affinities (References 2.5-207 and 2.5-208) and high-pressure metamorphism (Reference 2.5-209) indicates a Middle Proterozoic amalgamation phase of the North American craton. This polydeformed and structurally imbricated metaigneous and metasedimentary sequence has been intruded by a post-tectonic suite of granitic plutons, sills, and dikes that range in age from 1070 to 1116 Ma (Reference 2.5-206). In general, the crystalline crust in the site area consists primarily of cratonic crustal components subjected to Grenville-aged orogenic activity.

The Laurentian Margin was formed by Late Proterozoic to Early Cambrian rifting and breakup of a preexisting continental mass known as Rodinia (Reference 2.5-204). Magmatic and structural evidence of the early phases of this rifting event are preserved in the Southern Oklahoma Aulacogen that occurs in the northern portions of the site region (Figure 2.5.1-208) in the Arbuckle and Wichita Mountains (Wichita Igneous province). Exposed rift-related plutonic and volcanic activity in the Wichita Mountains consists of an early mafic component that was emplaced from the Late Proterozoic to the Cambrian, and a later granitic and rhyolitic phase at about 525 Ma that was accompanied by diabase dikes (Reference 2.5-210). The mafic, mantle-derived components of the igneous suite occur along the deep central axis of the aulacogen.

The later phases of Laurentian Margin development are characterized by stable margin, drift-related deposition. Drift-phase deposition resulted in a carbonate-dominated platform sequence characteristic of the Early Paleozoic eastern and southern Laurentian Margin. Rocks of the Cambrian and Ordovician section are preserved, but the Late Ordovician to Early Mississippian part of the section now occurs only as infilling of karstic features developed at the top of the Ellenburger Group (Figure 2.5.1-203).

2.5.1.1.2.2 Ouachita Tectonic Phase

The Laurentian Margin existed as a stable platform from the Late Cambrian Period until the Mississippian Epoch, when the platform was involved in southerly directed subduction beneath an oceanic volcanic arc proximal to Gondwana, and

subsequently the Gondwanian continental mass itself. The associated tectonism (Ouachita orogenesis) resulted in the obduction of the volcanic arc and adjacent accretionary wedge onto and over the Laurentian platform, with associated thrust imbrication of the Gondwanian oceanic arc–accretionary wedge and synorogenic clastic wedges deposited on the Laurentian platform itself. Comprehensive reviews of the tectonic history of the Ouachita event can be found in References 2.5-204 and 2.5-211.

The tectonic front associated with emplacement of the Ouachita thrust sheets and foreland deformation transects the site region and occurs in general spatial correlation with the previously existing Laurentian cratonic edge. The Ouachitian tectonic stratigraphy comprises an innermost (Laurentian side) frontal zone of thrust-imbricated, unmetamorphosed sediments that probably contain minor amounts of off-shelf to abyssal deepwater facies and pre-orogenic sediments, but consists mainly of synorogenic clastic material of the Atoka Group (Reference 2.5-212). The frontal zone is structurally overlain by a weakly metamorphosed interior core area that is composed of pre-orogenic deepwater facies siliceous rocks, carbonates, shales, and turbiditic sandstones flanked by Atoka Group synorogenic clastic material. Deformational intensity decreases to the south, away from the interior core, to transition into Late Paleozoic strata that dip consistently to the south. Here the deformed synorogenic clastic stratigraphy is overlain by undeformed, Middle Pennsylvanian, shallow marine-shelf facies and, in turn, Permian marine clastic and carbonate successor-basin deposits. This stratigraphy suggests an accretionary wedge composed of deepwater, abyssal material scraped off the ocean floor that was in turn overridden and offlapped by large volumes of clastic material resulting from erosion of an emerging high-tectonic topography (Reference 2.5-203).

On the Laurentian Margin, deposition of westerly and northerly prograding deltas (Atoka and Strawn Groups) composed of syntectonic clastic material occurred in association with the development of orogen-parallel, elongate troughs (foredeeps; e.g., Fort Worth Basin) that developed along with related arching of the crust (Bend Arch). These features resulted from the loading of the Laurentian Margin as the crust was thickened by thrust imbrication from the southeast. At least some of the flexure that resulted in development of these structures was accommodated by the development of high-angle, down-to-the-southeast faulting.

Deposition of shallow-water facies carbonates (Marble Falls Formation; Figure 2.5.1-203) occurred on the arches contemporaneously with the infilling of the foredeep basins by synorogenic shales and sandstones. However, as the flexural arches retreated in front of the advancing orogenic front, the shallow arch facies were overridden from east to west by the Atoka Group and Strawn Group deltas (Figure 2.5.1-203. Reference 2.5-213).

The waning phases of the Ouachitian orogeny are preserved in the Strawn and Canyon Groups that were the final sediments shed from the Ouachita foldbelt and associated uplift. The Strawn Group consists of sequences of shale and sandstone with some limestone and coal that accumulated as the foredeep basins

filled and shallower water conditions prevailed. The reduction in clastic input allowed open marine-shelf carbonates to spread over the subsiding delta platforms (Reference 2.5-213).

The effects of the Ouachita orogeny ceased earliest in the eastern portions of the orogenic belt (Late Pennsylvanian Epoch) in the site region and during the Early Permian Period in the western portions, and resulted in the final construction of the supercontinent Pangaea.

2.5.1.1.2.3 Gulf of Mexico Formation

Rifting of Pangaea led to the opening of the Gulf of Mexico in the Late Triassic and ended in the Middle Jurassic Periods. Rifting began in the Late Triassic to Early Jurassic by the formation of rapidly subsiding, extensional basins that filled with nonmarine clastic sediments and basaltic volcanics (Reference 2.5-214). Although no evidence of these basins is exposed at the surface, related sediments and igneous activity (Eagle Mills Formation) have been sampled by deep borings in eastern Texas and northwestern Louisiana (Reference 2.5-214). The later stages of the rifting event were marked by the widespread deposition of Middle Jurassic (Louanne, Werner) salt, which preferentially accumulated in the structural lows formed by the extensional-related basins. Extensional thinning and thermal relaxation of the newly formed margin led to subsidence, and the Gulf of Mexico rifted margin developed into a stable shelf in the Upper Jurassic Period on which a succession of shales and limestones were deposited (Reference 2.5-215).

As tectonically stable conditions prevailed and thermally controlled subsidence continued into the Early Cretaceous Period, stable margin development was accompanied by the deposition of a shallow-water carbonate and clastic transgressive–regressive sequences shoreward of a well-developed reef complex that developed at the edge of thick transitional crust (Reference 2.5-216). This is now represented by the Comanche series, which records these cycles as interbedded, clastic, shallow-water carbonate sediments (Reference 2.5-217). The Comanche series includes the Trinity Group and its Glen Rose and Paluxy formations outcrop within the CPNPP site area.

During the deposition of the Comanche series, the Gulf of Mexico was connected to both the Atlantic and Pacific Oceans. The northern margin of the Gulf of Mexico existed as a shallow-water shelf margin until the end of the Early Cretaceous, when the shelf was uplifted and marine deposition interrupted. In the Late Cretaceous, however, the shelf was again submerged and maximum transgression led to connection to the Pacific Ocean through the “Western Interior Seaway.” Beginning in the Coniacian Stage through the Maastrichtian Stage, increasing terrigenous input from westerly sources likely heralded the initial phases of tectonic uplift related to the Laramide orogeny (Reference 2.5-218).

In Late Cretaceous time, the northern and northwestern rim of the Gulf of Mexico was subject to alkaline volcanic activity. This activity was concentrated in southern

Arkansas and surrounding areas, and in the site region in the Balcones volcanic province (Reference 2.5-214). The Balcones volcanic province contains the most mafic to ultramafic compositions of the Late Cretaceous igneous sequence and represents intrusive activity from the Cenomanian to upper Maastrichtian Stages (approximately 69 to 100 Ma), with maximum activity occurring in the Campanian Stage (approximately 74 to 84 Ma) (Reference 2.5-214).

The Cenozoic history of the Gulf of Mexico is characterized by the influx of large amounts of clastic material along the major drainages beginning in the Paleocene and Eocene (Reference 2.5-219). The large-volume sediment resulted in crustal loading that became a main driving force for basin subsidence. Three distinct periods of sediment influx are recognized (Reference 2.5-219).

a. Late Paleocene to Early Miocene deposition in east Texas (lower Wilcox Group; Figure 2.5.1-203) resulting from early orogeny in the Rocky Mountains;

b. Oligocene deposition of the Vicksburg and Frio sequences of south Texas associated with calc-alkaline volcanism in Mexico and southwest Texas (Reference 2.5-214); and

c. Miocene to Quaternary deposition in coastal and offshore Louisiana that was associated with reactivation of the Rocky Mountains, uplift of the Colorado Plateau, eastward tilting of the Great Plains, and renewed uplift of the southern Appalachians.

2.5.1.1.2.4 Neogene and Quaternary Geologic History and Climate Change

The Neogene and Quaternary stratigraphic record for the site region reflects the effects of continental glaciation and associated climatic changes. Continental glaciation began in the southern hemisphere by the earliest Oligocene (Reference 2.5-474) and continued intermittently through several cycles through the Miocene (Reference 2.5-463). In the northern hemisphere, moderate-sized ice sheets began to develop in the Pliocene between 2.55 to 2.4 ma and the amount of ice underwent relatively mild 41,000 year periodic fluxuations that resulted mainly from orbital obliquity forcing (Reference 2.5-467). About 900,000 years ago, the amplitudes of the fluxuations doubled. About 800,000 years ago orbital eccentricity became the dominating climatic forcing mechanism and established a 100,000 year periodicity on the glacial cycle and climatic record (Reference 2.5-467). Over the last 850,000 years the Northern Hemisphere has seen ten major ice advances (Reference 2.5-453).

Transitions between glacial and interglacial periods and the transfer of mass between on land ice and oceanic water resulted in several worldwide effects that had consequences on sedimentation and isostacy for the site region. In addition to worldwide cyclic climatic changes, the accompanying transfer of mass to the oceans, in the form of melt water, and back to continental ice resulted in

alternating hydroisostatic depression of the oceanic crust and compensating uplift of nearby land masses accompanied by glacioeustatic sea level changes (Reference 2.5-472 and 2.5-456). Also, in association with hydroisostatic and glacioeustatic effects, the transfer of large quantities of melt water to the oceans down major drainages such as the Mississippi River resulted in large discharge and consequent sediment transport events.

In addition to eustatic and isostatic phenomena, the warming and cooling of the climate and associated alternating wet and dry periods, would have been accompanied by precipitation events of enhanced duration and amplitude (Reference 2.5-469), along with changes in vegetation density and weathering profiles that would have affected the susceptibility and rates of erosion and sediment production. World wide climatic, isostatic and eustatic consequences related to cyclic glaciation would have began in the Oligiocene with the accumulation of ice on the Antarctic landmass and become more prominent in the site region with the appearance of ice in the Northern Hemisphere during the Pliocene. However, the appearance of much larger glacial – interglacial amplitude events beginning in the late Pleistocene, with the establishment of 100,000 year eccentricity climatic forcing, resulted in associated large amplitude climatic fluxuations.

The onset of the effects of cyclic continental glaciation and associated climatic effects on the stratigraphic record probably resulted in uplift of the Rocky Mountains and Appalachians beginning in the Miocene and continuing to the present as recorded in increased clastic sedimentation in surrounding basins (Reference 2.5-465 and 2.5-466). The Ogallala Formation is composed of clastic detritus from the Rocky Mountains. Enhanced precipitation in the form of increased snow pack in the Rocky Mountains and intense fluvial activity during spring melt has been proposed for the cause of erosion and deposition of the Miocene – Pliocene Ogallala Formation, which occurs in the western portions of the site region (Pelletier, 2009). Regional uplift resulting from erosional unloading may be the result of isostatic rebound as a response to removal of material by enhanced erosion (Reference 2.5-465).

Neogene and Quaternary deposits in northern and northwestern portions of the site region include alluvium, eolian sand, lacustrine deposits, residuum, colluvium, cave and sinkhole deposits (Reference 2.5-455 and Reference 2.5-462). The Ogallala Formation forms the cap rock for the southern high plains (Reference 2.5-459) which occurs in western and northwestern portions of the site region (Figure 2.5.1-1). Incision of the cap rock surface by tributaries of the Colorado, Brazos and Red Rivers has resulted in steep-walled valleys that are probably late Pleistocene in age (Reference 2.5-459). These valleys began to store sediment intermittently about 11,000 years B. P. and now contain 3 to 5 meters of alluvial, lacustrine and eolian deposits. Alluvial sand and gravel was deposited prior to 10,000 years B.P. followed by lacustrian deposition that abruptly began about 11,000 years B. P. and continued in most places until about 8500 years B. P. Localized eolian deposits are recorded beginning about 10,000 years ago but became widespread from 9000 to 5500 B. P. and ended about 4500 B. P.

(Reference 2.5-459). Quaternary intrastratal dissolution of subsurface Permian evaporites resulted in subsidence basins and sinkholes that controlled deposition of the Quaternary sediments (Reference 2.5-455).

In other portions of the site region alluvial deposition associated with fluvial systems are significant components in the Neogene and Quaternary stratigraphic record. Madole et al. (Reference 2.5-462) recognize alluvial deposits that occur in three distinct settings associated with river valleys in northern and northwestern portions of the site region. These settings include upland areas, terrace deposits along valley sides and deposits beneath valley floors. The upland deposits occur as poorly preserved, arkosic residuum on hill crests and divides, some of which are not associated with interfluves of current drainages. The upland deposits consist of a diverse assemblage of vein quartz, quartzite, volcanic, plutonic and metamorphic rocks eroded from the Rocky Mountains, and some may be reworked Ogallala Formation. These deposits are early Pleistocene or older in age in that they occur above (higher in elevation) terrace deposits that contain Lava Creek B volcanic ash (Reference 2.5-462 and 2.5-468). These deposits probably result from enhanced erosional and sediment transport due to pre-middle Pleistocene climatic events, before the current drainage system was established.

Alluvium that constitutes terrace and valley floor deposits consist of two lithofacies (Reference 2.5-462). The stratigraphically lower facies consists of coarse-grained material deposited during lateral accumulation events and are predominately gravel in larger drainages but may be coarse sand in smaller drainages. The overlying facies is fine-grained and consists of silts, clays and fine sand that represent vertical accumulative events in overbank and flood plain settings. The fine-grained facies is typically thicker than the coarse grained facies except in floodplains and the total thickness of alluvium varies directly with stream size (Reference 2.5-462).

The detailed signature of climatic change and other related phenomena such as base level changes due to glacioeustasy on the continental fluvial record are uncertain (Ethridge et al., 1998). Climatic effects due to cyclic glaciation have been proposed to have affected fluvial terrace development globably (Reference 2.5-454). Pliocene obliquity forced climatic cycles are associated with relatively broad sheet-like fluvial deposits with well-defined terrace development beginning with the transition to 100,000 year climatic forcing about 900,000 years ago. Bridgland and Westaway (Reference 2.5-454) proposed a worldwide correlation in terrace formation with the 100,000 year eccentricity cycle. However, the detailed response to climatic forcing and associated uplift required for terrace formation is dependant on the age of the crust and related to the thermal profile and consequently strength of the lower crust (Reference 2.5-475).

The effects of Quaternary climate cyclisity on channel morphology have been documented in central Texas for the Colorado River (Reference 2.5-454). Low sinuosity phases in channel morphology were found to be associated with poorly sorted, coarse-grained sediment with no silt and clay. In contrast, meandering

channel morphology was associated with transportation of silt, clay and fine sand. Meandering morphology results from relatively humid conditions that promoted thick soil development and vegetative cover, and inhibited erosion. During arid conditions the plant cover would diminish and result in increased sediment load due to enhanced erosion.

Late Quaternary climatic changes for central Texas have been reconstructed based on the analysis of vertebrate, pollen and plant macrofossil data in addition to carbon and oxygen isotopes. Nordt et al. (Reference 2.5-454 and 2.5-464) present a late Quaternary climatic reconstruction from central Texas and compare the results to several other regional reconstructions for the same climatic period. These studies generally indicate progressive warming and dryer conditions from the late Pleistocene glacial conditions from 11,000 to 8000 years B. P. This transitional period was followed by middle-Holocene warm and dry conditions that probably represents the Holocene climatic optimal (Altithermal). By about 4000 years B. P. Conditions had returned to those present at the end of the transitional period from the late Pleistocene cool and wet conditions at about 8000 years B. P. and persist to the present. However, there was a possible brief period of dry and warm conditions around 2000 years B. P. These studies all indicate late Pliestocene wet conditions followed by progressively warmer and drier conditions. Based on their analysis of the lower Brazos River, Silva and Galloway (Reference 2.5-454) proposed that the late Pliestocene in central Texas was dominated by super El Nino precipitation events that occurred on millennial scale climatic subcycles and resulted in river valley erosion and filling.

Reconstructed environments from the Edwards Plateau (Reference 2.5-471) indicate that there was more moisture present during full glacial conditions than any time since. By about 13,000 years B. P., summer temperatures had warmed considerably from full glacial conditions and were within a few degrees of those of today. During the late glacial period (14,000 – 10,500 years B. P.) moisture decreased and then increased. After this time, fundamental changes in fauna at the end of the Pleistocene indicate the climate became warmer and dryer so that the early- and middle-Holocene was characterized by a protracted decrease in effective moisture. During this period at about 8000 years B. P., vertical accretation of sediments doubled, and deposits changed color and composition to larger clast sizes indicating that upland soil mantles were being stripped (Reference 2.5-471). This trend resulted in conditions dryer than modern time in the early part of the late Holocene from about 5000 to 2500 years B. P. More moist condition prevailed from 2500 to 1000 years B. P., at which time the drought-prone present climatic cycle was established.

Similar climatic conditions for the Holocene in Texas are documented by carbon isotopic fractionation (Reference 2.5-459), which indicate dry conditions during the middle Holocene from about 8000 years B. P. and ending between 6500 to 4000 years before present. However, Humphery and Ferring (Reference 2.5-461) correlate these dry conditions with slow alleviation and a period of soil formation.

The effects of late Quaternary climatic changes on the fluvial characteristic of the Brazos River to the southeast of the site vicinity (Reference 2.5-473) indicate that during the last transition from glacial conditions during the late Pleistocene (18,000 to 8500 year B.P.) that the Brazos was a high discharge meandering stream that meandered across its entire flood plain. These conditions resulted in primarily lateral accretion that produced a thick layer of coarse-grained basal deposits that fined upward. In the period following 8500 years B.P., discharge decrease resulted in under fit conditions so that the river became confined to relatively narrow channels and meander belts within the older extensive floodplain, and deposition resulted in primarily vertical accretion. The reduced discharge was unable to incise into the coarse clastic material at the base of the old floodplain, so the older floodplain formed a foundation for subsequent depositional sequences. The meander belts periodically abandoned their channels and shifted location each time producing fluvial depositional sequences capped by paleosols. Each of these depositional sequences has been correlated with climatic episodes (Reference 2.5-473). Significant vertical accretion episodes occurred between 8100 to 4200 years B. P. and 2500 to 1250 years B. P., separated by paleosol development from 4200 to 2500 years B. P. and 1250 to 500 years B. P. After this period, vertical accretion slowed to the present time although avolution continued. The modern channel was established around 300 year B. P. These results are complemented by Hall (Reference 2.5-460) which also indicates a period of stream incision in the site region at about 1000 years B.P.

Glacioeustatic-driven base level changes are well documented on the Texas Gulf Coast, especially for the late Quaternary (Reference 2.5-448). In this setting the eustatic effects overwhelm those of climate change (Reference 2.5-448) and have resulted in the development of well-defined stratigraphic sequence systems tracts that record rise and fall of sea level in the late Quaternary (Reference 2.5-451) in addition to climatic effects (Reference 2.5-450).

2.5.1.1.3 Regional Gravity and Magnetic Features

Subsection 2.5.1.1.3 describes the regional gravity and magnetic data available for the site region and relates the anomalous features in the regional gravity and magnetic fields to their regional sources.

2.5.1.1.3.1 Regional Gravity Field

The Bouguer gravity field (Figure 2.5.1-205, A and B) was obtained from the gravity database provided by the University of Texas at El Paso PACES Geonet. The isostatic gravity field (Figure 2.5.1-205, C and D) was obtained from the U. S. Geological Survey “Texas Magnetic and Gravity Maps and Data” (Reference 2.5-220). On the isostatic map, positive anomalies indicate upper crustal material with densities greater than the Bouguer reduction density of 2.67 g/cc and negative anomalies indicate upper crustal material less than 2.67 g/cc.

Kruger and Keller (Reference 2.5-221) and Keller, et al. (Reference 2.5-222) describe the Bouguer gravity field for the site region (Figure 2.5.1-205, A and B). As described by Kruger and Keller (Reference 2.5-221) and Keller, et al. (Reference 2.5-222), the gravity field exhibits characteristics correlative with the presence of the Laurentian cratonic edge in the subsurface. This is manifested as a broad area of relatively low gravity in western portions of the site region. Bouguer gravity values in this area rise from a little above -120 milligals (mGal) about 50 mi west-northwest of the site to -40 mGal about 40 mi east-southeast of the site (Figure 2.5.1-205). Isostatic gravity values do not exhibit the same southeasterly increasing, steady positive gradient, but are instead relatively constant at about -50 to -40 mGal, with an increase to about -20 mGal in the site vicinity. This is due to the relatively thick, low-density crustal material of the Laurentian craton that underlies this portion of the site region. The influence of the crustal root of the southern Rocky Mountains that occurs west of the site region explains the difference in response between the Bouguer and isostatic gravity fields, as the Bouguer gravity is uncompensated for the thicker, low-density material in the subsurface.

About 40 mi east of the site both the Bouguer and isostatic gravity fields exhibit a steep gradient with isostatic values increasing from -20 to +30 mGal over an interval of about 20 mi. This steep gradient marks the edge of the Laurentian craton to the west and northwest and thinner, attenuated transitional crust due to thinning associated with extension leading to Gulf of Mexico formation in the east (Reference 2.5-219).

Just to the east and southeast of this transition, a parallel interior-zone gravity maximum occurs associated with several parallel elongate gravity highs. Gravity modeling (Reference 2.5-222) has shown that the first-order effect resulting in this feature is due to a major transition in crustal structure as discussed above. However, some of the second-order features associated with this anomaly are probably due to a variety of sources including metamorphism and crustal imbrication in the Ouachita orogenic core, basement uplifts, and mafic intrusions (Keller, et al., Reference 2.5-222). In general, the Bouguer gravity field increases towards the Gulf of Mexico due to the thinning continental crust and the increasing influence of oceanic crustal material. However, this long wavelength increase in the field is locally influenced by the presence of low-density, diapiric salt structures associated with the East Texas salt basin (Figure 2.5.1-205).

In the northern portion of the region, a northwesterly trending linear anomaly comprising several gravity maxima marks the position of the interior of the Southern Oklahoma Aulacogen. The source of this anomaly is the dense mafic rocks that compose the centerline of the rifted trough that forms the aulacogen. In addition, these higher-density rocks have been brought nearer to the surface due to the Wichita–Criner and Arbuckle uplifts developed in association with the Late Paleozoic Ouachita orogeny (Figure 2.5.1-205).

In the southwestern portion of the region, a circular anomaly that includes several subordinate circular and elliptical gravity highs marks the location of the Llano

Uplift. At this location the low-density sedimentary cover has been stripped from the basement, bringing relatively denser material to the surface.

A circular gravity low occurs in the southern part of the Fort Worth Basin (Figure 2.5.1-205). The source of this low is low-density sedimentary basin infilling material. However, the gravity signature of the northern part of the basin is not as well defined due to interference effects from surrounding high-density sources.

In summary, the western and northwestern portions of the site region exhibit gravity field characteristics consistent with the fact that this area is underlain mainly by thick, low-density crust characterized in general by isostatic gravity values of about -50 mGal. Local variations in this regional field are due to density variations caused by low-density sedimentary material in depositional basins, high-density mafic material, or areas where low-density sedimentary cover is not present. The southeastern portion of the site region is characterized by a higher regional gravity field that is the result of a transition to thinner crust associated with extension during the formation of the Gulf of Mexico. The tectonic features imaged in the gravity data can all be associated with early Paleozoic development of the Laurentian Margin, Late Paleozoic orogeny resulting from the Ouachita event, or rifting and subsequent deposition related to the development of the Gulf of Mexico.

2.5.1.1.3.2 Regional Aeromagnetic Field

The regional magnetic field (Figure 2.5.1-205) was obtained from the U. S. Geological survey, “Texas Magnetic Gravity Maps and Data,” (Reference 2.5-220) which is a compilation of several proprietary and non-proprietary data sets. The component datasets were obtained at different flight line spacings, so the data resolution is variable throughout the site region. However, the final map (Figure 2.5.1-206) was processed with a constant grid spacing of 1000 meters and shows the magnetic field measured or calculated at 1000 ft aboveground.

The regional aeromagnetic field (Figure 2.5.1-206) exhibits broad-scale features correlative with those exhibited by the gravity field. The crustal transition marked by the edge of the Laurentian craton is evident in the character of the magnetic field anomalies. The western and northwestern portions of the site region associated with thick Laurentian cratonic crustal material is characterized by abundant circular, elongate, and linear magnetic dipole anomalies due to magnetic sources in the crystalline basement that probably represents more mafic materials. The most prominent of these is the northwest-trending linear magnetic anomaly that is associated with the mafic material in the axial core of the Southern Oklahoma Aulacogen. However, the signature of the aulacogen has been locally modified by thrusting associated with the Ouachita orogeny, which raised and juxtaposed crustal blocks of different magnetic susceptibilities (Reference 2.5-223). In this portion of the site region, thick accumulations of nonmagnetic sediments associated with depositional basins are not an anomaly source. However, these nonmagnetic blankets of material subdue and dampen magnetic anomaly magnitudes and gradients.

A series of isolated magnetic dipoles is associated with the gravity gradient and maximum that marks the edge of the Laurentian craton and the interior-zone gravity maximum. In contrast to the western and northwestern portions of the site region, the magnetic signature to the southeast is characterized by magnetic anomalies exhibiting relatively low values and subdued gradients. This response is due to the relatively thick blanket of nonmagnetic sedimentary material associated with the Gulf of Mexico Coastal Plain that serves to attenuate the underlying magnetic sources.

In concert with the regional gravity field, the features in the regional magnetic field can be attributed to development of the Laurentian Margin, the Ouachita orogeny, or development of the Gulf of Mexico. The location of the Meers fault, the only capable tectonic source recognized in the site region, is marked by the presence of a steep magnetic gradient (Figure 2.5.1-206) along its southeastern extension. However, this signature is the result of the juxtaposition of material of different magnetic susceptibilities during Late Paleozoic thrusting and not an expression of the recent kinematic history of the fault (Reference 2.5-223).

2.5.1.1.4 Regional Tectonic Setting

The CPNPP Units 3 and 4 site region is located within the Central and Eastern United States (CEUS), a stable continental region characterized by low rates of crustal deformation and no active plate boundary conditions. In 1986, the Electric Power Research Institute (EPRI) developed a seismic source model for the CEUS that included the CPNPP Units 3 and 4 region (Reference 2.5-369). This seismic source model was developed using the interpretations provided by six independent Earth Science Teams (ESTs) and aimed to reflect the general state of knowledge of the earth science community as of 1986. The source models developed by the ESTs combined tectonic setting and rates and distribution of historical seismicity; the models are summarized in Subsection 2.5.2.2. The following subsection summarizes the current state of knowledge of the tectonic setting and tectonic structures in the CPNPP site region, with a focus on post-1986 geologic, seismologic, or geophysical information that is relevant to assessing potential for seismic activity in the region.

2.5.1.1.4.1 Regional Tectonic History of the CPNPP Units 3 and 4 Site

Figure 2.5.1-207 shows the principal tectonic structures and features in the CEUS and within the 200-mi-radius CPNPP site region. Most of the Paleozoic structures are regional in scale and recognizable in geologic and geophysical data. There is generally some correlation between a tectonic structure’s physiographic province and the structure’s age and style. Figure 2.5.1-201 shows the physiographic provinces within the CPNPP site region.

The CPNPP Units 3 and 4 site lies in the Grand Prairie physiographic province (Figure 2.5.1-201). This province and the entire site region have a complex tectonic history beginning in the Proterozoic with igneous activity, metamorphism, and uplift. This early history is recorded in only limited Precambrian exposures

such as the Llano Uplift. In the Early Cambrian, the ancestral North American continent, Laurentia, underwent rifting along its southeastern margin. This rifting and subsequent subsidence and deposition are recorded by Early Paleozoic sedimentary sequences exposed throughout Texas and Oklahoma. Sedimentary basins developed in the Early Paleozoic were inverted and uplifted during Late Paleozoic compression associated with the Ouachita orogeny that accompanied the closing of the Paleozoic ocean basin along the southeastern margin of Laurentia. In the Late Triassic, continental rifting began again and continued until the Middle Jurassic as the Gulf of Mexico opened. The passive margin sedimentary sequences of the Mesozoic were overridden by Tertiary sediments shed from the Laramide orogeny to the northwest.

2.5.1.1.4.1.1 Late Proterozoic and Paleozoic Plate Tectonic History

The earliest record of tectonism in the site region is the deformation associated with the formation of the Rodinia supercontinent in the Mesoproterozoic. The only records of this folding, thrusting, and high-grade metamorphism are found in the Llano Uplift in central Texas. Rodinia existed for the rest of the Proterozoic, but began to break up in the Late Proterozoic to Cambrian as the Iapetus Ocean opened south and east of ancestral North America, Laurentia. This Early Paleozoic rifting of Laurentia away from Rodinia left a series of extensional troughs, or aulacogens, along the paleo-boundary of North America (Figure 2.5.1-208). The Southern Oklahoma Aulacogen, which stretched from eastern New Mexico across the Texas Panhandle to southern Oklahoma, accommodated the deposition of sedimentary sequences greater than 10,000 ft thick (Reference 2.5-203). Extension was accompanied by igneous activity in the Cambrian (References 2.5-225 and 2.5-226) and was followed by subsidence and sedimentation as shallow-water marine sedimentary rocks, such as the Ellenburger Group in Texas and the Arbuckle Group in Oklahoma, covered much of what is now Oklahoma and North Texas (References 2.5-227 and 2.5-228). Subsidence and deposition continued throughout the Early Paleozoic in most of Texas.

In the Late Paleozoic, beginning in the Mississippian Epoch, the Iapetus Ocean basin to the south and east of Laurentia closed as a south-dipping subduction zone migrated northwards. The resulting collision between Laurentia and combined Europe-Africa-South America led to the development of Pangaea in the Permian. Continent-scale thrust belts traceable around this orogen (the Ouachita-Appalachians; the Caledonides in Europe) record this collision. The portion of this orogen that is of interest to the tectonic history of the CPNPP site is the Ouachita Fold and Thrust Belt. This series of north- and northwest-directed thrusts is exposed and/or recorded in the subsurface across Arkansas, southeastern Oklahoma, and along a northeast-southwest trend across eastern and central Texas. Within the CPNPP site region, the Ouachita Fold and Thrust Belt is largely covered with later sedimentary sequences. The Permian collision culminated, at different times across the orogen, with the eastern Ouachita thrusts active in the Carboniferous followed by the Late Carboniferous to Early Permian thrusting of the Marathon Uplift in west Texas (Reference 2.5-222). This belt of thrusting

shares its origins with the north-south-trending belt of west-vergent thrusts that developed along the Appalachians from Georgia to New England due to the same collision (Reference 2.5-204). Furthermore, the Late Paleozoic Ouachita orogeny was responsible for the compressional reactivation of many originally extensional structures associated with the Precambrian to Early Paleozoic Southern Oklahoma Aulacogen rifting (Reference 2.5-228).

2.5.1.1.4.1.2 Mesozoic and Cenozoic History

The continental collision marked by the Late Paleozoic Ouachita Fold and Thrust Belt marked the end of the Wilson cycle that began with the Cambrian opening of the Iapetus Ocean recorded by the Southern Oklahoma Aulacogen. Another Wilson cycle began shortly thereafter with the opening of the Gulf of Mexico. Formation of the Gulf of Mexico Basin began in the Late Triassic time with rifting subparallel to the trend of the Ouachita belt (Reference 2.5-215). Detailed modeling of gravity data (Subsection 2.5.1.1.3.1) suggests that the locus of rifting and crustal extension occurred south of the main Ouachita collisional orogen, approximately beneath the present continental shelf and rise in the offshore region of the Gulf of Mexico (Reference 2.5-229).

Rifting lasted from the Late Triassic to the Late Jurassic and caused both the extension of pre-rift continental crust and the formation of new oceanic crust. This Mesozoic to Cenozoic process of rifting and subsidence associated with the opening of the Gulf of Mexico primarily affected the southeastern portion of the CPNPP Units 3 and 4 site region (Figure 2.5.1-207). Within the entire CPNPP Units 3 and 4 site region Sawyer, et al. (Reference 2.5-230) describe three classifications related to the rifting: (1) extended continental crust, (2) extended thick transitional crusts, and (3) extended thin transitional crust.

The initial stages of rifting occurred during the Late Triassic and Early Jurassic and are thought to have occurred along pre-existing crustal weaknesses and sutures from the earlier Precambrian-Cambrian rifting and Late Paleozoic Ouachita orogeny (Reference 2.5-230). The majority of rifting occurred in the Middle Jurassic and created the divisions between continental, thick transitional, and thin transitional crust (References 2.5-230 and 2.5-231). The thick transitional crust underwent moderate thinning, with post-extension crustal thicknesses ranging between 12.4 and 21.7 mi. This variable thinning occurred along gulf-perpendicular trends (Reference 2.5-230) and is proposed by some to influence the formation of the gulf-perpendicular basement highs and lows that form the alternating uplifts and embayments of the Gulf coastal margin (e.g., Sabine Arch), (Reference 2.5-232; Figure 2.5.1-207).

The thin transitional crust underwent considerably more thinning than the transitional crust described above, exhibiting post-extension crustal thicknesses of 5 to 9.3 mi (References 2.5-230 and 2.5-233). Sawyer, et al. (Reference 2.5-230) hypothesized that the contrast in thinning is due to the thin transitional crust having originally been weaker due to locally elevated crustal temperatures. In contrast to the thick transitional crust, the major crustal thickness variations in the

thin transitional crust are parallel to the Gulf Margin (Figure 2.5.1-207, Reference 2.5-230). Throughout the period of rifting, significant accumulations of non-marine clastic rocks, volcanic rocks, and salt were deposited in fault-bounded basins (References 2.5-215 and 2.5-230). Further rifting allowed for the development of typical oceanic crust in the center of the Gulf of Mexico Basin, to the southeast of the CPNPP Units 3 and 4 site region, in the Middle and Late Jurassic (References 2.5-230 and 2.5-231).

After the rapid phase of continental extension and rifting ended, a long period of tectonic quiescence ensued during which the new passive margin subsided and allowed for the deposition of thick, Late Jurassic, and Cretaceous marine sediments (Reference 2.5-215). The CPNPP Units 3 and 4 site was within the Cretaceous seaway that extended northwest from the Gulf around the uplifted Llano Uplift and Ouachita highlands to connect to the Great Western Seaway (Reference 2.5-218). At the very end of the Late Cretaceous, the Laramide orogeny to the northwest began to influence the site region. The Laramide orogeny resulted from the eastward subduction of the Kula and Farallon plates beneath western North America. A shallowing of the subducting slab led to a series of high-angle thrusts and thickened crust throughout western North America from Mexico to Alaska and led to the development of the Rocky Mountains (Reference 2.5-234). The erosion of these mountains provided a source for much of the clastic sediment deposited in the site region throughout the Cenozoic (Reference 2.5-219). The loading of the margin with these sediments led to further subsidence within and around the Gulf of Mexico Basin. This subsidence was partially accomplished via a series of margin-parallel normal faults, the nearest of which are located about 200 mi from the site (Reference 2.5-235).

The margin-parallel normal faults, or growth faults, began accommodating sedimentary compaction, subsurface salt migration, and basin-side-down slumping of the Gulf Coastal Plain in the Late Mesozoic (References 2.5-215 and 2.5-228). This faulting continues into recent time, but does not penetrate the crystalline basement rocks beneath the Gulf. These growth faults are confined to sediments and poorly lithified rocks, and therefore are not likely to sustain seismic ruptures that can produce damaging ground motions (Reference 2.5-236).

The other Cenozoic tectonic event of interest to the site is the development of the Rio Grande Rift system in New Mexico and westernmost Texas and the Basin and Range Province farther west. These features are outside of the site region, but their Miocene development led to broad epeirogenic uplift and erosion of the Paleocene and Upper Cretaceous strata in central Texas. The resulting flexure of the lithosphere occurs along a northeast-southwest-trending line between the uplifted Edwards Plateau of central Texas (on which the CPNPP is sited) and the down-to-the-southeast warped coastal plain to the southeast. The northeast-southwest-trending Balcones and Luling-Mexia-Talco fault zones are spatially associated with this hingeline and geomorphic transition from the Edwards Plateau to the interior zone of the Gulf Coastal Plain (Reference 2.5-237). These fault zones were active in the Late Oligocene or Early Miocene (Reference 2.5-

237), and were probably driven by the crustal flexure and tilting associated with sedimentary loading of the Gulf of Mexico Basin. The tectonic activity of the CPNPP site region since the Miocene has been minimal. The site region has predominately undergone local erosion and deposition along rivers and drainages while transporting sediments shed from the Rockies and the Appalachians south to depocenters in the Gulf of Mexico.

2.5.1.1.4.2 Tectonic Stress

Three types of forces are generally responsible for the stress in the lithosphere:

a. Gravitational body forces or buoyancy forces, such as the ridge-push force resulting from hot, positively buoyant young oceanic lithosphere near the ridge against the older, colder, less buoyant lithosphere away from the ridge (Dahlen, Reference 2.5-238). This force is transmitted by the elastic strength of the lithosphere into the continental interior.

b. Shear and compressive stresses transmitted across plate boundaries (such as strike-slip faults or subduction zones).

c. Shear tractions acting on the base of the lithosphere from relative flow of the underlying asthenospheric mantle.

Earth science teams (ESTs) that participated in the EPRI (Reference 2.5-369) evaluation of intra-plate stress found that tectonic stress in the CEUS region is primarily characterized by northeast-southwest-directed horizontal compression. In general, the ESTs concluded that the most likely source of tectonic stress in the mid-continent region was ridge-push force associated with the Mid-Atlantic Ridge, transmitted to the interior of the North American Plate by the elastic strength of the lithosphere. Other potential forces acting on the North American Plate were judged to be less significant in contributing to the magnitude and orientation of the maximum compressive principal stress.

In general, the ESTs focused on evaluating the state of stress in the mid-continent and Atlantic seaboard regions, for which stress indicator data were relatively abundant. Fewer stress indicator data were available for the Gulf of Mexico, Gulf Coastal Plain, and Western Great Plains, and thus these areas received less scrutiny in the EPRI (Reference 2.5-369) studies. Notably, the Dames & Moore, Law Engineering, and Bechtel ESTs observed that the orientation of maximum horizontal compression in the Gulf Coastal Plain and west Texas may be perturbed from the regional northeast-southwest orientation that characterizes much of the CEUS.

Since 1986, an international effort to collate and evaluate stress indicator data culminated in publication of a new World Stress Map in 1989 (References 2.5-239 and 2.5-240) that has been periodically updated (Reference 2.5-241). Plate-scale trends in the orientations of principal stresses were assessed qualitatively based on the analysis of high-quality data (Reference 2.5-242) and previous delineations

of regional stress provinces were refined. Statistical analyses of stress indicators confirmed that the trajectory of the maximum compressive principal stress is uniform across broad continental regions at a high level of confidence (Reference 2.5-243). In particular, the northeast-southwest orientation of principal stress in the CEUS inferred by the EPRI ESTs is statistically robust and is consistent with the theoretical orientation of compressive forces acting on the North American Plate from the Mid-Atlantic Ridge (Reference 2.5-242).

According to the continental U.S. stress map of Zoback and Zoback (Reference 2.5-239), the site is located in the Mid-Plate Stress province, a large area of the CEUS that displays a consistent northeast-southwest maximum compressive stress orientation (Figure 2.5.1-209). Portions of the site region are also in the Southern Great Plains Stress province, which is characterized by a northeast-southwest-oriented extensional stress regime, and the Gulf Coast Stress province, which is characterized by northeast-southwest to north-northeast to south-southwest horizontal tension (Reference 2.5-239).

2.5.1.1.4.2.1 Mid-Plate Stress Province

The Mid-Plate Stress province characterizes most of the CEUS (Figure 2.5.1-209). This province may exhibit reverse or strike-slip faulting under east-northeast- to west-southwest- to northwest-southeast-oriented compressive stress. This region extends from an approximately north-south-oriented line through Texas, Colorado, Wyoming, and Montana east all the way to the Atlantic Margin and potentially into the Atlantic Ocean Basin (Reference 2.5-239). Within this province, the orientation of maximum compressive stress is generally parallel to plate velocity direction (Reference 2.5-240). Richardson and Reding (Reference 2.5-244) were able to reproduce the northeast-southwest orientation of principal stress in CEUS with numerical models that assume horizontal shear tractions acting on the base of the North American Plate from the underlying asthenospheric mantle. Humphreys and Coblentz (Reference 2.5-245) concluded that a dominant control on the northeast-southwest orientation of the maximum compressive principal stress in the CEUS is ridge-push force from the Atlantic Ocean Basin.

Richardson and Reding (Reference 2.5-244) concluded that the observed northeast-southwest trend of principal stress in the Mid-Plate Stress province of the CEUS dominantly reflects ridge-push body forces associated with the Mid-Atlantic Ridge. They estimated the magnitude of these forces to be about 2 to 3 x 10^{12} N/m (i.e., the total vertically integrated force acting on a column of lithosphere 3.28 ft wide), which corresponds to average equivalent stresses of about 40 to 60 MPa distributed across a 30-mi-thick elastic plate.

Using numerical models, Humphreys and Coblentz (Reference 2.5-245) evaluated the contribution of shear tractions on the base of the North American lithosphere to intra-continental stress, and concluded the following:

a. There is a viscous drag or resisting force acting on the cratonic root of North America as it moves relative to the asthenospheric mantle, and this drag supports part of the ridge-push force acting from the east and creates a stress shadow for the western U.S.

b. Shear tractions on the base of North America from flow of the underlying asthenospheric mantle are a minor contribution to stress in the mid-continental lithosphere.

Humphreys and Coblentz (Reference 2.5-245) concluded that the dominant control on the northeast-southwest orientation of the maximum compressive principal stress in the CEUS is ridge-push force from the Atlantic Ocean Basin.

2.5.1.1.4.2.2 Southern Great Plains Stress Province

The tensile stress regime in the Southern Great Plains Stress province was interpreted by Humphreys and Coblentz (Reference 2.5-245) to arise from positive buoyancy forces associated with the high potential energy of the elevated Cordilleran topography to the west. Essentially, the tensile stress in the western Cordillera, and in the Southern Great Plains along its southeastern flank, is an on-land version of the ridge-push buoyancy force. The magnitude of the positive buoyancy force and resulting tensile stress decays eastward in the Southern Great Plains Province, coincident with the eastward decrease in topography and potential energy from the southern Rocky Mountains to the interior of the continent as noted by Zoback and Zoback (Reference 2.5-239).

Zoback and Zoback (Reference 2.5-239) interpreted the Southern Great Plains Province to be a transition between tensile stress and active extension in the Basin and Range to the west, and compressive stress in the tectonically stable mid-continent to the east. The boundary between the Mid-Plate and Southern Great Plains Stress Provinces is shown as approximately located by Zoback and Zoback (Reference 2.5-239), reflecting the paucity of stress indicator data to precisely constrain the location of the boundary. Zoback and Zoback (Reference 2.5-239) observed that the Southern Great Plains Province “generally coincides with the major topographic gradient (about 100 m, 225 km) separating the thermally elevated western Cordillera from the mid-continent area.” If this correlation is valid in Texas, then the boundary between the Mid-Plate and Southern Great Plains Stress provinces probably is located near the eastern foot of the mountains in West Texas, west of the site.

2.5.1.1.4.2.3 Gulf Coast Stress Province

The southeastern portion of the site region is in, or adjacent to, the Gulf Coast Stress province. This province generally coincides with the belts of growth faults in the coastal regions of Texas, Louisiana, Mississippi, Alabama, and northwestern Florida. The Gulf Coast Stress province is characterized by north-south-directed tensile stress (Reference 2.5-239) and is spatially associated with down-to-the-Gulf extension and slumping of the Coastal Plain stratigraphic section. Because

these strata are deforming above subhorizontal detachment faults and/or large bodies of Jurassic salt, gravitational tensile stress driving growth faulting is confined to the sedimentary section, and thus decoupled mechanically from the state of stress in the underlying crystalline basement.

The state of stress in the crystalline basement underlying the Coast Plain strata is very poorly constrained (Reference 2.5-244) and may be affected by flexural loading of the lithosphere due to rapid and voluminous sedimentation in the Gulf of Mexico during the Pleistocene. Detailed numerical modeling of flexural deformation associated with sedimentary loading in the Gulf by Nunn (Reference 2.5-246) suggested that large bending stresses may be present in the crust and systematically vary from north-south tension in the Coastal Plain, to north-south compression in an approximately 62-mi-wide zone in the northern offshore region directly adjacent to the coast, to north-south tension at distances of greater than 62 mi from the coast.

To summarize, research on the state of stress in the continental U.S. since the publication of the EPRI (1986) studies has confirmed observations that stress in the CEUS is characterized by relatively uniform northeast-southwest compression, and that this regional trend may be perturbed at distances beyond 150 mi from the CPNPP Units 3 and 4 site due to the influence of buoyancy forces in the uplifted Cordillera to the west, and flexure of the crust due to sedimentary loading of the Gulf of Mexico to the southeast. Very few new data have been reported since the EPRI (Reference 2.5-369) study to better determine the orientations and relative magnitudes of the principal stresses in the site region. Given that the current interpretation of the orientation of principal stress is similar to that adopted in EPRI (Reference 2.5-369), a new evaluation of the seismic potential of tectonic features based on a favorable or unfavorable orientation to the stress field would yield similar results. Thus, there is no significant change in the understanding of the static stress in the site region since the publication of the EPRI source models in 1986, and there are no significant implications for existing characterizations of potential activity of tectonic structures.

2.5.1.1.4.3 Principal Tectonic Features

The tectonic features within the site region are discussed below, categorized by their age of movement or activity. Generally, these features were most recently active in either the Late Paleozoic (associated with the Ouachita orogeny) or Mesozoic to Eocene (related to the opening of the Gulf of Mexico). Specifically, workers have found evidence for only one tectonically capable fault or feature within the 200-mi radius, the Meers fault. Given the low seismic hazard associated with the majority of features within the 200-mi radius, 3 features outside of this radius are discussed which contribute to the hazard at the site: the Rio Grande Rift, the Cheraw fault, and the New Madrid seismic zone.

2.5.1.1.4.3.1 Late Proterozoic Tectonic Features

The oldest outcropping rocks in Texas occur in part in the Llano Uplift in south-central Texas ((Figures 2.5.1-202 and 2.5.1-207), 90 mi south-southwest of the site. Ultramafic to amphibolitic metamorphic rocks and plutons record Mesoproterozoic high-grade metamorphism and deformation as part of the Grenville orogeny (References 2.5-247 and 2.5-248). This deformation primarily comprises broad folds and thrusts within the metamorphic units and resulted from a north-directed collision of a continental block with the southern margin of North America during the formation of Rodinia, likely between ~1300 and 1080 Ma (References 2.5-228 and 2.5-248). The Mesoproterozoic rocks are surrounded by Cambrian-Mississippian marine strata that were deposited during the Early Paleozoic rifting and ocean development that preceded the Late Paleozoic Ouachita orogeny (Reference 2.5-249). The current map pattern of the Llano Uplift is dominated by northeast-trending exposures of normal to oblique faults that have Late Paleozoic ages (Reference 2.5-249). These faults originated during the Ouachita orogeny and exhumed the Llano basement rocks to temperatures of less than 120 °C in the Late Permian (Reference 2.5-250). This thermal history indicates that the Llano Uplift experienced little uplift since the Permian. The Mesoproterozoic basement and Paleozoic marine strata are then overlain by nearly flat-lying Lower Cretaceous shallow marine deposits that also limit the deformation in the Llano Uplift to pre-Cretaceous (Reference 2.5-249).

2.5.1.1.4.3.2 Early Paleozoic Tectonic Features

There are few exposures of faults that accommodated the Cambrian rifting of Laurentia. The most abundant evidence for this extension is recorded by the sedimentary sequences deposited during and after extension--the Southern Oklahoma Aulacogen, located 100 mi north of the site (Figure 2.5.1-208). Normal faults and fault-bounded basins associated with Late Proterozoic to Early Paleozoic rifting of Laurentia are inferred from geophysical surveys to lie beneath overthrust rocks of the Late Paleozoic Ouachita orogenic belt and Mesozoic to Tertiary Gulf Coastal Plain strata (References 2.5-204 and 2.5-228), but these structures are not exposed in central Texas, and are not well documented in peer-reviewed geologic literature. Additionally, many of the faults associated with Precambrian to Cambrian rifting of the Southern Oklahoma Aulacogen were reactivated during the Late Paleozoic compression as thrusts bounding the Amarillo, Wichita, and Arbuckle uplifts.

The southern boundary of the Anadarko Basin is an uplifted zone of Precambrian basement and overlying Cambrian to Permian sedimentary units that were deposited during Early Paleozoic rifting and later subsidence (Figure 2.5.1-207). Subsequently, this composite structure was dismembered into three pieces during Late Paleozoic Ouachita deformation: the western Amarillo Uplift in the Texas Panhandle (230 mi from site), the central Wichita Uplift in southwestern Oklahoma (180 mi from the site), and the eastern Arbuckle Uplift (155 mi from the site) in southeastern Oklahoma. The northwest-southeast-trending uplifts are bound by steeply dipping faults. The basement within these uplifts was uplifted relative to

the Anadarko Basin to the northeast and the Whittenburg Trough to the southwest during the Pennsylvanian and Early Permian (Reference 2.5-251). However, in the Texas Panhandle (320 mi from CPNPP), a small offset of Triassic Dockum group across the steep faults that bound the westernmost Amarillo Uplift indicate that some post-Paleozoic reactivation has occurred in conjunction with the Rio Grande Rift, probably during the deposition of the Tertiary Ogallala Formation (Reference 2.5-251). The Wichita Uplift exposes Precambrian to Cambrian metamorphic and igneous basement rocks. Permian unconformities and the presence of clasts of Cambrian units in the Permian Post Oak Conglomerate indicate that these rocks were probably uplifted in the Late Paleozoic (Reference 2.5-252). Additionally, the thermal history of rocks in the Wichita Uplift suggests that there has been some post-Permian heating, potentially from loading of Laramide-derived sediments from the north and west (Reference 2.5-252). Furthermore, denudation of 1 to 3 km of material from the southern mid-continent region occurred since the mid-Cenozoic (Reference 2.5-252). The timing of the Arbuckle Uplift, using sediments from the adjacent basins, suggests it was last active in Late Pennsylvanian to Permian time, as it is covered with undeformed Permian to Cretaceous rocks (Reference 2.5-228 and 2.5-253). In summary, the Early Paleozoic rift-related activity associated with the Southern Oklahoma Aulacogen was later overprinted as the region was reactivated in a series of contractional uplifts from southeastern Oklahoma to the Texas Panhandle. The thrusts bounding the Wichita and Arbuckle uplifts were active into the Permian.

2.5.1.1.4.3.3 Late Paleozoic Tectonic Features

The site region includes portions of the Ouachita Fold and Thrust Belt within 33 mi of the site (Figure 2.5.1-207). The Carboniferous collision of Laurentia and Gondwana led to the development of a mostly northwest-vergent fold and thrust system exposed in central and eastern Texas, southern Oklahoma, and central Arkansas. Aside from exposures such as the Marathon Uplift in west Texas and the Ouachita Mountains of Arkansas and Oklahoma, these thrusts and Paleozoic preorogenic to synorogenic rocks are largely buried beneath Cretaceous and younger postorogenic strata (Reference 2.5-212). Structures in the region of Arkansas and Oklahoma that were a part of this Late Paleozoic thrusting include the Windingstair, Ti Valley, Boktukola, and Octavia faults; the Hochatown dome; the Carter Mountain and Cross Mountain anticlinoriums; and the Linson Creek synclinorium (Reference 2.5-254). The undisturbed Cretaceous rocks unconformably overlying the Carboniferous to Permian synorogenic sediments support the notion that the thrusts, structures, and uplifts associated with the Ouachita orogeny had no activity after the Permian throughout the CPNPP site region.

The central core of the Ouachita Mountains is composed of the Benton and Broken Bow uplifts, which expose Cambrian to Mississippian deformed strata (Reference 2.5-254). These deformed rocks were once thought to be North American basement, however it is now recognized that these rocks are allochthonous and represent the northward-translated deformed accretionary wedge from the south-dipping subduction zone (Reference 2.5-255).

A series of foreland basins, including the Fort Worth, Ardmore/Marietta, and Arkoma basins, are oriented sub-parallel to the trend of the thrust belt and lie northwest of the trace of the Ouachita thrusts. The basins are filled with Late Paleozoic synorogenic sediments like the Atoka Group series, and preserve the record of Ouachita deformation (Reference 2.5-204). These foreland basins are buried by strata of the Gulf Coastal Plain, and are known from subsurface data gathered during oil and gas exploration (References 2.5-204 and 2.5-256). The basins primarily formed by flexural loading of the crust as the Ouachita orogen developed structural and topographic relief. Geophysical data from other parts of the Ouachita foreland indicate that these basins typically subsided along down-to-the-south normal faults, which in some cases were overthrust by the frontal thrust sheets during the final stages of the Ouachita orogeny (References 2.5-204 and 2.5-256). Stratigraphic data indicate that these basins generally formed during the deposition of the Atoka to Desmoinesian (Reference 2.5-204). The Bend Arch, which forms the western margin of the Fort Worth Basin, is the hingeline that accommodated the downward flexure of the Fort Worth Basin during the Ouachita orogeny (Pennsylvanian) (Reference 2.5-203).

2.5.1.1.4.3.4 Mesozoic Tectonic Features

Mesozoic tectonic structures within the site region are generally confined to the Gulf Coastal Plain, a broad homocline comprising gently gulfward-dipping Mesozoic and Cenozoic strata. The disturbances to this plain are either broad, regional arches and embayments or normal fault systems. These two types of Mesozoic deformation features are described separately below.

2.5.1.1.4.3.4.1 Arches and Embayments

The Gulf Coastal Plain, in the southeastern portion of the site region, includes a series of Mesozoic, gulf-perpendicular, regional arches and basins. These features, such as the San Marcos Arch, the East Texas Basin, and the Sabine Uplift are discussed below. Stratigraphic evidence indicates that the relief on these features had diminished during the Eocene (Reference 2.5-257).

The Sabine Uplift is a broad, basement-cored north-trending anticline centered on the Texas-Louisiana border, 184 mi east of the site (References 2.5-258 and 2.5-259). On geologic maps, the Sabine Uplift appears as a circular outcrop of Eocene Wilcox Group surrounded by younger Claiborne Group (Figure 2.5.1-202a, Figure 2.5.1-202b, Reference 2.5-257). The change in thickness of Middle Cretaceous strata over the area indicate that about 550 ft of relief existed during this time and probably began around 100 Ma. This uplift region was later submerged and not present at the time of the deposition of the regional Austin chalk at 90.5 Ma (Reference 2.5-257). The 650 ft of uplift which allows for the current map pattern probably occurred between 58 and 46 Ma (Early Eocene) (Reference 2.5-257). Similarly, the San Marcos Arch, a gently southeast-plunging fold that extends southeast from the Llano Uplift, developed in Late Cretaceous time (References 2.5-228 and 2.5-260). The San Marcos Arch extends for over 250 mi from the Rio Grande Embayment to the East Texas Basin, and cuts across

the structural trend of the Ouachita belt south of the site (Reference 2.5-261). The San Marcos Basin formed during the Early Mesozoic rifting that eventually led to the formation of the Gulf of Mexico (Reference 2.5-228). Similarly, the East Texas Basin is filled with dominantly Mesozoic (Upper Triassic Eagle Mills Formation) to Tertiary (Eocene Claiborne) sediments, sometimes deformed by the movement of the Jurassic Louann salt. This unit is responsible for several salt domes in the region (Reference 2.5-262).

The formation of the series of arches and basin along the Texas Gulf Coastal Plain was likely caused by the combination of Mesozoic rifting of Pangea and the Late Cretaceous Laramide orogeny. Mesozoic rifting and extension may have caused gulf-perpendicular trends in basement thickness that allowed for variable subsidence among crustal blocks, and differing amounts of sediment accommodation space along the Gulf Coastal Plain (Reference 2.5-230). Laubach and Jackson (Reference 2.5-260), Ziegler, et al. (Reference 2.5-263), and Ewing (Reference 2.5-228) proposed that the arches are genetically related to east-west compressive stresses during the Late Cretaceous to Early Tertiary Laramide orogeny to the north and west along the western edge of paleo-North America.

2.5.1.1.4.3.4.2 Normal Fault Systems

Several Mesozoic fault systems in the Gulf of Mexico region are related to bodies of Jurassic salt at depth. These fault systems include the Luling-Mexia-Talco and the Mt. Enterprise-Elkhart graben system (Figures 2.5.1-207and 2.5.1-210). In general, these fault systems lie updip of salt pinchouts or welds, and motion on these faults is related to salt migration, with the exception of some Luling faults, which are not clearly salt-related (Reference 2.5-264 and 2.5-265). The youngest rocks these faults displace are Eocene in age.

The Luling-Mexia-Talco fault zone describes a series of normal faults located ~50 mi east of the CPNPP site. This fault system is parallel to and developed above the Pennsylvanian Ouachita thrust belt, and comprises the northern and western margins of the East Texas Basin (Reference 2.5-262). The Luling fault zone is southeast of the Balcones fault zone and comprises a series of north-side-down normal faults including the Staples, Larremore, Lytton Springs, Luling, Darst Creek, Salt Flat, Somerset, and Alta Vista faults (Reference 2.5-266). The Mexia fault zone is over 500 mi long (Reference 2.5-266). The Mexia-Talco fault zone is a graben system coincident with the updip extent of subsurface Middle Jurassic Louann salt, and was active from the Jurassic to Eocene based upon the ages of the oldest and youngest strata offset by this fault system (Reference 2.5-262).

The Mt. Enterprise-Elkhart graben fault system is a zone of normal faults that obliquely crosses the southeastern margin of the East Texas Basin and extends eastward to the western flank of the Sabine Uplift (References 2.5-228 and 2.5-262). The Mt. Enterprise-Elkhart graben fault system strikes east-northeast and extends for a total distance of 90 mi from south of Carthage, Texas, to the Trinity River near Palestine, Texas (Figure 2.5.1-210). At its closest point, the Mt. Enterprise-Elkhart graben fault system is about 129 mi southeast of the site. Like

the Luling-Mexia-Talco fault zone, the Mt. Enterprise-Elkhart graben fault system is characterized by a structurally complex series of grabens that are interpreted to root in Middle Jurassic Louann salt (References 2.5-262 and 2.5-267). The Mt. Enterprise-Elkhart graben faults were primarily active in Late Jurassic-Early Cretaceous time and the youngest rocks they offset are Eocene in age (References 2.5-228 and 2.5-262). No data have been published to support an interpretation that the Mt. Enterprise-Elkhart graben fault system is a capable tectonic structure. It should be noted, however, that the CPNPP FSAR for Units 1 and 2 described the most recent movement to be Eocene or younger on the Mt. Enterprise-Elkhart graben fault system. In publications that predate the 1986 EPRI studies, lines of evidence indicating potential Quaternary motion and active creep along the Mt. Enterprise-Elkhart graben fault system include the following:

- Three faults at the western end of the Mt. Enterprise-Elkhart graben fault system in the Trinity River Valley near Palestine, Texas, displace Late Quaternary deposits overlying Eocene Claiborne strata (Reference 2.5-268; Figure 2.5.1-210). Maximum normal displacement of the Eocene strata on the fault at this site is 46 inches (in), with maximum offset of the overlying Quaternary gravels of 26 in. On the basis of an estimated age of 37 thousand years (ka) for the Late Quaternary gravels (Reference 2.5-258), the implied average, Late Quaternary separation rate across the fault is about 0.02 mm/yr.

- Geodetic leveling data showing a relative movement of 130 mm across the geographic center of the Mt. Enterprise-Elkhart graben fault system between 1920 and the mid 1950s, with a down-to-the-south displacement across the southern margin (Reference 2.5-268). If this motion is due to slip on normal faults, then the average vertical separation rate is 4.3 mm/yr.

- Historical and instrumentally located seismicity is spatially associated with the Mt. Enterprise-Elkhart graben fault system, including the 1891 Rusk earthquake (maximum Modified Mercalli Intensity [MMI] VI: magnitude (unspecified scale) 4.0 and location estimated from felt effects); four earthquakes in 1957 (maximum intensity III to V; magnitudes (unspecified scale) 3.0 to 4.7 and locations estimated from felt effects); and the 1981 Center (m_b 3.0) and Jacksonville (m_b 3.2) earthquakes (References 2.5-269 and 2.5-270). Locations and estimated magnitudes are further discussed in Subsection 2.5.2.1.

As discussed in Subsection 2.5.1.1.4.3.3, seismic reflection data suggest that the Mt. Enterprise-Elkhart graben fault system is rooted in the Jurassic Louann salt at maximum depths of 4.5 to 6 km (References 2.5-262 and 2.5-267). This suggests that movement of salt at depth may drive observed Late Quaternary displacement and contemporary creep across the Mt. Enterprise-Elkhart graben fault system and that the fault is not accommodating tectonic deformation and thus is not an independent source of moderate to large earthquakes. Presumably, this was the evaluation of the EPRI ESTs, who had access to the pre-1986 literature on the Mt.

Enterprise-Elkhart graben fault system and did not specifically characterize it as a Quaternary tectonic fault and potentially capable structure. However, Ewing (Reference 2.5-228) commented in a post-EPRI publication that ". . . surface strata are displaced and seismicity suggests continuing deformation. . ." on the Mt. Enterprise-Elkhart graben fault system.

On the basis of a review of post-EPRI scientific literature, no new data have been published to support an interpretation that the Mt. Enterprise-Elkhart graben fault system is a capable tectonic structure. Recent reviews of suspected Quaternary tectonic features in the CEUS by Crone and Wheeler (Reference 2.5-271) and Wheeler (Reference 2.5-272) did not identify or discuss the Mt. Enterprise-Elkhart graben fault system as a potential tectonic fault. Because of the unverified statement that the western end of the Mt. Enterprise-Elkhart graben fault system could potentially be seismogenic (Reference 2.5-268), William Lettis & Associates, Inc., conducted a field reconnaissance study. This study did not find evidence to support post-Eocene tectonic activity on the Mt. Enterprise-Elkhart graben fault system. The documented association of the Mt. Enterprise-Elkhart graben fault system with Jurassic salt deposits and the high rate of active creep measured by geodetic methods both support the interpretation that Quaternary activity of the Mt. Enterprise-Elkhart graben fault system is related to salt migration at depth. The separation rate of 4.3 mm/yr implied by the geodetic data is highly anomalous for a fault located in a stable continental block; if tectonic, deformation rates and fault slip rates of about 4 to 5 mm/yr are more characteristic of those associated with an active plate boundary. There is broad consensus within the informed geosciences community that the Grand Prairie of Texas is not part of an active plate boundary. The high geodetic deformation rates, if accurate, are most simply explained by movement at depth and do not reflect whole-crustal strain. In conclusion, there is no new information bearing on the Quaternary activity of the Mt. Enterprise-Elkhart graben fault system faults requiring a revision of the EPRI seismic source characterization of this region.

2.5.1.1.4.3.5 Tertiary Tectonic Features

South and east of the Llano Uplift, the Balcones fault zone is a series of faults that generally strike north to northeast and dip 45 to 85° southeast, with down-dip fault striae indicating normal sense of displacement (References 2.5-266 and 2.5-273). The fault zone is approximately 75 mi east of the site and the throw on the faults varies from 500 to 1200 ft (Figure 2.5.1-210, Reference 2.5-266). The fault zone has resulted in a series of fault-line scarps between Uvaled and Georgetown, Texas, known as the Balcones Escarpment (Reference 2.5-273). The Balcones fault zone includes multiple fault blocks (2 to 7 mi wide) bounded by the en échelon normal faults, each with 100 to 850 ft throws, northwest-dipping antithetic faults, and relay ramps between the en échelon fault (Reference 2.5-273).

Initial movement on the Balcones fault zone may have occurred in the Mesozoic, because Late Cretaceous volcanic rocks of the Balcones igneous province generally are exposed along the trend of the fault zones, and in some cases

volcanic centers are aligned along the faults (Reference 2.5-228). The youngest rocks cut by the faults are Eocene, though a lack of Oligocene to Miocene deposits adjacent to the structure has been interpreted as evidence for post-Eocene movement (Reference 2.5-266). Collins (Reference 2.5-274) stated that most of the displacement on the Balcones fault zone occurred in Late Oligocene and Early Miocene, but did not provide a basis for this assessment.

The fault zone is associated with the southeast-facing Balcones Escarpment, a prominent geomorphic feature in central Texas (Reference 2.5-275). The Balcones Escarpment is a fault-line scarp produced by differential erosion of these units (Reference 2.5-276). Rocks exposed on the upthrown side of the fault zone are dominantly Lower Cretaceous, erosion-resistant carbonates, whereas strata on the downthrown side are less resistant, Upper Cretaceous chalk and mudstone.

2.5.1.1.4.3.6 Quaternary Tectonic Features Within the Site Region

The site region is part of a tectonically stable continental margin. No capable tectonic faults were identified within the CPNPP Units 1 and 2 site region during the 1986 EPRI studies, and the CPNPP FSAR for Units 1 and 2 concluded that there were no capable tectonic faults within the site region. The Great Plains region, in general, and the CPNPP Units 3 and 4 site region, in particular, is characterized by very low rates of background seismicity (Subsection 2.5.2.1). Within the site region, only the Meers fault has been identified as demonstrating evidence for Quaternary activity. However, a nearby fault bounding a Late Paleozoic Uplift, the Criner fault, has been speculated to have Quaternary activity because of its proximity to the Meers. Therefore, we also discuss this feature, concluded to be a Late Paleozoic thrust with a fault-line scarp, in detail below.

2.5.1.1.4.3.6.1 Meers Fault

Quaternary activity on the Meers fault was not recognized until the early 1980s (References 2.5-277 and 2.5-278) after completion of the FSAR for CPNPP Units 1 and 2. Following from these studies, the Meers fault is the only capable fault within the CPNPP Units 3 and 4 site region, and the Meers fault should be characterized as a seismic source for CPNPP Units 3 and 4. The seismic source characterization of the Meers fault used for CPNPP Units 3 and 4 is presented in Subsection 2.5.2.4.2.3.2.

This source characterization is developed following the Senior Seismic Hazard Analysis Committee (SSHAC) guidelines for a SSHAC level 2 study described in NUREG/CR-6372. Following the guidance of NUREG/CR-6372, this characterization of the Meers fault represents the legitimate range of technically supportable interpretations of the seismic capability of the Meers fault among the informed technical community. This characterization is based on the existing literature of the Meers fault and on the elicitation of expert opinion. A summary of these opinions and a review of the existing literature used in the Meers source characterization is reviewed below.

The Meers fault is the southern boundary of the Frontal Wichita fault system in southern Oklahoma and is approximately 180 mi from the site (Figures 2.5.1-207 and 2.5.1-210). The history of the Meers fault, like the majority of the Frontal Wichita fault system, largely reflects the history of rifting and orogenesis in southern Oklahoma (see discussion in Subsection 2.5.1.1.4.1.1, Southern Oklahoma Aulacogen and Wichita uplift). The Meers fault may have initiated as a rift-bounding normal fault during the formation of the Southern Oklahoma Aulacogen (Reference 2.5-223). During the Permian, the Meers fault accommodated some contraction associated with the closing of the Atlantic Ocean and the Ouachita orogeny that led to the formation of the Wichita Uplift (References 2.5-223, 2.5-279, 2.5-280, 2.5-281 and 2.5-282). Slip on the Meers fault during this time was characterized by up-to-the-north motion on a southward dipping fault with an unknown component of left-lateral slip. Ultimately approximately 7.5 mi of vertical offset is thought to have occurred across the Frontal Wichita system, and roughly 1.2 mi was accommodated on the Meers fault (References 2.5-223, 2.5-280, and 2.5-281).

Since formation of the Wichita Uplift, the Meers fault has been reactivated at least twice: first during the Late Permian, and most recently during the Late Holocene. During the known reactivations, the sense of vertical slip on the Meers fault reversed from north-side-down to south-side-down. The change in sense of slip during the Permian reactivation was determined from observations of sedimentary material eroded off of the northern, upthrown side of the fault and deposited on the southern, downthrown side of the fault (References 2.5-281 and 2.5-282). The second known reactivation of the Meers fault began sometime in the Quaternary with the most recent slip in the Late Holocene (References 2.5-223, 2.5-281, 2.5-283, 2.5-284 and 2.5-285).

The trace of the Meers fault is easily identified on aerial photographs for a total distance of approximately 23 mi as a south-down topographic escarpment (Figure 2.5.1-211). The scarp over much of this extent has been visited by various researchers, and is thought to be related to Holocene rupture along the Meers fault (References 2.5-277, 2.5-278, 2.5-281, 2.5-284, 2.5-285 and 2.5-286).

2.5.1.1.4.3.6.1.1 Existing Literature

The modern state of knowledge regarding the Quaternary activity of the Meers fault is primarily based on four sets of studies: the studies of Ramelli and others (References 2.5-271 and 2.5-286); the studies of Madole (References 2.5-283 and 2.5-287); the study of Crone and Luza (Reference 2.5-284); and the studies of Swan and others (References 2.5-277 and 2.5-285). These studies are summarized in Table 2.5.1-202. Other studies of the Meers fault have been conducted (References 2.5-223, 2.5-288 and 2.5-289), but these studies do not significantly add to the modern state of knowledge of the Meers fault as a potential seismic source. The seismic source characterization of the Meers fault developed in Subsection 2.5.2.4.2.3.2 is based on the first four studies. A summary of each of these four Meers fault studies is presented in Table 2.5.1-202.

2.5.1.1.4.3.6.1.1.1 Meers Fault Studies of Ramelli and Others

The major contribution of the studies of Ramelli and others (References 2.5-281 and 2.5-286) to investigations of the Quaternary activity of the Meers fault was to acquire and analyze low-sun-angle aerial photography and to extend the mapped length of the Meers fault an additional 6.8 mi (11 km) to the southwest, for a total length of approximately 23 mi (37 km) (Figure 2.5.1-211).

The southeast extension of the scarp identified by Ramelli, et al. (Reference 2.5-286) and further discussed by Ramelli and Slemmons (Reference 2.5-281) is described as more subtle and discontinuous than the originally identified 16-mi-long scarp. Ramelli and Slemmons (Reference 2.5-281) argue that the southeastern continuation of the scarp shares the same history of events on the Meers fault due to its alignment with the original scarp, the consistent down-to-south separation across the scarp, its proximity to the original scarp, and the presence of a small drainage aligned parallel to the scarp and across the pattern of local drainage networks. However, Ramelli and Slemmons (Reference 2.5-281) also acknowledge uncertainty in the structural relationship between the northwest and southeast scarps due to a left step in the scarp near the junction of the two scrap strands and the absence of a scarp across East Cache Creek (Figure 2.5.1-211). In addition, field evaluation of the southeast extension of the scarp has not been possible because the scarp traverses onto the Fort Sill Military Reservation (References 2.5-278, 2.5-281 and 2.5-286).

The studies of Ramelli, et al. (Reference 2.5-286) and Ramelli and Slemmons (Reference 2.5-281) also discuss:

- Estimates of vertical separation and left-lateral offsets across the fault (16 ft and 33 to 66 ft, respectively);
- Magnitude estimates of earthquakes that caused the scarp (Ms 6.75 to 7.25);
- Fault dip (sub-vertical to vertical); and
- Dating of the last surface-rupturing event (within the last several thousand years).

All of these characteristics of the fault are more thoroughly investigated in studies that post-date the work of Ramelli, et al. (Reference 2.5-286) and Ramelli and Slemmons (Reference 2.5-281).

2.5.1.1.4.3.6.1.1.2 Meers Fault Studies of Madole

The studies of Madole (References 2.5-283 and 2.5-287) used radiocarbon dating of organic material to constrain the timing of well-defined movement along the Meers fault. Madole mapped alluvial stratigraphy at two sites along the fault (Canyon Creek and Browns Creek; Figure 2.5.1-211) and used radiocarbon dates

determined from deposits distal to the fault trace (tens to hundreds of ft from downthrown side of the fault) to estimate depositional ages of the pre-faulting, fault-related, and post-faulting alluvial units.

The two key sedimentary units used by Madole's study are the Browns Creek alluvium, the youngest faulted unit, and the East Cache alluvium, the oldest unfaulted unit. Madole reported three ages for the Browns Creek alluvium taken from clay and humus layers (9,880 ± 160 and 12,240 ± 240 C-14 years B.P.) and snail shells (13,670 ± 120 C-14 years B.P.) (Table 2.5.1-201) that suggest deposition of the unit began around 13,000 B.P. in C-14 years. Madole presented four ages from the East Cache Creek Alluvium taken from clay and humus layers (310 ± 150 and 470 ± 150 C-14 years B.P.) and charcoal fragments (70 ± 150 and 600 ± 50 C-14 years B.P.). Madole concluded that the unit was deposited after 800 years B.P. and before 100 years B.P.

Madole (Reference 2.5-283) also constrained the age of deposition of alluvial fans derived from the Meers fault scarp. At the Canyon Creek site, Madole reported a C-14 age (1,280 ± 140 years B.P.) (Table 2.5.1-202) from charcoal buried by scarp-derived alluvial-fan deposits that he interprets as providing a maximum age of faulting. Madole (Reference 2.5-283) concluded that this date combined with the C-14 ages of the East Cache Creek alluvium at this site (600 ± 50 years B.P.) bounds the age of faulting at Canyon Creek. At the Browns Creek site, Madole reported two C-14 ages (1,740 ± 140 years B.P. and 1,360 ± 100 years B.P.) (Table 2.5.1-201) from a clay and humus layer buried by the fault-related fan. Again, Madole (Reference 2.5-283) concluded that these ages combined with the C-14 ages of the East Cache Creek alluvium at this site (70 ± 150 years B.P., 310 ± 150 years B.P., and 470 ± 150 years) constrains the age of faulting at Browns Creek. Overall, Madole (Reference 2.5-283) concluded that the 1,280 ± 140 C-14 years age from the Canyon Creek site is the best estimate for the time of faulting.

2.5.1.1.4.3.6.1.1.3 Meers Fault Studies of Crone and Luza

Crone and Luza (Reference 2.5-284) and Luza, et al. (Reference 2.5-290) completed two fault-perpendicular trenches at the Canyon Creek site, four fault-parallel trenches at the ponded alluvium site, and several excavations of the fault scarp near the ponded alluvium site to investigate the paleoseismic history of the Meers fault (Figure 2.5.1-211). The ponded alluvium site was used to estimate the ratio of lateral to vertical offset along the fault, and the excavations were used to estimate the number of Holocene events. Here we discuss their results as presented in the Crone and Luza (Reference 2.5-284) publication. The best constraints on the timing of faulting came from the Canyon Creek trenches.

<u>Canyon Creek Site</u>

Trench 1 of Crone and Luza (Reference 2.5-284) was excavated across the fault in the Holocene Browns Creek alluvium. In the trench, Browns Creek alluvium overlies Permian Henessey Shale. Crone and Luza (Reference 2.5-284) also

suggested that the stratigraphic relations show evidence of only one surface-faulting event.

Crone and Luza (Reference 2.5-284) used two radiocarbon dates to constrain the timing of faulting in the trench. One age (1570 cal. years B.P.) (Table 2.5.1-201) was taken from soil humus interpreted to have fallen into a crevice caused by surface faulting. The second age (1646 cal. years B.P.) (Table 2.5.1-201) was taken from soil humus deposited at the base of the scarp shortly after faulting. Crone and Luza (Reference 2.5-284) interpreted these ages as maximum ages for faulting because they are determined using soil humus likely to have long-lived organic components that predate soil deposition. Crone and Luza (Reference 2.5-284) corrected for this long-lived C-14 component of the soil by subtracting 300 years (their estimate of the Average Mean Residence Time, AMRT) from the calibrated radiocarbon ages to give estimates of scarp formation. As such, they interpreted the samples from this trench to indicate scarp formation between 1,200 to 1,300 years B.P.

Trench 2 of Crone and Luza (Reference 2.5-284) was excavated across the scarp in Middle Pleistocene Porter Hill alluvium. Overlying the Porter Hill alluvium, on the downthrown side, were scarp-derived deposits. The Hennessey Shale bedrock was only encountered on the upthrown side of the fault. A stratigraphic offset of 10 to 11 ft was observed in the trench, and Crone and Luza (Reference 2.5-284) interpreted stratigraphic relationships within the trench as providing evidence of only one episode of faulting. Crone and Luza (Reference 2.5-284) also indicated that the amount of offset observed in the Porter Hill alluvium in the trench is roughly equivalent to the offset observed in the younger Browns Creek alluvium in trench 1. Crone and Luza (Reference 2.5-284) interpreted this observation as indicating that there has not been any substantial vertical movement other than that observed in trench 1 since deposition of the Porter Hill Alluvium.

Crone and Luza (Reference 2.5-284) used one radiocarbon date to constrain the timing of faulting in trench 2. An age of 1290 calendar years B.P. was determined for a soil 7 to 10 ft (2 to 3 m) downslope of the scarp that was buried by scarp-derived colluvium (Table 2.5.1-201). Given this stratigraphic relation and the distance from the scarp, Crone and Luza (Reference 2.5-284) interpreted this as a minimum age for the time of faulting.

<u>Ponded Alluvium Site</u>

The ponded alluvium site of Crone and Luza (Reference 2.5-284, Figure 2.5.1-211) consists of a southwest-facing fault scarp cutting across three northeast-draining gullies. Alluvium from the gullies ponded against the scarp, creating a well preserved history of Holocene faulting. At the site, Crone and Luza (Reference 2.5-284) excavated two pairs of fault-parallel trenches and several soil pits across the scarp.

Each pair of the fault-parallel trenches was excavated with one trench on each side of the fault, and each pair exposed a bedrock paleo-channel and a stratigraphy consisting of ponded alluvium and fault-related alluvium and colluvium. Crone and Luza (Reference 2.5-284) obtained radiocarbon ages from three different horizons at the easternmost set of trenches: (1) a non-fault related, pebbly silt alluvial deposit (1816 cal. years) from near the base of the trench, (2) a silt layer interpreted as a ponded alluvium deposited immediately after the initial scarp formation (1539 cal. years), and (3) a sample 12 in stratigraphically higher in a similar silt layer (1354 cal. years) (Table 2.5.1-201). Crone and Luza (Reference 2.5-284) report that after correcting 300 years for the AMRT, the oldest two ages bracket formation of the scarp. At the western set of trenches, Crone and Luza (Reference 2.5-284) determined only one radiocarbon age (1606 cal. years), and they believe the sample contained pre- and post-faulting organic material. As such, they do not believe the age provides a reliable constraint on scarp formation.

Crone and Luza (Reference 2.5-284) also used the bedrock gullies in the two pairs of trenches to estimate the amount of lateral and vertical offset across the Meers fault. For each pair of trenches, they determined the position of the channel thalweg and estimated the offset accounting for channel gradient and potential variations in channel orientation. In the westernmost pair of trenches, they estimated a vertical separation of 4.9 ft and a left-lateral separation of 16 ft. In the easternmost pair of trenches, they estimated a vertical separation of at least 6.9 ft and left lateral separation of 11 to 17 ft.

Crone and Luza (Reference 2.5-284) also excavated several soil pits across the fault scarp to constrain the number of scarp-forming faulting events. Crone and Luza (Reference 2.5-284) reported that stratigraphic relations within the pits provide evidence of one to two faulting events. However, Crone and Luza (Reference 2.5-284) preferred the single event interpretation due to the evidence in the Canyon Creek trenches of only one event.

Summary

In summarizing their results, Crone and Luza (Reference 2.5-284) stated their best estimate for the age of the Meers fault scarp as 1200 to 1300 years B.P. They also estimated that the magnitude of the event that caused the scarp was approximately Ms 7 or Mw greater than 7.

2.5.1.1.4.3.6.1.1.4 Meers Fault Studies of Swan and Others

Geomatrix Consultants undertook a detailed study of the Meers fault in the late 1980s funded by the U. S. Nuclear Regulatory Commission (Reference 2.5-291). The study resulted in two reports: a contribution to a proceedings volume for a NRC meeting (Reference 2.5-277), and a draft report to the NRC (Reference 2.5-285). These reports present the same material with the draft report providing the greatest level of detail.

The Swan, et al. (Reference 2.5-285) study consisted of numerous trenches, soil pits, hand auger samples, surveys of offset features, and over 30 radiocarbon dates (all of which were converted to calibrated ages and deemed not needing an AMRT correction). With respect to the Meers fault, the study focused on four sites (Figure 2.5.1-211): the valley site, the NW ponded alluvium site, the SE ponded alluvium site (the same location as the ponded alluvium site of Crone and Luza (Reference 2.5-284), and the Canyon Creek site (the same location as the Canyon Creek site of Crone and Luza (Reference 2.5-284) and Madole (Reference 2.5-283).

Valley Site

The valley site of Swan, et al. (Reference 2.5-285; Figure 2.5.1-211) is characterized by a 4.9-ft-high scarp in Holocene valley fill deposits. At this site Swan, et al. (Reference 2.5-285) reported that the Browns Creek alluvium is faulted, the scarp-derived colluvium is faulted, and the stratigraphically highest alluvium is unfaulted. They interpreted these observations as documenting two surface-rupturing events: (1) an older event that faulted the Browns Creek alluvium, formed a scarp, and created the scarp-derived colluvium; and (2) a younger event that faulted the initial scarp-derived colluvium. Swan, et al. (Reference 2.5-285) presented calibrated radiocarbon dates that constrain event timing as follows (Table 2.5.1-201) (Figure 2.5.1-212):

- An age of 2918 years B.P. (sample PITT-0373) from the uppermost section of the Browns Creek alluvium is a maximum age for the oldest event;
- Two ages from the base and middle of the scarp-derived colluvium (1942 and 1610 years B.P. for samples PITT-0370 and PITT-0369) provide minimum ages for the oldest event and maximum ages for the youngest event; and
- Four ages from the post-faulting colluvium and alluvium (1296, 1296, 777, and 777 years BP for samples PITT-0372, PITT-0375, PITT-0368, and AA-4093, respectively) constrain the minimum age of the youngest event.

Swan, et al. (Reference 2.5-285) measured a stratigraphic separation of 12 $\pm$ 2 ft (3.6 $\pm$ 0.6 m) associated with the fault. Lateral offset at the site was not as well constrained, but Swan, et al. (Reference 2.5-285) estimated an approximate left-lateral offset of 30 $\pm$ 7 ft (9 $\pm$ 2 m).

NW Ponded Alluvium Site

The NW ponded alluvium site of Swan, et al. (Reference 2.5-285; Figure 2.5.1-211) is characterized by Holocene alluvial and colluvial sediments ponded behind the Meers scarp. At this site, Swan, et al. (Reference 2.5-285) excavated seven trenches and found Late Quaternary deposits from a paleo-channel overlying Post Oak Conglomerate.

From oldest to youngest Swan, et al. (Reference 2.5-285) reported the two stratigraphically highest units as: (1) faulted colluvium; and (2) unfaulted ponded alluvial deposits and colluvium. Swan, et al. (Reference 2.5-285) concluded that these units document two surface rupturing events: (1) an older event that led to the formation of the deeper colluvium, and (2) a younger event that faulted the deeper colluvium and led to the deposition of the ponded alluvium and unfaulted colluvium. Swan, et al. (Reference 2.5-285) also presented calibrated radiocarbon dates from these units to constrain the timing of faulting as follows (Table 2.5.1-201) (Figure 2.5.1-212):

- An age of 1912 years B.P. (sample PITT-0378) from the middle of the deeper, faulted colluvium interpreted as a minimum age for the oldest faulting event;
- An age of 1484 years B.P. (sample PITT-0379) from the top of the deeper, faulted colluvium interpreted as a maximum age for the youngest faulting event; and
- Two ages from the base of the unfaulted ponded alluvium (1238 and 1265 years B.P. for samples PITT-0380 and PITT-0381) interpreted as minimum ages for the youngest faulting event.

The buried channel exposed in the trenches also provided Swan, et al. (Reference 2.5-285) with a channel thalweg with which they estimated fault offset. Their best estimates of lateral and vertical offset were 10 $\pm$ 3.3 ft of left-lateral offset and 7.9 $\pm$ 1 ft of vertical offset.

SE Ponded Alluvium Site

The SE ponded alluvium site of Swan, et al. (Reference 2.5-285) is at the same location as the ponded alluvium site of Crone and Luza (Reference 2.5-284; Figure 2.5.1-211). At the site, the stratigraphy of the site is equivalent to that at the NW ponded alluvium site: Post Oak Conglomerate bedrock is overlain by Late Quaternary alluvial and colluvial deposits (Reference 2.5-285).

Swan, et al. (Reference 2.5-285) reported the three stratigraphically highest units, from oldest to youngest, as: (1) a faulted, silty and clayey alluvium likely deposited in a paleo-channel that was cut by the fault, (2) faulted ponded alluvium and colluvium, and (3) a stratigraphically distinct, unfaulted second set of ponded alluvium and colluvium deposits. Swan, et al. (Reference 2.5-285) interpreted these relations as documenting two surface rupturing events: (1) an older event that cut the paleo-channel deposits and led to the formation of the deeper, ponded alluvium and fault-derived colluvium, and (2) a younger event that faulted the deeper, ponded alluvium and colluvium and led to the deposition of the stratigraphically higher ponded alluvium and fault-derived colluvium. Swan, et al. (Reference 2.5-285) also presented calibrated radiocarbon dates to constrain the event ages (Table 2.5.1-201) (Figure 2.5.1-212):

- Two ages of 6836 and 5943 calibrated years B.P. (samples PITT-0476 and PITT-0475, respectively) from the deep paleo-channel alluvium were interpreted as chronologically high maximum ages for the oldest event;
- An age of 3397 calibrated years B.P. (sample PITT-0477) from the middle of the deeper, faulted colluvium was interpreted as a maximum age for the oldest event;
- An age of 2093 calibrated years B.P. (sample PITT-0478) from the base of the deeper, faulted ponded alluvium was interpreted as a minimum age for the oldest event;
- An age of 1669 calibrated years B.P. (sample PITT-0479) from the middle of the deeper, faulted ponded alluvium was interpreted as a maximum age for the youngest event;
- Two ages of 1336 and 1167 calibrated years B.P. (samples PITT-0481 and PITT-0489, respectively) from the base of the unfaulted ponded alluvium were interpreted as minimum ages for the youngest event;
- An age of 1053 calibrated years B.P. (sample PITT-0480) from the middle of the unfaulted, colluvium was interpreted as a minimum age for the youngest event; and
- An age of 684 calibrated years B.P. (sample PITT-0482) from the middle of the unfaulted, ponded alluvium was interpreted as a minimum age for the youngest event.

The trenches also exposed two channel thalwegs that Swan, et al. (Reference 2.5-285) used to estimate fault displacement. Their best estimates of lateral and vertical offset from the thalwegs are 11 ± 3.3 ft of left-lateral offset and 8.9 m ± 3.3 ft of vertical offset for the upper thalweg and 12 ± 3.3 ft of left-lateral offset and 8.9 ± 2 ft of vertical offset for the lower thalweg. Finally, Swan, et al. (Reference 2.5-285) also conducted several surveys of ridge crest offset at the site and found that additional Quaternary events besides the two Holocene events are required to generate the observed ridge crest offsets.

<u>Canyon Creek Site</u>

Swan, et al. (Reference 2.5-285) visited the same Canyon Creek site as Crone and Luza (Reference 2.5-284, Figure 2.5.1-211) to survey the vertical separation of the Holocene Browns Creek alluvium and Pleistocene Porter Hill alluvium. Swan, et al. (Reference 2.5-285) conducted two scarp-perpendicular surveys of terrace elevations, nine test pits, and three hand-auger boreholes. They reported that the vertical separation at the contact of Browns Creek alluvium and bedrock is 17 ± 5.3 ft and that the vertical separation at the contact of Porter Hill alluvium and underlying bedrock is 17 ± 3.9 ft. Swan, et al. (Reference 2.5-285) interpreted the similarity in offset between the two bedrock contacts as an indication that there

has been no faulting between the deposition of the Porter Hill and Browns Creek alluvium. To temporally constrain this period of fault inactivity, Swan, et al. (Reference 2.5-285) correlated the soil development of the Porter Hill alluvium to a soil at a distant site that overlies a 560,000-year-old ash deposit. From this correlation Swan, et al. (Reference 2.5-285) estimated that the Porter Hill alluvium was deposited around 200,000 to 500,000 years ago and that this time period reflects the minimum period of inactivity between earthquake clusters. This time period is consistent with Madole's conclusion that the Porter Hill alluvium was deposited in the Middle Pleistocene (Reference 2.5-287).

2.5.1.1.4.3.6.1.2 Meers Fault Expert Opinions

As part of the SSHAC level 2 process (Reference 2.5-292), a group of experts was queried to further assess and document the range of opinions within the informed technical community with respect to the seismic characterization of the Meers fault. The experts consulted were:

- Keith Kelson, a Principal Geologist with William Lettis & Associates, Inc;
- Kathryn Hanson, a Principal Geologist with Geomatrix Consultants;
- Dr. Frank Swan, a Principal Geologist with Geomatrix Consultants;
- Dr. Anthony Crone, Senior Research Geologist with the USGS;
- Alan Ramelli, Research Engineering Geologist with the Nevada Bureau of Mines and Geology; and
- Dr. Ken Luza, Engineering Geologist, Oklahoma Geological Survey.

Each of the experts were asked the following questions:

- Is the Meers fault active?
- Can the Meers fault be adequately represented as a line source?
- What is your estimate of Mmax for the Meers fault?
- What approach would you use to estimate Mmax?
- What recurrence model would you use to parameterize the Meers fault?
- If clustered, is the fault currently in a cluster?
- What data would you use to determine the recurrence rate?

A summary of the responses from each expert is presented in Table 2.5.1-203. In general, the opinions of the experts are consistent with the published work summarized in Subsection 2.5.1.1.4.3.6.1.1.1.

In summary, the results of the SSHAC level-2 study were incorporated into a new seismic source characterization for the Meers fault. The seismic source characterization of the Meers fault used for CPNPP Units 3 and 4 is presented in Subsection 2.5.2.4.2.3.2.

2.5.1.1.4.3.6.2 Criner Fault

The Criner fault is exposed in southern Oklahoma and coincident with the northern edge of the Wichita Uplift. The Criner fault strikes N45°W and produces a southwest-facing escarpment, similar to the expression of the Meers fault about 120 km along strike to the northwest. The escarpment is 12 km long and 0.5 to 2 m high. Given the evidence for Quaternary activity along the Meers fault (References 2.5-283, 2.5-284, 2.5-286, and 2.5-293) workers have speculated that the Criner may also have been active in the Quaternary.

In 1986, none of the EPRI ESTs recognized the Criner fault as a structure with Quaternary earthquake hazard potential (Reference 2.5-369). Preliminary studies by Geomatrix Consultants, however, had indicated that the Criner fault displaces Middle to Late Pleistocene fluvial deposits adjacent to Hickory Creek in Love County (Reference 2.5-277). Following the submission to the NRC of the CPNPP Units 1 and 2 FSAR and communication with Geomatrix Consultants, a field party, including workers from EPRI, the NRC, Geomatrix Consultants, and the Oklahoma State Geological Survey, conducted a geological reconnaissance of the Criner fault in 1989 (NUREG-0797, CPNPP SSER 23, 1990). This study concluded that the escarpment was either (1) a fault-line scarp resulting from differential erosion of units juxtaposed along the fault in the Late Paleozoic or (2) a fault-line scarp with a 10- to 20-cm free face resulting from Late Quaternary displacement. The team was denied access to the key exposure along Hickory Creek and determined that there was insufficient evidence available to prove or disprove the capability of the Criner fault. Therefore, the team conservatively concluded that if the Criner fault was capable, it would have a length and surface displacement less than that of the Meers fault (NUREG-0797, CPNPP SSER 23, 1990).

Since these studies, new work has indicated that the Criner fault is less of a hazard than conservatively estimated in the late 1980s. In 1996, Williamson (Reference 2.5-294) conducted a thorough hand-excavation of the exposures along Hickory Creek. The resulting thesis concluded that only Pennsylvanian units were faulted along Hickory Creek in the same location cited by Geomatrix Consultants, and this faulting was overlapped by Quaternary alluvial units composed of sand, clay, and gravel (Reference 2.5-294). Furthermore, Williamson (Reference 2.5-294) pointed out that the scarp is restricted to the area where the resistant Ordovician limestone is adjacent to the fault and interpreted the scarp as a fault-line scarp. In addition, follow-up studies conducted by

Geomatrix Consultants have suggested that displacement along the scarp may be related to Late Pleistocene landslides (Reference 2.5-295). Geomatrix Consultants reported that small alluvial fans overlap the fault and that the fault could not be seen cutting units younger than Paleozoic (Reference 2.5-285).

Based on a review of post-EPRI scientific and industry literature, it is concluded that there is no conclusive evidence of the fault yielding Quaternary tectonic slip (e.g., CPNPP SER Suppl. 23; References 2.5-236 and 2.5-272). Because of the proximity of this structure to the CPNPP Units 3 and 4 site, William Lettis & Associates, Inc., conducted a field reconnaissance study along the escarpment in Love and Carter counties, Oklahoma. This study also found no evidence to support Quaternary tectonic activity on the Criner fault. In conclusion, the newest information bearing on the Quaternary activity of the Criner fault indicates that fault is not capable and should not be included in the EPRI seismic source characterization of this region.

2.5.1.1.4.3.7 Quaternary Tectonic Features Beyond the Site Region

In addition to the Quaternary tectonic features within the site region discussed above, three structures play significant roles in the hazard of the CPNPP Units 3 and 4 site, but are outside of the 200-mi radius (Figure 2.5.1-213). These features, the Rio Grande Rift, the Cheraw fault, and New Madrid fault zone, are discussed below.

2.5.1.1.4.3.7.1 Rio Grande Rift

The Rio Grande Rift (RGR) is a north-south-trending continental rift system that is recognized to extend from central Colorado through New Mexico, Texas, and into northern Mexico (Reference 2.5-296, 2.5-297, 2.5-298, 2.5-299, 2.5-300, and 2.5-301; Figure 2.5.1-213). At the time of the CPNPP Units 1 and 2 FSAR, relatively little was known about the seismogenic potential of faults within the RGR. However, more recent research has documented previously unrecognized Late Quaternary fault activity within the RGR (References 2.5-302, 2.5-303, 2.5-304, 2.5-305, 2.5-306, 2.5-307, 2.5-308, and 2.5-309). These studies indicate that the RGR is a zone of distinct and elevated tectonic activity relative to other regions at a similar distance from CPNPP Units 3 and 4. On the basis of these observations, the tectonic features of the RGR are relevant to CPNPP Units 3 and 4, despite the greater than 400 mi distance between the RGR and the site, because the faults of the RGR are some of the closest capable tectonic features.

The RGR is commonly thought to have developed in two main stages. The first stage, from approximately 30 to 20 Ma, involved low-angle normal faulting and basaltic volcanism. The second stage, from approximately 10 to 3 Ma, involved high-angle normal faulting that cut across and overprinted the earlier faulting and more expansive basaltic volcanism (Reference 2.5-310). The precise cause of the rifting during these two phases of activity is debated, but it is generally thought that a combination of elevated lithospheric temperatures and east-west tensional stress caused by plate interactions in western North America led to the thinning of

the lithosphere and thus the associated faulting and volcanism (Reference 2.5-310, 2.5-311, and 2.5-312). Despite the cessation of large-scale RGR formation, numerous faults within the RGR have been active within the Quaternary (References 2.5-302, 2.5-303, 2.5-304, 2.5-305, 2.5-306, 2.5-307, and 2.5-309).

Presently, the RGR is characterized by north-trending grabens centered on a broad topographic high, elevated heat flow, and a tensile stress regime (References 2.5-296, 2.5-300, 2.5-310, and 2.5-313). The east-west extent of the RGR surficial expression (e.g., faults and elevated topography) occupies a narrower region than the lithospheric structure of the RGR (region of tensile stress, thinned crust, elevated mantle, gravity anomaly) (References 2.5-241, 2.5-245, 2.5-300, 2.5-314, and 2.5-315). This observation suggests that the processes driving the Quaternary seismic activity observed within the RGR also extend beyond the region of the surficial expression of the rift (Reference 2.5-316).

An example of this phenomenon is the April 14, 1995, Alpine earthquake in West Texas discussed in Subsection 2.5.2.1.3.1 that occurred significantly eastward of the nearest RGR fault (Figure 2.5.1-240). The focal mechanism for this event shows that the earthquake was a normal faulting event with the minimum compressive stress (tensile stress) oriented north-northeast and the maximum horizontal stress oriented east-west (Reference 2.5-317). This event and others with similar focal mechanisms have been interpreted as reflecting the interaction of the topographically high RGR with relatively stable and low-lying Great Plains further east (References 2.5-318 and 2.5-319). Essentially, the RGR region is characterized by large gradients in gravitational potential energy caused by a combination of excess topography and variations in lithospheric density. These potential energy gradients create a tensile stresses regime at the eastern edge of the RGR, with the maximum horizontal compressive stress generally oriented east–west. These tensile stresses partially drive deformation within and well eastward of the physiographic RGR (References 2.5-245 and 2.5-220) as evident with the 1995 Alpine earthquake.

Quaternary faulting within the RGR has been reported in numerous studies that are well summarized and documented in the USGS Quaternary Fault and Fold Database of the United States (Reference 2.5-308). Summaries of these faults are not presented here due to the large number of faults. However, some of these faults have been studied in enough detail to generate complete seismic source characterizations, and these faults are included in the 2002 USGS National Seismic Hazard Maps (Reference 2.5-321). The seismic source characterizations of these faults are discussed in detail in Subsection 2.5.2.4.2.3.3.

2.5.1.1.4.3.7.2 Cheraw Fault

The Cheraw fault is located in southeastern Colorado over 500 mi from the site (Figure 2.5.1-213). The potential for Quaternary events on the fault was first noted by Scott (Reference 2.5-322) and three Late Quaternary events were dated by Crone, et al. (Reference 2.5-323). The fault is included in this discussion because,

despite its great distance from the site, it is one of the closest capable faults to the site. The Cheraw fault is structurally positioned above and between the Las Animas Arch, an approximately 200-mi-long arch in Precambrian crystalline rocks, and the Denver Basin, a complementary basement low to the northwest of the arch (References 2.5-324 and 2.5-325). Offset across the fault is concordant with the offset in the basement surface between the arch and basin, down to the northwest, but the fault is not observed to offset the basement surface (Reference 2.5-324). Fault offsets across buried bedrock horizons are on the order of tens to hundreds of ft, and fault offsets of Quaternary deposits are only 23 to 26 ft (References 2.5-323 and 2.5-326). These observations suggest that the fault has not had a long history of movement (millions of years).

The surface trace of the fault has been mapped for approximately 27 mi, but in many places the fault is mapped as approximately located, inferred, or concealed. Where observed, the fault displaces Early Pleistocene piedmont surfaces, and in trenches the fault is observed to displace Late Pleistocene deposits (References 2.5-325 and 2.5-326). A trenching study by Crone, et al. (Reference 2.5-323) found evidence for three surface-rupturing events at approximately 8, 12, and 20 to 25 ka. Prior to these three events, Crone, et al. (Reference 2.5-323) hypothesize that the fault was inactive since approximately 100 ka based on the presence of a filled paleo-stream channel. These observations suggest that the fault may have a clustered earthquake behavior (Reference 2.5-326).

2.5.1.1.4.3.7.3 New Madrid Seismic Zone

The New Madrid Seismic Zone (NMSZ) extends from southeastern Missouri to southwestern Tennessee and is located approximately 500 mi northeast of the site (Figure 2.5.1-213). The NMSZ lies within the Reelfoot rift and is defined by post-Eocene to Quaternary faulting with previous older seismic activity. Given its significant distance from the site, the NMSZ did not contribute to the seismic hazard calculated by the Electric Power Research Institute Seismicity Owners Group (EPRI-SOG) for CPNPP Units 1 and 2 (Reference 2.5-327). However, the NMSZ needs to be reconsidered for CPNPP Units 3 and 4 because several more recent studies provide significant new information regarding characterization of the seismic capability of the NMSZ.

The NMSZ is approximately 125 mi long and 25 mi wide. Research conducted since the EPRI-SOG study has identified three distinct fault segments embedded within the seismic zone, consisting of a southern northeast-trending dextral slip fault, a middle northwest-trending reverse fault, and a northern northeast-trending dextral strike-slip fault (Reference 2.5-271). In the current east-northeast to west-southwest directed regional stress field, Precambrian and Late Cretaceous age extensional structures of the Reelfoot rift appear to have been reactivated as right-lateral strike-slip and reverse faults.

The NMSZ produced a series of historical, large-magnitude earthquakes between December 1811 and February 1812 (Reference 2.5-328). The December 16, 1811, earthquake is associated with strike-slip fault displacement along the

southern part of the NMSZ. Johnston (Reference 2.5-329) estimates a magnitude of Mw 8.1 ± 0.31 for the 16 December 1811 event. However, Hough, et al. (Reference 2.5-328) re-evaluated the isoseismal data for the region and conclude that the December 16 event had a magnitude of Mw 7.2 to 7.3. Bakun and Hopper (Reference 2.5-330) similarly concluded this event had a magnitude of Mw 7.2.

The February 7, 1812, New Madrid earthquake is associated with reverse fault displacement along the middle part of the NMSZ (Reference 2.5-331). This earthquake most likely occurred along the northwest-striking Reelfoot fault that extends approximately 43 mi from northwestern Tennessee to southeastern Missouri. The Reelfoot fault is a northeast-dipping, southwest-vergent reverse fault. The Reelfoot fault does not extend updip to the earth's surface, but a topographic scarp has developed above the buried tip of the fault as a result of fault-propagation folding (References 2.5-332, 2.5-333, and 2.5-334). Johnston (Reference 2.5-329) estimated a magnitude of Mw 8.0 ± 0.33 for the 7 February 1812 event. However, Hough, et al. (Reference 2.5-328) re-evaluated the isoseismal data for the region and concluded that the February 7 event had a magnitude of Mw 7.4 to 7.5. More recently, Bakun and Hopper (Reference 2.5-330) estimated a similar magnitude of Mw 7.4.

The January 23, 1812, earthquake is associated with strike-slip fault displacement on the East Prairie fault along the northern part of the NMSZ. Johnston (Reference 2.5-329) estimates a magnitude of Mw 7.8 ± 0.33 for the January 23, 1812, event. Hough, et al. (Reference 2.5-328), however, re-evaluated the isoseismal data for the region and concluded that the January 23, 1812, event had a magnitude of Mw 7.1. More recently, Bakun and Hopper (Reference 2.5-330) estimated a similar magnitude of Mw 7.1. The upper-bound Mmax values used in the EPRI-SOG study (References 2.5-369 and 2.5-335) for the NMSZ range from m_b 7.2 to 7.9, generally consistent with the revised magnitudes for the three events reviewed here.

Because there is very little surface expression of faults within the NMSZ, earthquake recurrence estimates are based largely on dates of paleoliquefaction and offset geological features. The most recent summaries of paleoseismologic data (References 2.5-336, 2.5-337, and 2.5-338) suggest a mean recurrence time of 500 years, which was used in the 2002 USGS model (Reference 2.5-321). This recurrence interval is half of the 1,000-year recurrence interval used in the 1996 USGS hazard model (Reference 2.5-339), and an order of magnitude less than the seismicity-based recurrence estimates used in the EPRI-SOG study (Reference 2.5-369 and 2.5-240).

The NMSZ studies described above that post-date the EPRI-SOG study require an updated NMSZ source model for CPNPP Units 3 and 4 because the studies provided revised estimates of the source geometry, maximum magnitudes, and recurrence intervals compared to those of the EPRI-SOG study (References 2.5-221 and 2.5-239). The updated source model used for CPNPP Units 3 and 4 is described in Subsection 2.5.2.4.3.

2.5.1.2 Site Geology

This subsection discusses details about the site area and site, geologic history, physiography, stratigraphy, lithologies, and geologic structure.

2.5.1.2.1 Site Physiography and Topography

This subsection discusses the physiography, geologic history, stratigraphy, and tectonic setting within a 5- and 0.6-mi radius of the site.

Topographic maps of the site area (5-mi radius) and site (0.6-mi radius) are shown in Figures 2.5.1-214 and 2.5.1-215, respectively. The site area is almost completely contained within the Grand Prairie physiographic province, which is underlain by flat-lying Lower Cretaceous limestones and shales with intervening sandstone units that mark transgressive events. The limestone–shale sequences are variably resistant to erosion with the harder, more resistant limestone units forming steeper slopes than the less resistant shale units. This results in the stair-step topographic expression characteristic of this region.

The major drainage is the Brazos River, which is expressed as several incised meander loops in eastern portions of the site area (Cox Bend) and beyond. Southern portions of the site area are drained by the Paluxy River and Squaw Creek, which bisects the site area from northwest to southeast. Most of the Squaw Creek drainage area in the site area is now covered by Squaw Creek Lake. The site location is situated between the now submerged Squaw Creek drainage and Panther Branch.

Surface elevations in the site area range from about 1190 ft to about 580 ft msl, and this relief is due to incision by the major drainages and the differential weathering resistance of site area rocks. The higher elevations occur in the northern portions of the site area in association with the southern flanks of Comanche Peak, a distinctive topographic high in the area that is located between the drainages of Squaw Creek and the Brazos River. Comanche Peak is an erosional remnant of the Fredericksburg Formation, underlain by resistant Edwards limestone. However, throughout most of the site area the major drainage divides are underlain by Paluxy Formation. The Paluxy Formation forms soils that are conducive to cultivation, in contrast to the thin rocky soils formed by the Glen Rose limestone, shale sequences that occur in the areas dissected by the drainages. The lower elevation occurs at the level of the Brazos River Valley in the southeastern portions of the site area.

Surface elevations of the site (0.6-mi radius) range from 870 to 680 ft msl. The highest elevation is located on the drainage divide between the submerged Squaw Creek and Panther Branch. The pool level of Squaw Creek Lake is elevation 775 ft msl, so the quoted lower elevations now occur in the submerged Squaw Creek Valley.

2.5.1.2.2 Site Geologic History

Geologic maps of the site vicinity (25-mi radius), site area (5-mi radius), and site (0.6-mi radius) are shown in Figures 2.5.1-216, 2.5.1-217, and 2.5.1-218, respectively. Also, the stratigraphy for the site area and site is given in Figure 2.5.1-219. The geologic setting of the site area (5-mi radius) is characterized by an Early Paleozoic stable Laurentian Margin basement-cover sequence that has been modified by crustal flexure and associated synorogenic, Late Paleozoic sedimentation resulting from the Ouachita orogeny. At the conclusion of Late Paleozoic orogenic activity a paleoplain developed on the synorogenic clastic material, which was on-lapped by Lower Cretaceous clastic and carbonate sequences developed on a stable platform during transgressive phases of the Gulf of Mexico.

The Laurentian shelf geologic setting represents the drift stage evolution of a trailing-edge continental margin. In the site area, this evolution is represented by deposition of carbonate-dominated sequences (Canadian Provincial Series, Figure 2.5.1-219) with interlayered clastic material that records several transgressive–regressive depositional cycles. The passive evolution of the Laurentian Margin sequence persisted to the Middle Mississippian. However, the top of the Ellenburger Group records an extensive erosional surface with widespread development of karst. Therefore, post-Upper Ellenburger–pre-Upper Mississippian sediments are absent or poorly preserved.

During the Late Mississippian, the Laurentian Margin was involved in southern subduction beneath peri-Gondwanian, subduction-related volcanic-arc terrane(s). The tectonic effects of this orogenic event (Ouachita orogeny) included thrust assembly of tectonostratigraphic units that lie in the subsurface east of the Ouachita orogenic front several miles to the east of the site area. However, crustal loading of this tectonostratigraphy onto the Laurentian Margin resulted in the development of crustal arches and related downwarps that developed into foreland basins filled by synorogenic and postorogenic material. As the tectonostratigraphy was emplaced onto the margin from southeast to northwest, the crest of the crustal arches and the depocenters associated with the foreland basins migrated in a similar fashion. The site and site area are located above one of these foreland basins, the Fort Worth Basin, and consequently the Late Paleozoic history of the site area is closely associated with the history of this basin.

Initial arching and subsequent subsidence of the Fort Worth Basin was marked by the deposition of shallow-water facies associated with the arches and arch margins, and starved-basin, deep-water clastic and carbonate facies in the basin interior. These deposits comprise the Osagean and Chesterian–Meramecian Series. As the arch and following basin retreated to the west and northwest in response to the advancing Ouachita thrust sheets, the shallow water arch and deep basin facies were followed and overridden by thick sequences of synorogenic clastic material derived from the topographic highlands to the

southwest and from nearby arches, primarily the Munster Arch. This material now comprises the Atoka Series units.

As the suturing of Laurentia and Gondwana was completed, and Ouachita tectonic kinematic effects waned, the Fort Worth Basin filled and became a shallow water environment with decreased paleoslopes and decreased clastic input. This phase of lower tectonic activity lead to deposition of the Des Moines and Missouri Series of the Strawn Group. Although deposition probably persisted for some time into the Late Pennsylvanian, in the Late Pennsylvanian the eastern side of the Fort Worth Basin was uplifted and rotated so that the depositional sequence was truncated by an erosional surface. At the site location, this erosional surface is the top of the Mineral Wells Formation.

During the Early Cretaceous, northward and northwestward transgression by the Gulf of Mexico onto a stable platform reached the site area and resulted in deposition of sequences of shallow-water carbonate and clastic strata that are now preserved as the Comanchean Series deposits. At various times until the Late Cretaceous (Maastrichtian), the Gulf Basin was hydrologically connected with the Pacific Ocean by the “Western Interior Seaway” which coved the site area. This implies that, at one time, sediments as young as Late Maastrichtian were probably present.

Subsequently, the area was uplifted in the Late Cretaceous by initial Laramide activity to the west. Erosion related to this uplift has stripped sediments younger than the Paluxy Formation from the site area, except for limited exposures of Fredericksburg Group associated with the Comanche Peak topographic high and the presence of Quaternary alluvial and terrace deposits in the major drainages.

2.5.1.2.3 Site Stratigraphy

The exposed stratigraphy in the site area consists entirely of the Lower Cretaceous Comanche Series, comprising the Trinity and Fredericksburg groups (Figure 2.5.1-219). In addition, more recent Quaternary alluvial sequences associated with fluvial deposition in drainages, primarily the Brazos River Valley in the southeastern portion of the site area, the Paluxy River Valley in the southwestern portion of the site, and Squaw Creek, are present.

The oldest unit exposed in the site area is the Glen Rose Formation (Trinity Group) and most of the site is underlain by Trinity Group lithologies (i.e., Glen Rose Formation and the overlying Paluxy Formation). Erosional remnants of Fredericksburg Group (Walnut and Comanche Peak formations) are preserved in the northernmost portions of the site area around the Comanche Peak topographic high, Figure 2.5.1-214.

Although not exposed in the site area, several geotechnical borings drilled for the CPNPP Units 3 and 4 investigation sampled the Twin Mountains Formation (lower Trinity Group), and one boring (B-1012) penetrated the sub-Cretaceous unconformity to sample the Upper Paleozoic Mineral Wells Formation. The

nearest exposures of these units occur approximately 5 mi west of the site area. Although most of the detailed discussion of the site stratigraphy will focus on the stratigraphic units that occur above the Mineral Wells Formation, some information for the deep stratigraphic units has been incorporated as it pertains to the site dynamic profile for ground-motion response (Subsection 2.5.2.5). Therefore, the deeper stratigraphic section down to crystalline basement will also be briefly discussed. Information on the deeper stratigraphic units in the site area is extrapolated from deep borings and from exposures of correlative units to the west of the site area and in the region of the Llano Uplift.

2.5.1.2.3.1 Precambrian and Cambrian to Ordovician

The Late Cambrian to Ordovician section rests unconformably on Grenville-age crystalline rocks. These units represent Laurentian basement-cover successions that were deposited along the southern Laurentian stable continental shelf platform. The basement component consists of undifferentiated metaigneous and metasedimentary crystalline lithologies that were subject to Grenville-age metamorphism. This crystalline basement is unconformably overlain by carbonate-dominated shelf deposits of the Canadian Series that record several transgressive–regressive events associated with the Laurentian Margin drift stage evolution. The characteristics of these strata are discussed in Bell and Barnes (Reference 2.5-341).

The lowest formation in the Canadian Series is the Riley Formation, which represents a marine transgressive–regressive cycle. The base of the Riley Formation consists of the Hickory Sandstone Member, which is a noncalcareous quartz sandstone that rests directly on Precambrian basement. The basal unit is overlain by the Cap Mountain Limestone Member, which is a glauconitic, impure sandy to clayey limestone, and is in turn overlain by the highly glauconitic quartz-rich Lion Mountain Sandstone Member.

The Wilberns Formation disconformably overlies the Riley Formation. The Wilberns Formation consists of four members, in stratigraphic order from oldest to youngest: Welge Sandstone, Morgan Creek Limestone, and Point Peak and San Saba Members. The Point Peak has a diverse lithologic composition consisting of siltstone, limestone, conglomerate, and stromatolitic bioherms. The San Saba Member consists of calcitic, dolomitic, and sandstone components. Projection of the isopachs given in Bell and Barnes (Reference 2.5-341) indicates that the total thickness of the Riley and Wilberns formations at the site location is slightly greater than 250 ft.

The Lower Ordovician section is an incomplete carbonate sequence (Ellenburger Group). This group consists of the basal Tanyard Formation, Gorman Formation, and Honeycut Formation. These are all predominately dense limestones and dolomites. The combined thickness of the Tankard and Gorman formations in the site vicinity is on the order of 1500 ft (Reference 2.5-342). Thicknesses of the Honeycut and post-Honeycut formations (Simpson – Viola) are a little less than 750 ft (Reference 2.5-342). An extensive unconformity with extensive karstic

development is present at the top of the Ellenburger Formation. Pre-Mississippian overlying units are missing, thinly developed, or only preserved in association with karst features.

2.5.1.2.3.2 Mississippian to Upper Pennsylvanian

The Upper Mississippian to Lower Permian section consists of synorogenic Ouachita sediments that were deposited on arches and in basins associated with crustal flexure resulting from crustal loading to the east. These sediments were deposited in the Fort Worth Basin, which developed as a foreland basin in response to the Ouachita orogeny.

The lower part of the section, consisting of the Middle to Upper Mississippian Chappel Formation, Barnett Formation, and lower Marble Falls Formation were deposited in response to the initial development and subsidence of the Fort Worth Basin (Reference 2.5-343). The Chappel Formation consists of crinoidal limestones and pinnacle reefs. The Barnett consists primarily of siliceous shale with minor limestone and dolomite, which were deposited mainly in a starved-basin environment. Based on the isopach map presented in Montgomery and others (Reference 2.5-343), the Barnett Shale is about 250 ft thick at the site location. The lower portion of the Marble Falls Formation consists of interbedded dark limestone and grey-black shale, which is less organic and radioactive than the shale lithologies in the lower Barnett Shale.

The upper interval of the Marble Falls Formation consists primarily of shallow-water limestone deposited in association with arches that formed topographically high areas. As the crustal arch (Bend Arch) retreated to the west in front of the deepening Fort Worth Basin, Marble Falls limestone deposition was replaced and overridden first by prodelta facies, deep-water deposits and later deltaic sequences of the Atoka Group (Reference 2.5-213). The lower Atoka Group sequences correspond approximately to the Big Saline Formation (Reference 2.5-344), which is primarily sandstone with some thin limestone units. The upper Atoka Group sequences consist of shale (Smithwick) and sandstones (Atoka sand). Atoka Group sediments are approximately 2000 ft thick beneath the site area based on well data from the nearby Officers' Club Number 7 boring.

Following Atoka Group deposition, decreased subsidence and filling of the Fort Worth Basin with sedimentary material led to shallow-water conditions and deposition of a series of deltaic, terrigenous clastics characterized by lower paleogradient (Reference 2.5-344). These sediments comprise the formations of the Strawn Group, which consist of a series of primarily shales interbedded with sandstones, conglomerates, and limestones with some coal. The youngest Strawn Group Formation present at the site is the Mineral Wells Formation, which lies below the pre-Cretaceous unconformity. Based on the Officers' Club Number 7 well, Strawn Group sediments are about 2200 ft thick beneath the site.

The Mineral Wells Formation consists primarily of shale, with sandstone and thin limestone intervals. Petrographic analysis of samples from geotechnical boring

B-1012 indicate that the Mineral Wells lithology at the site is mostly a silty claystone with a poorly developed fabric along with interbedded, immature sandstone with significant amounts (18 percent) of K-feldspar. The Mineral Wells is the deepest unit encountered by borings drilled for the CPNPP Units 3 and 4 investigation.

2.5.1.2.3.3 Lower Cretaceous

The Lower Cretaceous Trinity Group forms the bedrock lithologies for most of the site area and site. The Trinity Group consists of the basal Twin Mountains Formation, the intermediate Glen Rose Formation, and the overlying Paluxy Formation. This stratigraphic section at the site location was subdivided into several engineering layers based on lithologic parameters noted from the geotechnical borings and geologic mapping of the area. These engineering layers are discussed in Subsection 2.5.4.3.1 (Reference 2.5-345).

At the type section locality on the north side of Twin Mountains, Erath County, northeast of the site area, the Twins Mountains Formation is about 150 ft thick and subdivided into three intervals:

- A lower interval with a basal conglomeratic, cross-bedded sand with well-rounded pebbles overlain by a grey and green, locally yellow and red silty and sandy clay followed by a buff, medium-grained well-sorted sand locally, with concretions and pebbles

- An intermediate interval with green and grey silty clay overlain by light grey to buff, fine- to medium-grained, well-sorted cross-bedded sand, with pebble-size conglomerate

- An upper clay interval consisting of red and grey silty clay overlain by green, grey, and yellow silty clay.

Beneath the site, as sampled in geotechnical boring B-1012, the Twin Mountains Formation is over 200 ft thick, with a well-developed, conglomeratic, medium to coarse sand at the base*.* This is overlain by primarily sandstone, with an intervening interval composed primarily of shale with sandstone interbeds. The upper part of the formation is transitional with the Glen Rose Formation and consists of limestone shale and sandstone lithologic components. These relationships are broadly consistent with those seen at the type locality.

The type locality for the Glen Rose Formation is located in the Paluxy River Valley in the southwestern portions of the site area (Reference 2.5-346). At the type locality, the Glen Rose Formation contains three members. The lower member consists of an alternating series of terrigenous clastics and carbonates. The middle of the Glen Rose Formation consists mostly of a massive limestone (Thorp Spring Member). The upper unit is similar to the lower member and consists of alternating sequences of carbonates and clastic sediments.

Glen Rose Formation lithologies at the site location range from primarily wackestone to packstone-dominated carbonates that are interbedded with mudstones and shale. Some of the carbonate intervals contain significant amounts of sparite cement and form hard, extremely resistant, units. At least one of these units is present near the top of the formation and forms conspicuous pavement outcrops that are readily visible and mappable from aerial photographs.

The upper unit of the Trinity Group is the Paluxy Formation. This unit typically outcrops at the higher elevations associated with drainage divides in the site area. This unit is only present at the site (0.6-mi radius) in a small area in the western portion. The Paluxy Formation is a loosely consolidated to unconsolidated well-sorted sandstone that is typically reddish brown, but can also be tan to buff colored. It may also contain up to 10 percent clay.

As previously discussed, Fredericksburg Group units only occur in a small area in the northern portion of the site area near Comanche Peak where it occurs stratigraphically above the Paluxy Formation. Most of the exposure consists of the Walnut Formation of the lowermost Fredericksburg Group and is a dark grey to brown clay that is fossiliferous with nodular limestone and shale.

2.5.1.2.3.4 Quaternary

Quaternary sediments in the site area, occur as terrace and floodplain deposits associated with the Paluxy and Brazos Rivers and Squaw Creek. The terraces and modern floodplain along the Brazos River have been studied by Stricklin (Reference 2.5-470) and Epps (Reference 2.5-457). In addition to the modern floodplain these authors recognized at least three terraces in the site vicinity distinguished by their position relative to the modern floodplain, referred to as upper, middle and lower. The modern floodplain and the terraces consist of a “normal sequence of graded alluvium” (Reference 2.5-470) that is characterized by a fining upward sequence defined by a basal gravel in the lower 10 ft that grades upward into sands, silts and clay. This depositional sequence represents channel migration and lateral aggradation at the base followed by vertical aggradation due to flooding in the upper part of the section. The composition of the gravel component varies based on the terrace level, becoming richer in limestone and chert in the lower, and consequently younger terraces (Reference 2.5-457).

Based on the presence of both vertebrate and mulluscan fauna (Reference 2.5-470 and 2.5-457) the terraces are Pleistocene in age. Epps (Reference 2.5-455) proposes that the terrace formation was a response to climatic changes associated with Pleistocene cyclic glaciation with incision and floodplain abandonment associated with glacial periods and floodplain development and soil formation during interglacial events.

2.5.1.2.4 Site Structural Geology

The investigation of the CPNPP Units 3 and 4 structural geology was performed at the 25-mi (vicinity), 5-mi (area), and 0.6-mi (site) radii and included analysis of previously published studies and maps of the area (including peer-reviewed literature, Bureau of Economic Geology map compilations, and the CPNPP Units 1 and 2 FSAR), proprietary petroleum industry data purchased from Geomap Company, geotechnical data collected by William Lettis & Associates, Inc., and its contractors, and field mapping and reconnaissance studies conducted by William Lettis & Associates, Inc. All of these data sources indicate that the structural geology within the 25-mi radius of the site is dominated by relatively undisturbed, primary, sedimentary structures (i.e., depositional bedding). There is a high degree of correspondence between topography and the trace of geologic contacts for maps at all three scales (25-, 5-, and 0.6-mi radii, Figures 2.5.1-216, 2.5.1-217, and 2.5.1-218, respectively). The only tectonic structures found within the 25-mi radius of the site are a set of buried faults that were active in the Paleozoic. No tectonic structures were found within 5 mi of the CPNPP Units 3 and 4 site.

2.5.1.2.4.1 Site Vicinity (25-mi radius)

The geologic map of the site vicinity indicates a set of virtually flat Cretaceous sedimentary units that dominate the exposures in the area (Figure 2.5.1-216). The simple layer-cake stratigraphy is highlighted by the good correspondence of unit contacts to topography. In the northwestern portion of the area, Paleozoic units, which are unconformably overlain by the Cretaceous rocks, are exposed and indicate the regional erosional limit of Cretaceous units in Texas. Along the river drainages such as the Brazos, Quaternary terrace and alluvial units outcrop. No faults are exposed at the surface within the site vicinity. However, planar bedding is disturbed by faults buried below the base of the Cretaceous rocks within the 25-mi radius, but outside of the site area (Figure 2.5.1-220). These buried faults were mapped by recognizing offsets in the tops of Paleozoic units (such as the Mississippian-Pennsylvanian Mable Falls with well-log data from petroleum exploration. Generally, these faults are normal faults, which strike northeast-southwest. These faults cut Pennsylvanian and older units, but terminate upwards into the Strawn series, indicating that slip on these structures stopped during the deposition of the Strawn Formation (CPNPP Units 1 and 2 FSAR). There is no evidence of reactivation after deposition of the middle to upper Strawn (Pennsylvanian). These faults are interpreted as accommodating the flexure of the Fort Worth Basin due to loading from the southeast as sediment accumulated adjacent to the Ouachita thrust front.

2.5.1.2.4.2 Site Area (5-mi radius)

The structural geology within the 5-mi radius of the CPNPP Units 3 and 4 site is dominated by flat-lying bedding of the Cretaceous Glen Rose limestone and overlying Paluxy sand (Figure 2.5.1-217). No tectonic structures (such as faults, folds, or shear zones) were found within 5 mi of the CPNPP Units 3 and 4 site.

Minor warping (<3 ft in amplitude) of sedimentary beds was noted in two outcrops within the site area from geologic mapping. A discussion of aerial photography, and a lineament analysis and field reconnaissance is provided in Subsection 2.5.3.2 as part of the surface faulting studies.

Stratigraphic horizons picked in wells (Figure 2.5.1-221) were used to determine the stratigraphy at depth and construct two cross sections (Figure 2.5.1-222 and Figure 2.5.1-223). The seven horizons displayed are the pre-Cretaceous unconformity at the top of the Pennsylvanian Strawn Group (also the top of the Mineral Wells Formation), the top of the Atoka Sand, top of the Smithwick Formation, top of the Big Saline Formation, the top of the Marble Falls Limestone, the top of the Barnett Shale, and the top of the Ordovician Ellenburger Group. For simplicity, the rocks separated by these horizons are labeled as Cretaceous, Strawn, Atoka, Smithwick, Big Saline, Marble Falls, Barnett Shale, and Ellenburger in Figures 2.5.1-222 and 2.5.1-223. These structural sections reveal undisturbed flat to very gently dipping bedding beneath the site. For example, the erosional top of the Ellenburger limestone dips gently (<2°) to the east (Figure 2.5.1-222). The simple, gently dipping structure of the bedding is reflected in the structure contour maps of the Marble Falls limestone and Barnett Shale (Figures 2.5.1-224 and 2.5.1-225). These maps reveal that the top of the Marble Falls limestone and the Barnett Shale have similar morphologies at depth, both dipping gently to the east <1° (~80 ft/mi). Both structure contour maps also reveal a small nose plunging northward from the extreme western edge of the area. The structure contour maps and cross sections do not indicate any potential faulting at depth within the site area. If future well data reveal any subsurface faults, it is expected that they will be similar to the normal faults found within the site vicinity. Therefore, any potential fault is expected to be located >2000 ft beneath the site and have no offset since the Paleozoic.

At the surface, the Cretaceous rocks and overlying alluvial Quaternary units are virtually undisturbed within the site area. However, fieldwork did reveal two localized what appear to be gentle folds (warps) in bedding (Figure 2.5.1-217). However, it is not clear if these structures have a well defined fold axis or if they are more domal. While these features are discussed here for completeness, the previous FSAR discussed them only to mention that "slight, gently warping of the sediments is occasionally evidenced in road cuts and natural exposures in the area surrounding the site" and attributed such features to differential settlement during compaction and diagenesis (CPNPP Units 1 and 2 FSAR). The present investigation revealed these two disruptions are primarily thickness variations in specific sedimentary beds. The first disruption is in a sand of the Paluxy Formation along Highway 56. Seen in a roadcut, one bed within the Paluxy Formation appears to dip more steeply than beds adjacent (Figure 2.5.1-226). This outcrop was inspected for evidence of tectonic deformation; no faults or shear zones were discovered. Small veins or fractures filled with calcite are found in this outcrop, but there is no offset across these features. The second disruption is in limestone of the Glen Rose Formation near the dam for the SCR (Figure 2.5.1-217). This is visible in a roadcut for the road that leads to the spillway from the top of the dam. This roadcut, on the northeast side of the road, shows a

portion of an apparent gentle fold in which a bed changes thickness by 1 to 3 ft over approximately 30 to 40 horizontal ft (Figure 2.5.1-227). No fractures, faults, lineaments, or other deformational indicators were found to provide a tectonic cause for this feature. Additionally, it appears that beds above and below this thinned bed have no or lessened disruption. The cause of these small thickness variations is not obvious. The large scale at which they occur suggests that only a minimal amount of differential consolidation/compaction, mineralization, or dissolution could be responsible for the minor changes in the attitude of the beds. Inspection of the outcrop did not reveal significant mineralogical or textural changes that could cause the warping. However, anhydrite and gypsum are known to occur in the Glen Rose limestone in Somervell County (Reference 2.5-346) that could cause volumetric changes, and therefore bed thickness changes. The features are confined to the Cretaceous units, and based on the vast majority of the Cretaceous units being undisturbed, we expect that beds overlying these warped areas (which have been eroded away) would be unwarped. We conclude that the two features are probably sedimentary in nature and developed close in time to the original deposition of these units (in the Cretaceous).

2.5.1.2.4.3 Site (0.6-mi radius)

The Glen Rose limestone and artificial fill (associated with the construction of CPNPP Units 1 and 2) are the dominant materials exposed within the 0.6-mi radius of the CPNPP Units 3 and 4 site (Figure 2.5.1-218). A small outcrop of Paluxy sandstone is found in the western portion of the area and a drainage in the southern portion of the area exposes Quaternary alluvium. Again, the trace of units within the Glen Rose limestone is largely influenced by topography (Figure 2.5.1-218). Analysis of published studies and remotely sensed data as well as field mapping, geotechnical borings, and the construction of engineering cross sections (Figure 2.5.1-218; see discussion and figures in Subsection 2.5.4.3) reveal no geologic structures within the 0.6-mi radius of the CPNPP Units 3 and 4 centerpoint.

2.5.1.2.4.4 Site Area Gravity and Magnetic Fields

This subsection and those that follow evaluate both naturally occurring and man-induced geologic conditions that pose potential risks to the CPNPP site. The evaluation presented herein is based on the design and construction plans of CPNPP Units 3 and 4, augmented by a search of published maps and reports, field reconnaissance, consultation with researchers in the area, as well as data acquired from the geotechnical exploration program discussed in Subsection 2.5.4. The following engineering geology aspects were identified as pertinent to the site.

The available resolution of the gravity data in the site area is limited due to the fact that only about 10 data points for the gravity data set fall within 5 mi of the site. Similarly, the flight lines on which the aeromagnetic data were derived were spaced 1.25 mi apart (Reference 2.5-220). The resolution limits imposed by this relatively low data density preclude detailed analysis of the gravity and magnetic

data fields for the site area and site location. However, examination of the regional gravity and magnetic fields as presented in Figures 2.5.1-205 and 2.5.1-206 does allow some inferences of general features for the site area.

The site area is located on a subdued gravity high with isostatic gravity field values of about -20 mGal. In contrast, the isostatic gravity field to the northwest and southeast is characterized by values of -40 to -50 mGal. As discussed in Keller, et al. (Reference 2.5-222), this gravity high is known to distort the signature of the Fort Worth Basin, in that the basin is known to extend to the north of the gravity low that characterizes the basin to the south. This would indicate that the crust beneath the northern part of the Fort Worth Basin, and consequently the site area, is anomalously dense compared to that to the south and northeast.

As can also be seen in Figure 2.5.1-206, the site area also occurs near and in association with a regional aeromagnetic high of approximately 250 nanoTeslas (nT). The coincidence of both high aeromagnetic and relatively high gravity anomalies indicates the presence of magnetic and relatively dense material in the crystalline basement beneath the site area. These relationships indicate that the crystalline basement beneath the site area probably contains significantly more mafic material than the surrounding basement.

2.5.1.2.5 Site Engineering Geology

The following sections evaluate both naturally occurring and man-induced geologic conditions that pose potential risks to the site area. This evaluation is based on the design and construction plans of CPNPP Units 3 and 4 augmented by a search of published maps and reports, field reconnaissance, consultation with researchers in the area, as well as information acquired from the geotechnical exploration program discussed in Subsection 2.5.4. The following engineering geology aspects were identified as pertinent to the site.

2.5.1.2.5.1 Dynamic Behavior During Past Earthquakes

The CPNPP site is located in a tectonically stable region as indicated by the compilation of earthquake activity for the region, as discussed in Subsection 2.5.2.1, and a thorough study of the regional geologic history, presented in Subsection 2.5.1.1.2. Subsection 2.5.2.1.1 discusses historic earthquake activity in the region surrounding the CPNPP site. Subsection 2.5.2.3 documents that no evidence for correlating earthquake activity of E_{mb} > 3.0 to any known seismic sources exists within 90 mi of the site. Although the region is not well instrumented to measure small-magnitude earthquakes, a screening of the region within the 200-mi area surrounding the site shows no seismic activity, as discussed in Subsection 2.5.2.1.

Field reconnaissance of the region and immediate site area indicates no evidence of seismic activity, either recent or historic. Field reconnaissance, other than that conducted on the site location, consisted of visiting publicly accessible locations in the site area and immediately surrounding vicinity (Figure 2.5.1-231 and 2.5.1-

232). Generally, all publicly accessible locations in and around the site area were visited in order to verify the accuracy of the site area map, to search for signs of deformation (faulting or folding) in bedrock and surficial outcrops, and to search for paleoliquefaction features. Minor flexures, limited in both vertical and lateral extent (less than 3 ft and 40 ft, respectively) have been noted in surrounding exposures of the Glen Rose Formation. However the limited extent and lack of evidence of offset or brittle deformation indicate that these flexures may be related to non-tectonic factors such as differential consolidation, or dissolution of underlying sediments. This interpretation is strengthened by the observation that underlying beds do not mimic the structure in the case of the fold in the Glen Rose Formation. A review of the core that was obtained from the borings drilled as part of the site geotechnical investigation (discussed in Subsection 2.5.4) shows no evidence for brittle or ductile deformation that can be related to seismic activity.

2.5.1.2.5.2 Zones of Weathering, Alteration or Structural Weakness

The area for CPNPP Units 3 and 4 is cut to a yard grade of elevation 822 ft msl and all weathered materials are to be removed, as discussed in Subsection 2.5.4.5.1. All Category 1 structures are founded directly on a thick (average 65 ft), laterally extensive, limestone unit within the Glen Rose Formation, at about elevation 782 ft msl. Subsection 2.5.4 discusses these conditions, including the static and dynamic properties of this and other subsurface layers. Site reconnaissance of exposures surrounding the site, a review of aerial photography, and examination of borings drilled as part of the geotechnical investigation showed no zones of enhanced weathering or structural weakness such as fractures or joints. Also, petrographic analysis of samples acquired from core borings drilled as part of the geotechnical investigation, as well as samples taken from exposures surrounding the site, does not indicate any secondary alteration of minerals.

Reconnaissance of the site area included several of the incised ephemeral drainage valleys and outcrops of the Glen Rose Formation. As described in Subsection 2.5.1.2.3, with the exception of limited Paluxy Formation exposures, most of the outcroppings at the surface, and within incised valleys surrounding the CPNPP site, consist of shales and limestones of the Glen Rose Formation. The surface of the Glen Rose is typically a weathered mantle ranging from 0 to several ft thick, as noted from reconnaissance studies of the site area as well as from borings drilled as part of the geotechnical investigation discussed in Subsection 2.5.4. It was also noted from the geotechnical borings that the weathered zone, discounting areas where fill has been placed, ranges from 0 to about 10 ft thick and terminates abruptly at competent rock with rock quality designations (RQD) and percent recoveries of core of greater than 90 percent. It is also noted from the geotechnical borings that bedding planes within the shale layers are primarily horizontal and develop partings along these planes as the material desiccates. These shale layers overlie the limestone foundation-bearing unit and will be excavated and removed during construction. Excavation walls will require engineered reinforcement as these shale layers exist near the base of the excavation as described in Subsection 2.5.4.

Subsurface materials below the elevation of surrounding exposures were evaluated from borings drilled for the geotechnical investigation including drilling notes and core descriptors. Drill water returns were consistent for nearly all borings, indicating that no major joints or fractures were intersected by the borings: joints or fractures would have resulted in loss of fluid return. Minor fluid loss was noted in some borings that encountered sands of the Twin Mountain Formation; however, no evidence for joints or fractures was noted in the recovered core corresponding to these intervals. Thus, these fluid losses are attributed to porous conditions of the Twin Mountain sands or excessive pump pressures during drilling. To further investigate the potential for near-vertical joints or fractures that may have been missed by the vertically drilled geotechnical borings, two inclined borings were drilled under each reactor footprint at different bearings. No joints or fractures were noted in the core from these borings.

2.5.1.2.5.3 Deformational Zones

No deformational zones have been found near the CPNPP site. Joints and lineaments observed in aerial photographs and in the field are randomly oriented and have no associated offsets. Virtually planar bedding is undisturbed by folding, fractures, faults, or shear zones.

2.5.1.2.5.4 Karst Zones of Dissolution and Subsidence

There is no evidence of active karst conditions and related subsidence within the CPNPP site or in the surrounding area. A review of recent and historic aerial photography, field reconnaissance, and geotechnical drilling results for the CPNPP Units 3 and 4 site was conducted to evaluate the presence of potential karst features.

The Glen Rose contains appreciable amounts of calcareous material that may be susceptible to dissolution. Aerial photography of the area indicates that the bedding of the Glen Rose and overlying units exposed in surrounding drainage cuts is extensively uniform in thickness and nearly horizontal over large distances (Figure 2.5.1-230). Field reconnaissance of outcrops and large exposures do not indicate any significant zones of active dissolution. Some minor flexures on the order of less than 3 ft of vertical relief over less than 100 ft lateral distance were noted in limited exposures of the Glen Rose south of the SCR dam as discussed in Subsection 2.5.1.2.5.1. Healed joints in the Glen Rose limestone and overlying Paluxy were noted as being calcite-filled.

A review of the geotechnical borings drilled in the CPNPP Units 3 and 4 area indicates a thin, uniform weathering profile that mantles the site. Bedding below the weathered zone is horizontal, indicating no evidence of karst features. Minor vugs were noted in a few of the boring logs. No significant loss of drill fluid was noted in the geotechnical borings, indicating that no solution cavities were encountered. Results of in situ packer tests indicate hydraulic conductivities within the Glen Rose limestone beds are low. Petrographic analysis indicates that the

limestone of the Glen Rose is tightly compacted, and no indications of secondary alteration were noted.

2.5.1.2.5.5 Groundwater

Withdrawal of groundwater from aquifers beneath the site does not pose a risk of subsidence at the current withdrawal rates. A discussion of groundwater withdrawals for the site is provided in Subsection 2.4.12. The strata underlying the site are cemented limestones and indurated shales of the Glen Rose Formation underlain by semi-indurated to indurated sandstones and silty sandstones of the Twin Mountains Formation. The uppermost potable aquifer beneath the site is within the Twin Mountains Formation. The measured data from the regolith and upper Glen Rose Formation monitoring wells within the CPNPP Units 3 and 4 area suggest that the piezometric levels range between about elevation 775 ft and 858 ft, with a number of wells remaining dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site. A discussion of groundwater conditions for the site is provided in Subsection 2.4.12.The low compressibility of these materials and the lithified nature of the overlying Glen Rose Formation are not conducive to settlement caused by groundwater draw-down.

Perched water is noted within the Glen Rose Formation and may be encountered during excavation for CPNPP Units 3 and 4; however, the extent and volumes are anticipated to be low due to the low hydraulic conductivity of the Glen Rose Formation and the lack of extensive joints and fractures.

2.5.1.2.5.6 Reservoir Effects

No adverse effects due to the construction of man-made reservoirs in the CPNPP area, including SCR, Lake Granbury, and Lake Whitney, have been noted (Figure 2.5.1-218). The SCR is located immediately to the north of the CPNPP Units 3 and 4 site. Groundwater conditions are discussed in Subsection 2.4.12.

No reservoir-induced earthquakes have been noted since the construction of SCR and other large reservoirs in the site area. This absence may be attributed to the low hydraulic conductivity of the subsurface materials as well as to lack of faults or planes of weakness that may respond to increased pore fluid pressure from the downward migration of water from the reservoirs.

The pool elevation of SCR is 775 ft msl. The excavation for CPNPP Units 3 and 4 extends to approximately elevation 782 ft msl to facilitate removal of a shale layer, so that the Category 1 structures are directly founded on a limestone layer or fill concrete, at elevation 782 ft msl, as discussed in Subsection 2.5.4. There are two areas of undocumented artificial fill near the CPNPP Units 3 and 4 area, as shown on (Figure 2.5.1-218). Groundwater within these fill areas is in communication and hydrostatic equilibrium with SCR, as indicated from monitoring wells. No Category 1 or critical structures are located over these areas.

2.5.1.2.5.7 Slope Stability

Slope stability is not considered a hazard to the site. The nearest slopes exist immediately north of the CPNPP Units 3 and 4 area along the SCR. These slopes will be re-graded as part of the general site grading plan. A detailed discussion of the slope stability analysis is presented in Subsection 2.5.5.

2.5.1.2.5.8 Unrelieved Residual Stresses in Bedrock

The regional tectonic setting, discussed in Subsection 2.5.1.1.4, indicates that no active tectonics exist in the region surrounding the CPNPP site. No active faults are noted within 25 mi of the CPNPP site, and the overlying Cretaceous section truncates several nonactive Paleozoic faults, which indicates that no reactivation has occurred in the past 65 million years. Subsection 2.5.1.2.5.10 discusses issues related to potential reactivation of faults due to man-induced activities.

2.5.1.2.5.9 Geologically Hazardous Materials

No geologically hazardous materials, such as expansive soils or reactive minerals (e.g., gypsum or anhydrite) of appreciable amounts, exist within 25 mi of the site and, thus, are not considered a hazard.

2.5.1.2.5.10 Effects of Man's Activities

No significant external hazards such as surface or subsurface mining operations exist within the site area, except for aggregate mines. These mining operations do not pose a hazard to the site. Three surface aggregate mining operations were identified within about 4 mi of the site. Each of the operations consisted of surface strip mining and aggregate processing. Mining operations are primarily focused within the Paluxy Formation for gravel and the underlying Glen Rose Formation for dimension stone. However, the CPNPP site is located within the Fort Worth Basin, which is a major hydrocarbon reservoir. Activity related to oil and especially gas production has significantly increased in the last several years within the site vicinity and area.

2.5.1.2.5.10.1 Oil and Gas Production Related Activities

Production of gas within the Barnett Shale, which exists at a depth of greater than 5000 ft in the site area, has significantly increased in the last several years. Other Paleozoic units in the Fort Worth Basin have been producing oil and gas for several decades (Reference 2.5-347). Because of this increase in gas production activity in the region, a study of potential hazards was performed by Professors Ellen Rathje and Jon Olson, University of Texas-Austin, and is summarized herein.

Figure 2.5.1-228 shows gas well locations demarked by production quantities in the Fort Worth Basin and also shows that current local production is concentrated in Hood County, particularly to the northeast of the site. Figure 2.5.1-229 shows

the increase in gas production of the Barnett Shale for the period 1993–2005. Figure 2.5.1-228 is intended to show the current gas production distribution in the region surrounding the CPNPP site and is not intended to forecast future production or distribution, as this is permitted through the Texas Railroad Commission. Similarly, Figure 2.5.1-229 is intended to show the history of Barnett Shale gas production within the Newark East Field of the Fort Worth Basin and does not represent the production volumes shown for the area on Figure 2.5.1-228. It is not intended to forecast production in the vicinity of the CPNPP site. Table 2.5.1-204 provides a summary of currently active or abandoned oil, gas and disposal wells within about 10 mi surrounding the CPNPP site, including information respective to the producing formation, depth, status, and location.

Two primary activities are associated with gas production in the region. First, fluid extraction involves the long-term production of gas and associated fluids from a gas-producing unit such as the Barnett Shale. This typically requires hydraulic fracturing of the production zone (namely the Barnett Shale) to facilitate gas extraction. Second, fluid injection involves the disposal of water or liquid waste generated during gas production back into deep geologic units as well as fluid injection related to hydrofracturing or formation stimulation. Potential issues related to these activities are identified and discussed in the following subsections. In the following discussion, fluid injection and extraction activities may apply to various techniques and activities and are specified as required for the discussion. In the following discussion, fluid injection and extraction activities may apply to various techniques and activities and are specified as required for the discussion.

2.5.1.2.5.10.1.1 Potential Hazards Related to Hydraulic Fracturing

The potential hazards related to hydraulic fracturing for gas production include changes to the rock properties and induced seismicity. These issues are discussed below and are determined not to present a hazard to the CPNPP site.

2.5.1.2.5.10.1.2 Changes to Rock Properties Related to Hydraulic Fracturing

Because of the low porosity of the Barnett Shale, enhanced production techniques are required to achieve enough gas production to make the process economically feasible. Thus, hydraulic fracturing is commonly employed. Hydraulic fracturing involves injecting fluid into the gas-bearing strata to induce fractures that allow the gas to flow more easily to the production well. These induced fractures are on the order of 0.1 to 0.25 in thick and are filled with sand or other high-permeability materials (called proppant) so that they remain open and can conduct the gas to the well.

A hydraulic fracture is idealized as a single vertical plane of hundreds to a few thousand ft in total length, hundreds of ft in height, and a fraction of an inch in width. The actual size of a hydraulic fracture will largely depend on the amount of fluid and sand injected, the permeability of the formation, and the variation of the

minimum horizontal stress over depth (which determines whether or not the fracture is contained in height growth). Hydraulic-fracture diagnostic data from microseismic monitoring in the Barnett Shale suggests that hydraulic fracture growth is more complex than this simple idealization, with multiple strands forming as the propagating hydraulic fracture interacts with pre-existing natural fractures (Reference 2.5-348). Although there is no direct observation of subsurface hydraulic fracture geometry for the Barnett Shale, it is presumed that the created fractures approximate an orthogonal grid-like pattern (References 2.5-349 and 2.5-350), with minimum spacing on the order of 50 ft between fracture zones (which may be narrow vertical corridors of closely spaced fractures).

Rock fractures generally reduce the wave propagation velocities of rock (Reference 2.5-351). Leucci and De Giorgi (Reference 2.5-352) showed that for a fracture spacing of about 0.5 m and high-frequency waves (> 1 kHz), the shear wave velocity was reduced by about 30 percent for a sedimentary rock specimen under atmospheric pressure. Fratta and Santamarina (Reference 2.5-353) provide a relationship (Backus' average) that predicts the wave velocity of fractured rock (with the fractures filled with a material distinct from the intact rock) based on the characteristics of the rock and fracture infill material ratio. The velocity of the fractured rock is a function of the fracture ratio (equal to the fracture thickness / spacing between fractures), the velocity of the intact rock, the velocity of the fracture infill material, and the density of the intact rock and fracture infill material.

The gas production from the Barnett Shale, which is a low porosity (n < 5 percent; References 2.5-354 and 2.5-355) and very stiff unit (shear wave velocity about 8,000 ft/s, G about 2 x 10^6 pounds per square in (psi), E about 5 x 10^6 psi) is located about 5,000 ft below the surface and is approximately 250 ft thick, based on well data in the region and geologic maps. Considering the intact shear wave velocity of Barnett Shale (8,000 ft/s) and assuming a shear wave velocity of 800 ft/s for the proppant filling the fractures and a fracture ratio of 0.0004 (fracture thickness = 0.25 in, fracture spacing = 50 ft = 600 in), the model presented in Fratta and Santamarina (Reference 2.5-353) predicts less than a 5 percent reduction in shear wave velocity due to the fracturing.

Because of the broad spacing between fracture zones and the fact that they are filled with proppant and thus presumed to have highly resolved compressive stress across them, it is unlikely that the induced hydraulic fractures will substantially alter the velocity structure of the formation. The vibratory ground motion analysis for the site, as discussed in Subsection 2.5.2.5, modeled the deep layers, which include the Barnett Shale, with a coefficient of variation of about 31 percent to envelope the epistemic and aleatory uncertainty.

2.5.1.2.5.10.1.3 Induced Seismicity from Hydraulic Fracturing

Hydraulic fracturing for gas production in the vicinity of the CPNPP site may result in induced seismicity; however, it is expected to be below measurable levels and thus of no danger to surface structures at the CPNPP site. Albright and Pearson (Reference 2.5-356) reported magnitudes on the order of -6 to -2 for micro-

earthquakes associated with injection-related fracturing for hot dry rock applications, while Rutledge and Phillips (Reference 2.5-357) reported moment magnitudes between -1.5 and -3.5 for hydrofracturing in the Barnett Shale. Conventional hydraulic fracture treatments as conducted for Barnett Shale gas production are expected to have the same level of seismicity. Finally, there are no known instances of hydraulic fracturing causing a damaging earthquake (Reference 2.5-358).

2.5.1.2.5.10.2 Potential Issues Related to Fluid Extraction/Injection

Oil and gas production in the Fort Worth Basin requires both the extraction and injection of fluids. Extraction generally relates to the production of the oil and or gas as well as other formation fluids that require separation and disposal. Fluid injection operations consist of the disposal of associated water produced with oil and gas or hydraulic fracture water recovered in the well clean-up process.

Three potential issues have been identified related to long-term fluid extraction or injection in the CPNPP area: (1) changes to the rock properties due to the gas production, (2) induced compaction/subsidence due to gas production, and (3) induced seismicity due to gas production and fluid injection. These issues are discussed below.

2.5.1.2.5.10.2.1 Changes to Rock Properties Related to Fluid Extraction/Injection

Gas extraction changes the effective stresses in the gas-bearing strata due to a reduction in fluid pressure. This increase in stress can cause compaction (i.e., reduction in porosity/void ratio), as well as permanently increase the shear modulus/shear wave velocity of the rock. However, as mentioned above, because the Barnett Shale is very low porosity and has a large stiffness, it is unlikely that rock properties will change appreciably over the production life of the reservoir.

2.5.1.2.5.10.2.2 Induced Compaction/Subsidence Related to Fluid Extraction

The compaction of the gas-bearing strata due to gas extraction can cause subsidence at the ground surface. The magnitude and extent of this surficial subsidence is affected by various factors, such as the depth of the gas-bearing strata, the thickness of the strata, the properties of the strata, production rates, and the details of the extraction process. Using the properties for the Barnett Shale derived from nearby log data and assuming a 207-bar (3,000-psi) reduction in fluid pressure in the reservoir (the expected maximum drawdown), the computed elastic compaction for the 250-ft-thick shale layer was on the order of 0.2 ft. Because this estimated amount of compaction is small and the Barnett Shale reservoir is fairly deep (5500 ft), the associated subsidence propagated to the ground surface is expected to be negligible.

2.5.1.2.5.10.2.3 Induced Seismicity Related to Fluid Extraction/ Injection

Small earthquakes (magnitude less than about 5) can be induced by fluid (gas, oil, or water) extraction (References 2.5-359, 2.5-360, and 2.5-361) or fluid injection (References 2.5-358 and 2.5-362). There are no cases of human actions causing large earthquakes within the site region (References 2.5-359, 2.5-269, 2.5-474, 2.5-475). The mechanism for induced seismicity due to fluid injection is the reduction in effective stress (due to increased pore pressures) and subsequent weakening of faults (Reference 2.5-358). The most notable example of seismicity induced by fluid injection is the seismicity associated with waste fluid injection at the Rocky Mountain Arsenal near Denver, Colorado, in the mid 1960s. Some 35 earthquakes greater than m_b 3, and 3 earthquakes greater than m_b 5, occurred over a 5-year period (Reference 2.5-363). The rate of injection in this area was approximately 50 to 60 million gallons per year (mgy) over a three-year period. Only one example of injection-induced seismicity in Texas was identified: the earthquake sequence associated with the Cogdell oil field of West Texas (Reference 2.5-269). These earthquakes occurred in the Midland Basin in an area of fluid injection associated with secondary oil recovery (waterflooding). The net fluid injection in this area was 250 to 500 mgy between 1960 and 1970, and was increased to 500 to 1,000 mgy between 1970 and 1977. From 1974 to 1982, a total of 17 earthquakes greater than m_b 2 occurred, including a m_b 4.3 earthquake in 1978. This earthquake induced minor damage and the maximum MMI defined was reported as V (Reference 2.5-359). It is important to note that the injection rates at Cogdell are an order of magnitude greater than the rates injected at Rocky Mountain Arsenal, yet the induced rate of seismicity and the size of events were considerably smaller.

Water injection may be used for secondary oil recovery (waterflooding) or waste disposal. Because of its large extent, the Ellenburger Limestone, which is stratigraphically below the Barnett Shale, is a prime target for injection in the Fort Worth Basin. Such injection may increase fluid pressure in the subsurface, reducing the effective stress on faults and promoting slip. As mentioned above, there are documented examples of injection-induced seismicity. However, Davis and Pennington (Reference 2.5-269) find that even though modeling suggests that reported injection pressures in oil and gas fields under water injection in Texas should cause fault slip, only one field (Cogdell) was known to have seismic activity. Their conclusion to explain the apparent discrepancy between predicted fault failure and known seismicity was that much of the failure actually may be aseismic. In addition to changing the stress state, the injected fluid is suspected to weaken the faults to such an extent that they creep to relieve shear stress.

The mechanism for induced seismicity due to fluid extraction is not immediately known because the removal of fluid decreases pore pressures and increases effective stresses, a change that is generally expected to stabilize faults because it restrains slip (Reference 2.5-361). However, it is expected that poro-elastic changes in the in situ stress state are the causal mechanism for induced seismicity due to fluid extraction (References 2.5-361 and 2.5-364). The most

notable location of seismicity induced by gas or oil extraction is the Lacq gas field in France, which experienced 44 earthquakes with $M_l > 3$ and 4 events with $M_l > 4$ over a twenty year period (References 2.5-365 and 2.5-366).

Some earthquakes in south-central Texas have been related to local gas and/or oil extraction. The largest of these earthquakes was the April 9, 1993, m_bLg 4.3 event that occurred 50 mi south of San Antonio, with reported MMIs as high as VI (Reference 2.5-367). The most significant damage occurred at the Warren Petroleum Plant, and included cracking of reinforced concrete foundation blocks, failure of one pipe connection, damage to steel bolts, and horizontal movement on the order of an inch. Frohlich and Davis (Reference 2.5-359) estimate that of the 130 earthquakes that have occurred over the last 150 years in Texas and have been felt by residents, only 22 were induced by gas or oil production. Additionally, it is important to note that there has been significant gas and oil production within the state of Texas over the last century, including within the Fort Worth Basin, yet the seismicity rate remains relatively low. In particular, the Texas seismicity catalog generated by William Lettis & Associates, Inc., for the time period 1627 to 2006 shows no earthquakes greater than m_b 3 within the Fort Worth Basin. Subsection 2.5.2 contains a discussion on seismic activity in the region and the development of an earthquake catalog update.)

2.5.1.2.5.10.3 Probabilistic Seismic Hazard Analysis (PSHA) Considerations for Induced Seismicity

Current procedures used to perform PSHA for nuclear facilities incorporate background seismicity zones (Reference 2.5-335). The earthquake recurrence models for these background seismicity zones are derived from the observed earthquakes with body wave magnitudes (m_b) greater than 3.0. However, the minimum m_b magnitude that is considered to be of engineering significance is 5.0, and smaller magnitudes are not considered in the PSHA analysis to derive design ground motions.

It is very uncommon for induced earthquakes to exceed m_b 5.0. However, some of the earthquakes induced by injection at the Rocky Mountain Arsenal were larger than 5.0, so it is important to consider what characteristics might be favorable to generating earthquakes larger than 5.0. In the case of the Rocky Mountain Arsenal, injection took place in naturally fractured, otherwise non-porous, Precambrian crystalline rock (Reference 2.5-368). In such a situation, where there is little to no pressure diffusion into the pore space, injected fluid would be confined strictly to flow within the natural fractures, and thus could reduce effective stresses over very large fractures areas. Larger magnitude earthquakes require large slip areas, so injection into naturally fractured crystalline rock might reasonably be expected to result in larger induced earthquake magnitudes. Although the Barnett Shale and the Ellenburger Limestone of the Fort Worth Basin are competent sedimentary rocks, the crystalline rocks of Colorado are much stronger and allow for greater build-up of stress that can cause larger earthquakes. Finally, Gibbs, et al. (Reference 2.5-363) suggest that the Rocky

Mountain Arsenal injection was releasing built-up tectonic stress locked in the rock. Because the Denver area is one of more recent tectonic activity (the Laramide orogeny, ended about 25 million years ago, and ongoing post-Laramide Uplift) than the Fort Worth Basin (last major tectonic event was the Ouachita orogeny, which ended about 300 million years ago), the shear stress magnitudes and active tectonic strain rates are expected to be larger in Colorado than in North Texas, and consequently this may limit potential earthquake magnitude.

On the basis of information collected, it appears that any earthquake induced by gas production or fluid injection in the Fort Worth Basin would not be larger than m_b 5.0. Therefore, the enhanced seismicity that potentially would be induced would not need to be taken into account in the PSHA.

2.5.2 Vibratory Ground Motion

CP SUP2.5(2) Add the following after the content of DCD Section 2.5.2.

This subsection provides a detailed description of vibratory ground motion assessments, specifically the criteria and methodology for establishing the Ground Motion Response Spectra (GMRS) and Foundation Input Response Spectra (FIRS) for the Comanche Peak Nuclear Power Plant Units 3 and 4 (CPNPP Units 3 and 4). The development of the GMRS for CPNPP Units 3 and 4 follows a methodology consistent with the approach recommended in Regulatory Guide (RG) 1.208 and, therefore, satisfies the requirements set forth in Section 100.23, "Geologic and Seismic Siting Criteria," of Title 10, Part 100, of the Code of Federal Regulations (10 CFR 100), "Reactor Site Criteria." This subsection begins with a review of the approach outlined in RG 1.208 and is followed by these subsections:

- Seismicity (Subsection 2.5.2.1)
- Geologic and Tectonic Characteristics of the Site and Region (Subsection 2.5.2.2)
- Correlation of Earthquake Activity with Seismic Sources (Subsection 2.5.2.3)
- PSHA and Controlling Earthquake (Subsection 2.5.2.4)
- Seismic Wave Transmission Characteristics of the Site (Subsection 2.5.2.5)
- Ground Motion and Site Response Analysis (Subsection 2.5.2.6).

RG 1.208 provides guidance on methods acceptable by the Nuclear Regulatory Commission (Reference 2.5-369) for satisfying the requirements of developing the site-specific GMRS, which in turn represents the first step in developing the Safe Shutdown Earthquake (SSE) ground motion levels as a characterization of the seismic hazard at CPNPP Units 3 and 4. The process outlined in RG 1.208 for determining the GMRS includes:

- The geological, geophysical, seismological, and geotechnical investigations of the site and site region, including the identification of seismic sources significant to seismic hazard at the site.
- The procedures for performing a Probabilistic Seismic Hazard Analysis (PSHA) and deaggregating mean hazard.
- Characterization of the seismic wave transmission characteristics of the site.

- Development of the performance-based site-specific earthquake ground motion.

RG 1.208 states that an acceptable starting point for developing probabilistic seismic hazards calculations for a Combined Operating License (COL) is a PSHA model that has been reviewed and accepted by the NRC. This COL application uses the accepted PSHA model developed by the Electric Power Research Institute Seismicity Owners Group (EPRI-SOG) in the 1980s (Reference 2.5-369) as the starting point for determining the GMRS for CPNPP Units 3 and 4. The EPRI-SOG PSHA model (Reference 2.5-369) was developed as part of a comprehensive study of seismic hazard at nuclear power plants in the Central and Eastern United States (CEUS). The study involved a comprehensive compilation of geological, geophysical, and seismological data for the CEUS that was used by six independent and multi-disciplinary Earth Science Teams (ESTs) of experts in geology, seismology, and geophysics to develop seismic source characterizations for the CEUS that explicitly incorporated uncertainty in source geometry, earthquake recurrence, and earthquake magnitude. The seismic sources developed in the EPRI-SOG model were then used in a PSHA of the ground motions at nuclear power plants in the United States (U.S.) (Reference 2.5-370). This COL application uses the seismicity, seismic source models, ground motion equations, and PSHA methodology of the EPRI-SOG study (References 2.5-369 and 2.5-370) as a starting point for the PSHA at CPNPP Units 3 and 4. A more detailed discussion of the suitability of the EPRI-SOG seismic sources is presented in Subsection 2.5.2.2.1.

Following the guidance of RG 1.208, a comprehensive review of new geological, geophysical, and seismological data developed following the EPRI-SOG study was conducted to determine the need for updating the EPRI-SOG source models for CPNPP Units 3 and 4. Post-EPRI-SOG site and regional geologic and geophysical data are discussed in Subsection 2.5.1, and post-EPRI-SOG site and regional seismological data are presented in Subsection 2.5.2.1. Additionally, post-EPRI-SOG seismic source characterizations for sources relevant to CPNPP Units 3 and 4 are reviewed in Subsection 2.5.2.2.2. This information is reviewed to update some EPRI-SOG source zones and develop new source characterizations for CPNPP Units 3 and 4 in Subsection 2.5.2.4.2. Only those new source characterizations determined through a screening study to be significant to hazard at CPNPP Units 3 and 4 are included in the final calculation for the GMRS. Subsection 2.5.2.5 also describes the use of updated ground motion equations and the use of Cumulative Absolute Velocity (CAV) filtering to limit the effects of low-magnitude, non-damaging earthquakes on the GMRS.

Also following guidance provided in RG 1.208, the horizontal GMRS developed in Subsection 2.5.2.6 was calculated using a performance-based, risk-consistent method based on the ASCE/SEI Standard 43-05, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities (Reference 2.5-371) that takes into account soil amplification factors determined using Approach 3 of NUREG/CR-6769 and soil properties presented in Subsection 2.5.2.5. The method specifies the level of conservatism and rigor in the seismic design process

such that the performance of structures, systems, and components of the plant achieve a uniform seismic safety performance. Subsection 2.5.2.6 also describes the development of the vertical GMRS through the scaling of the horizontal GMRS by frequency-dependent vertical-to-horizontal response spectra and describes development of the FIRS for the four elevations at which seismic category I structures at CPNPP Units 3 and 4 will be founded.

2.5.2.1 Seismicity

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.1 with the following.

The EPRI-SOG PSHA methodology used as the basis for determining the GMRS at CPNPP Units 3 and 4 primarily relies on the analysis of historical seismicity within the CEUS to estimate seismicity rate and relative magnitude recurrence parameters (i.e., activity rates and Gutenberg-Richter b-values) for seismic sources defined by each of the ESTs (Reference 2.5-369). As part of the EPRI-SOG study, a seismicity catalog was developed for the CEUS spanning the years 1627 through the beginning of 1985. The resultant catalog is briefly reviewed in Subsection 2.5.2.1.1. As part of evaluating the impact of post-EPRI-SOG information on seismic source characterizations relevant to CPNPP Units 3 and 4, an updated seismicity catalog was developed that extends beyond the site region. Subsection 2.5.2.1.2 describes the development of this catalog. The seismicity catalog used for CPNPP Units 3 and 4 is the combination of the original EPRI-SOG catalog and the updated catalog developed here. Recent and historical earthquakes with the potential to affect CPNPP Units 3 and 4 are discussed in Subsection 2.5.2.1.3.

2.5.2.1.1 Seismicity Catalog Used in EPRI-SOG Seismic Hazard Analysis

The seismicity catalog used in the EPRI-SOG study (Reference 2.5-370) extends from the Rocky Mountain front to beyond the Atlantic coastline and from the U.S.-Canada border to the Gulf of Mexico, well beyond the extent of the CPNPP Units 3 and 4 site region. The EPRI-SOG study spent considerable effort in ensuring that the catalog is complete throughout the historical record to the time of the catalog compilation (early 1985) in that all instrumental earthquakes and significant historical earthquakes were included (References 2.5-369 and 2.5-370). In addition, all duplicate events were removed from the catalog, all non-earthquakes (e.g., explosions) were removed from the catalog, only main events of earthquake clusters were included in the catalog, and all event magnitudes were converted to a uniform estimate (Emb) of body-wave magnitude (m_b).

Given the characteristics of the seismicity catalog developed for the EPRI-SOG study (References 2.5-369 and 2.5-370), the EPRI-SOG catalog meets the requirement of RG 1.206 that a COL applicant shall “provide a complete list of all historically reported earthquakes that could have reasonably affected the region surrounding the site, including all earthquakes of modified Mercalli intensity

(RG 1.208) greater than or equal to IV or of m_b greater than or equal to 3.0 that have been reported within 200 mi of the site" up until 1985.

2.5.2.1.2 Updated Seismicity Catalog

The updated seismicity catalog for the years 1985 to 2006 is developed to:

- Satisfy the requirements of RG 1.206 regarding the reporting of earthquakes within 200 mi of the site, and
- Assist in the evaluation of the existing EPRI-SOG source model to adequately describe seismic hazard at CPNPP Units 3 and 4.

Spatially, the updated catalog extends over an update region defined as the area from 28° to 38° north latitude and 93° to 104° west longitude. Figure 2.5.2-201 shows the site, the site region, the extent of the updated catalog, earthquakes from the final updated catalog, and earthquakes from the EPRI-SOG catalog.

The updated catalog is based on a compilation of the following catalogs:

Advanced National Seismic System (ANSS) Catalog

The ANSS catalog was searched on February 9, 2007, for all earthquakes within the update region occurring between January 1, 1985, and December 12, 2006, resulting in a catalog of 231 events (Reference 2.5-372). The ANSS catalog is used as the base catalog for the CPNPP catalog update.

National Earthquake Information Center (NEIC) Catalog

The NEIC Preliminary Determination of Epicenters (PDE) catalog was searched on March 16, 2007, for all earthquakes within the update region occurring between January 1, 1985, and December 12, 2006, resulting in a catalog of 217 events (Reference 2.5-373). The NEIC catalog is used to supplement the ANSS catalog.

Oklahoma Geological Survey (OGS) Catalog

The OGS (Reference 2.5-374) operates ten seismograph stations in the state of Oklahoma and develops a local catalog with lower event-detection thresholds than catalogs generated from regional seismograph networks. The OGS archives the local catalog as separate annual files online at the Oklahoma Geological Survey Observatory (http://www.okgeosurvey1.gov/level2/okeqcat.index.html). Twenty-two annual files covering the years 1985 through 2006 were downloaded from the site on March 13, 2007, resulting in a combined 1327 events (Reference 2.5-374). The compiled OGS catalog is used to supplement the ANSS catalog in Oklahoma.

New Mexico Institute of Mining and Technology (NMT) Catalog

The NMT Seismological Observatory operates 17 seismograph stations in the state of New Mexico, some of which provide coverage of eastern New Mexico and west Texas. The NMT catalog is archived online and contains 768 events between 1985 and 1998 (Reference 2.5-375). The NMT catalog is used to supplement the ANSS catalog in west Texas.

Center for Earthquake Research and Information (CERI) Catalog

The CERI (Reference 2.5-376) at the University of Memphis was searched on March 14, 2007, for all earthquakes within the update region occurring between January 1, 1985, and December 12, 2006, resulting in a catalog of 20 events (Reference 2.5-376). The CERI earthquake catalog is used to supplement the ANSS catalog.

The above catalogs are compiled into a single catalog, and the updated catalog for CPNPP Units 3 and 4 is derived from this compiled catalog through the following steps:

- Duplicates in the catalog are removed by comparing origin time and location. For duplicate events, the event record from the source with the largest magnitude estimate is kept to ensure conservatism in earthquake magnitude reporting.

- Earthquakes occurring outside the specified time period (January 1, 1985, and December 31, 2006) are excluded.

- Earthquakes occurring outside the update region are excluded.

- Best estimate body-wave magnitudes (Emb) are determined for all events following the EPRI-SOG methodology (References 2.5-340 and 2.5-335). Within this methodology, reported m_b magnitudes for earthquakes are taken as equivalent to Emb magnitudes and, for other reported magnitudes, Emb magnitudes are determined using the relationships presented in Table 4-1 of EPRI (Reference 2.5-340):

  $Emb = 0.253 + 0.907 \cdot M_d$ Equation 1

  $Emb = 0.655 + 0.812 \cdot M_L$ Equation 2

  $Emb = 2.302 + 0.618 \cdot M_S$ Equation 3

  where M_d is duration or coda magnitude, M_L is local magnitude, and M_s is surface-wave magnitude. For these events the final Emb magnitude for an event is taken as the largest estimated Emb magnitude.

- All events with Emb less than 3.0 are excluded from the catalog, and

- Uniform m_b magnitudes (Rmb) are determined for all events for use in estimating seismicity parameters as outlined in the EPRI-SOG

methodology (References 2.5-340 and 2.5-335). Rmb is calculated using Equation 4-2 from EPRI (Reference 2.5-340):

$$Rmb = Emb + (1/2) \cdot \ln(10) \cdot b \cdot Smb^2 \qquad \text{Equation 4}$$

where Smb is the standard deviation of m_b. Values of Smb are estimated from the original EPRI-SOG catalog.

Table 2.5.2-201 presents the 97 events of the updated catalog for CPNPP Units 3 and 4. This updated catalog is used in conjunction with the EPRI-SOG catalog to determine seismicity parameters following the EPRI-SOG methodology (References 2.5-369 and 2.5-370). It should be noted that the updated catalog does vary from the EPRI-SOG catalog in that the updated catalog has not been declustered to remove dependent events. Therefore, seismicity rates determined using the updated catalog may be higher (i.e., more conservative) than if the catalog had been declustered. The combination of the updated catalog and the original EPRI-SOG catalog present a complete description of mainshock seismicity for CPNPP Units 3 and 4 through December 31, 2006.

2.5.2.1.3 Recent Earthquakes and Historical Seismicity

The updated seismicity catalog described in Subsection 2.5.2.1.2 and the original EPRI-SOG seismicity catalog described in Subsection 2.5.2.1.1 are shown in Figure 2.5.2-201 and Figure 2.5.2-202, respectively. These figures show that there is no significant difference in the spatial pattern of seismicity within the update region between the EPRI-SOG catalog and the updated catalog. Subsection 2.5.2.4.2.1 provides a quantitative comparison of seismicity rates and shows that there is also no significant difference between the two catalogs. As noted in the EPRI-SOG study, the most seismically active region within the extent of Figure 2.5.2-201 is the New Madrid Seismic Zone (NMSZ) in the northeast section of the figure, well outside of the CPNPP Units 3 and 4 site region. Seismicity within the NMSZ is discussed in more detail in Subsection 2.5.2.4.2.3.1 and in Subsection 2.5.1.1.4.3.7.3. Within the site region, the largest concentration of earthquakes occurs in Oklahoma and along the trend of the Southern Oklahoma Aulacogen (Figure 2.5.1-208 and Figure 2.5.2-202). The association of seismicity with the Southern Oklahoma Aulacogen is discussed in more detail in Subsection 2.5.2.3. No earthquakes with Emb > 3.0 have occurred within 50 mi of the site (Figure 2.5.2-202).

Also, there is no evidence of historical or modern earthquakes causing earthquake-induced geologic failure within the site region. The Holocene Meers fault scarp, discussed in Subsection 2.5.1.1.4.3.6.1.1, is the only fault with paleoseismic evidence of earthquake-induced geologic failure within the site region.

2.5.2.1.3.1 Recent Earthquakes

No significant earthquakes, defined as earthquakes with an impact on the seismic hazard at CPNPP Units 3 and 4 or seismic source characterization of sources relevant to CPNPP Units 3 and 4, have occurred within the site region since the end date of the EPRI-SOG seismicity catalog (i.e., post-1984). For example, the largest post-1984 earthquake within the site region is the September 6, 1997, Emb 4.5 earthquake in south-central Oklahoma, approximately 180 mi from CPNPP Units 3 and 4. However, four earthquakes have occurred outside of the site region with relevance to seismic hazard at CPNPP Units 3 and 4 and seismic source characterizations for CPNPP Units 3 and 4. Two of these earthquakes, the January 2, 1992, Emb 5.0 in southeast New Mexico and the April 14, 1995, Emb 5.7 Alpine earthquake in west Texas (Figure 2.5.2-201), are documented within the updated seismicity catalog (see Subsection 2.5.2.1.2). The other two events, the February 10, 2006, Ms 5.3 and September 10, 2006 earthquakes in the Gulf of Mexico (Reference 2.5-377), are well outside the update region (Figure 2.5.2-205) and are not in the updated catalog. Each of these events is discussed below.

<u>January 2, 1992, Emb 5.0 Rattlesnake Canyon, New Mexico</u>

The January 2, 1992, Emb 5.0 earthquake near Rattlesnake Canyon, New Mexico (Table 2.5.2-201) was felt over an area of approximately 440,000 km^2 and had a maximum Modified Mercalli Intensity of V (Reference 2.5-378). CPNPP Units 3 and 4 are outside of the felt area as defined by Frohlich and Davis (Reference 2.5-378), and no damage was reported from this earthquake within the felt area (Reference 2.5-378). A focal mechanism of the event determined by Sanford, et al. (Reference 2.5-379) shows that the event was characterized by thrust motion with an east-west compression axis. The event occurred within the central basin platform of the Permian basin, a region of active hydrocarbon exploration. Exploration within the basin produces some seismicity, but it is unknown if this earthquake is of tectonic or man-induced origin (References 2.5-379 and 2.5-380).

<u>April 14, 1995, Emb 5.7 Alpine, Texas</u>

The April 14, 1995, Emb 5.7 earthquake near Alpine, Texas, (Table 2.5.2-201) was felt over an area of approximately 760,000 km^2 and had a maximum intensity of MMI VI (Reference 2.5-378). CPNPP Units 3 and 4 are within the MMI I to III intensity isoseismal region defined by Frohlich and Davis (Reference 2.5-378). Near the epicenter, reported damage includes broken gas mains, cracked walls, and broken windows (Reference 2.5-378). Frolich and Davis (Reference 2.5-378) report that the earthquake was felt in Dallas, Texas, only in high-rise buildings. No known felt reports come from the region immediately surrounding CPNPP Units 3 and 4. A focal mechanism of the event determined by the Global Centroid Moment Tensor Project shows that the event was an earthquake with normal faulting motion with a tensile axis oriented approximately north-northeast (Reference 2.5-317). The event occurred along the eastern boundary of the Rio Grande Rift

(RGR) (Reference 2.5-318), an extensional tectonic province characterized by active seismicity related to normal faulting (see discussion in Subsection 2.5.1.1.4.3.7.1). Research has shown that the RGR influences the upper crustal state of stress well eastward of the topographically defined RGR (see discussion in Subsection 2.5.1.1.4.3.7.1). Partly based on these observations, some researchers believe that this earthquake is related to RGR tectonics. For the CPNPP Units 3 and 4 PSHA, this earthquake is interpreted as related to RGR tectonics.

<u>February 10, 2006, Ms 5.3 Green Canyon, Gulf of Mexico</u>

The February 10, 2006, Ms 5.3 event in the Gulf of Mexico is well outside the update region (Reference 2.5-377) and is not in the updated catalog. The event was felt in coastal Louisiana, Texas, and Florida and had a maximum intensity of MMI III (Reference 2.5-381). The earthquake occurred along the Sigsbee escarpment off Louisiana. Nettles (Reference 2.5-382) has interpreted this event as a gravity-driven landslide based on the lack of high-frequency energy in the waveforms, slow rise time, preliminary focal mechanism determinations, and the location of the event on the Sigsbee escarpment. Preliminary conclusions of Dellinger, et al. (Reference 2.5-383) also support this interpretation, but Dellinger, et al. (Reference 2.5-383) admit that neither a consensus nor conclusive interpretation of the event mechanism has been determined. The implication of the “landslide” interpretation is that large mass sliding events along the Sigsbee escarpment may be detectable on local and regional seismic networks. However, no other earthquakes within the Gulf of Mexico have been attributed to this mechanism, and other independent researchers have not confirmed the landslide mechanism for the February 10 event.

<u>September 10, 2006 Mw 5.8, Gulf of Mexico</u>

The September 10, 2006, Mw 5.8 event in the Gulf of Mexico is well outside the update region (Reference 2.5-478) and is not in the updated catalog. However, this event is one of the largest in the Gulf of Mexico and was considered during the investigations for the CPNPP Units 3 and 4 site (see Subsection 2.5.2.4.2.2). The event occurred within the oceanic crust within the eastern Gulf of Mexico. The focal mechanism for the earthquake indicates a reverse sense of motion, and the earthquake depth is reported as 13 to 19 miles (22 to 31 km) (Reference 2.5-478). The Mw 5.8 magnitude for this earthquake is equivalent to Emb 6.1 (see Subsection 2.5.2.1.2 for relationships used in magnitude conversions).

2.5.2.1.3.2 Historical Earthquakes

No additional significant historical earthquakes, defined as earthquakes having an impact on the seismic hazard at CPNPP Units 3 and 4 or seismic source characterization for CPNPP Units 3 and 4, other than those reported in the EPRI-SOG seismicity catalog have been reported since publication of the EPRI-SOG study (References 2.5-369 and 2.5-370). Below is a review of historical earthquakes that are thought to have had significant felt effects within the region

immediately surrounding CPNPP Units 3 and 4 (Figure 2.5.2-201). Magnitudes reported below are Emb magnitudes from the EPRI-SOG catalog (References 2.5-369 and 2.5-370).

1811 to 1812 Emb 7.0 to 7.4 New Madrid, Missouri

Frohlich and Davis (Reference 2.5-378) note that there were no reliable earthquake accounts in Texas prior to 1847, but they mention that the series of New Madrid, Missouri, earthquakes between 1811 and 1812 (December 16, 1811, Emb 7.2; December 16, 1811, Emb 7.0; January 23, 1812, Emb 7.1; February 7, 1812, Emb 7.4) event would have been felt in Texas, assuming isoseismal intensities from the earthquakes are roughly symmetrical about the epicentral area. Frohlich and Davis (Reference 2.5-378) reproduce a figure of Carlson (Reference 2.5-384) that estimates the intensity in the region of CPNPP Units 3 and 4 from the events as MMI IV to V.

October 22, 1882, Emb 5.4 Fort Gibson, Oklahoma

Frohlich and Davis (Reference 2.5-378) present an isoseismal map of the October 22, 1882, Fort Gibson earthquake as having intensities of MMI I to III within the region surrounding CPNPP Units 3 and 4, but they also state that Dallas newspapers at the time reported felt effects at more proximal cities but not in Dallas. Since Dallas is closer to the epicenter than is CPNPP Units 3 and 4, it is reasonable to assume that intensities at CPNPP Units 3 and 4 were very low if at all detectable. This is discussed in the FSAR for CPNPP Units 1 and 2 (Reference 2.5-201).

August 16, 1931, Emb 5.8 Valentine, Texas

Frohlich and Davis (Reference 2.5-378) report that the August 16, 1931, Emb 5.8 earthquake in Valentine, Texas, was felt as far east as Waco, Dallas, San Antonio, and Houston. Felt reports that Frohlich and Davis (Reference 2.5-378) compiled suggest intensities within the region surrounding CPNPP Units 3 and 4 of approximately MMI III to IV. Doser (Reference 2.5-303) determined a normal faulting mechanism with extension oriented northwest-southeast for the event and attribute the event to rupture along the Mayfield fault, a range-bounding fault within the Basin and Range physiographic province (Reference 2.5-385). This event is also discussed in the FSAR for CPNPP Units 1 and 2 (Reference 2.5-201) where it is reported as having an intensity of MMI II to III at the site. The measured intensity range (MMI II to III) is more precise than the felt intensity range (MMI III to IV) from the historical record.

April 9, 1952, Emb 4.9 El Reno, Oklahoma

Frohlich and Davis (Reference 2.5-378) present an isoseismal map for the April 9, 1952, Emb 4.9 El Reno earthquake as having intensities of MMI I to III near CPNPP Units 3 and 4. The closest felt reports to CPNPP Units 3 and 4 summarized by Frohlich and Davis (Reference 2.5-378) include swaying in the

upper floors of buildings in Austin, Abilene, and Wichita Falls. This event is also discussed in the FSAR for CPNPP Units 1 and 2 (Reference 2.5-201) where the event is reported as having intensities of MMI I to III for Dallas and Fort Worth.

2.5.2.2 Geologic and Tectonic Characteristics of the Site and Region

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.2 with the following.

Guidance from the NRC regarding seismic source characterizations is presented in RG 1.208. This guidance states that:

> "…PSHA should be conducted with up-to-date interpretations of earthquake sources, earthquake recurrence, and strong ground motion estimation" (page 3, RG 1.208).

The issued guidance also states that

> "… seismic sources and data accepted by the NRC in past licensing decisions may be used as a starting point (for the PSHA)" (page 14, RG 1.208).

Acceptable starting-point source zone characterizations identified within RG 1.208 include the Lawrence Livermore National Lab study presented in NUREG/CR-5250 and the EPRI-SOG study (References 2.5-369, 2.5-370, and 2.5-335). As part of the acceptance of these studies, RG 1.208 requires that site-specific geological, geophysical, and seismological studies be conducted to determine if these accepted source models adequately describe the seismic hazard for the site of interest given any new data developed since acceptance of the original models. The regulatory guidance explicitly states that:

> "The results of these investigations will also be used to assess whether new data and their interpretation are consistent with the information used in recent probabilistic seismic hazard studies accepted by NRC staff. If new data, such as new seismic sources and new ground motion attenuation relationships, are consistent with the existing earth science database, updating or modification of the information used in the site-specific hazard analysis is not required. It will be necessary to update seismic sources and ground motion attenuation relationships for sites where there is significant new information provided by the site investigation" (page C-1, RG 1.208).

For the case of new information requiring updated source characterizations, RG 1.208 requires that the development of updated source characterizations conform to the guidance presented in NUREG/CR-6372.

NUREG/CR-6372, prepared by a Senior Seismic Hazard Analysis Committee (SSHAC), provides recommendations on the development of PSHA studies for

nuclear facilities. A primary recommendation of the SSHAC is that for a given technical issue (i.e., source zone characterization),

> "The following should be sought … (1) a representation of the legitimate range of technically supportable interpretations among the entire informed technical community…" (page xv, NUREG/CR-6372).

The SSHAC outlines four levels of study for developing the range of interpretations with the choice of level depending on the complexity of the issue to be addressed. The four levels, Level 1 through 4, are distinguished by the increasing levels of sophistication, resources, and participation by technical experts.

For CPNPP Units 3 and 4, the EPRI-SOG source characterizations are used as the base source models for determining the GMRS (Reference 2.5-369). The EPRI-SOG model is chosen based on RG 1.208 that explicitly identifies the source characterizations as an acceptable base model and the availability of detailed documentation describing the EPRI-SOG model (References 2.5-369, 2.5-370, and 2.5-335). However, another supporting reason for using the EPRI-SOG model is that the EPRI-SOG methodology and resultant source characterizations (Reference 2.5-369) are consistent with a high level SSHAC study (Level 3 to 4), and the final aggregate source characterizations were developed to:

> "… reflect the range of current thinking on the causes of earthquakes in the eastern United States" (report summary page 1, Reference 2.5-369).

As required by RG 1.208, site and regional data collected for CPNPP Units 3 and 4 presented in Subsection 2.5.1 and Subsection 2.5.2.1 have been reviewed to:

> "…determine whether there are any new data or interpretations that are not adequately incorporated into the existing PSHA databases" (page 11, RG 1.208).

As required by the regulatory guidance, if significant new data or interpretations are found they require update of the EPRI-SOG source characterizations. Particular attention was paid to this review of new data collected for CPNPP Units 3 and 4 because of the time elapsed since development of the EPRI-SOG source characterizations. The source characterizations of the Dames & Moore (zone 20) and Law Engineering (zone 124) ESTs were subject to additional scrutiny because their respective source models generated the highest and lowest hazard estimates for CPNPP Units 3 and 4, respectively. From this review, it has been determined no new data exist requiring alteration of the EPRI-SOG source characterizations for CPNPP Units 3 and 4 with the exception of those updates presented in Subsection 2.5.2.4.2. The only significant update is that for the Meers fault, and, as described in Subsection 2.5.2.4.2.3.2, this update is developed following SSHAC guidelines.

The following subsections present the seismic source characterizations from the EPRI-SOG model (Reference 2.5-369) that are within the site region. Following those descriptions, a summary of seismic sources used in more recent seismic hazard studies relevant to CPNPP Units 3 and 4 are presented. Source characterizations developed since the EPRI-SOG study commonly use moment magnitude (Mw) to describe earthquake magnitude whereas the EPRI-SOG study used body-wave magnitude (m_b). To allow comparisons between these magnitudes, both m_b and Mw magnitudes are reported below. To convert between the two magnitude scales, the arithmetic mean of the magnitude conversions reported in Atkinson and Boore (Reference 2.5-386), Frankel, et al. (Reference 2.5-339), and EPRI (Reference 2.5-387) are used.

2.5.2.2.1 Summary of EPRI-SOG Source Model

The EPRI-SOG study completed during the 1980s (References 2.5-369, 2.5-370, and 2.5-335) captured uncertainty in seismic source characterizations for the CEUS through the elicitation of six independent ESTs to develop source models of the CEUS. The six teams (Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants) independently evaluated the same database of geologic, geophysical, and seismological observations to develop seismic sources for the CEUS. The teams began by developing criteria for assessing the seismogenic activity of a tectonic feature (e.g., spatial association with large- or small-magnitude earthquakes, evidence of geologically recent slip, orientation relative to the regional stress regime). The ESTs then used the common database to identify potentially seismogenic tectonic features and used their individual criteria to determine the probability of seismogenic activity for these features. Each EST then defined seismic sources from the tectonic features and characterized the sources using the EPRI-SOG PSHA methodology (References 2.5-369 and 2.5-335) within which each source is characterized by the following: probability of activity, maximum earthquake magnitude (Mmax) distribution, alternative source geometries, source interdependencies, and smoothing parameters for use in determining seismicity recurrence parameters.

Each EST team provided detailed documentation of their seismic hazard assessments and source characterizations in separate volumes of the EPRI-SOG study (Reference 2.5-369). However, for implementing the EST source zones into the EPRI-SOG PSHA model, some simplifications were made to the original source characterizations, as documented in the EQHAZARD Primer (Reference 2.5-335). These simplifications primarily reduced unneeded complexity in Mmax distributions. The EQHAZARD Primer (Reference 2.5-335) is the primary source of zone characterizations presented below.

Table 2.5.2-202 through Table 2.5.2-207 summarize the source zone characterizations for sources within 200 mi of CPNPP Units 3 and 4. The contributing sources are shown in Figure 2.5.2-203 through Figure 2.5.2-208 and indicated in Tables 2.5.2-202 through 2.5.2-207. These contributing sources were selected from the larger group by excluding all sources that contribute to less than

1% of the hazard at the site, as determined in a screening evaluation that used the updated source characterizations described in Subsection 2.5.2.4.2 and the updated ground motion equations described in Subsection 2.5.2.4.3. These contributing source zones are the starting point for the PSHA at CPNPP Units 3 and 4. Also shown in Figure 2.5.2-203 through Figure 2.5.2-208 are earthquakes from the combined catalog for CPNPP Units 3 and 4 (see Subsection 2.5.2.1) for earthquakes with Emb > 3.0.

In Subsection 2.5.2.2.1.1 through Subsection 2.5.2.2.1.6, the contributing source zones for each EST are briefly discussed. More detailed information on each source zone is provided in the EST volumes of the EPRI-SOG documentation (Reference 2.5-369).

2.5.2.2.1.1 Sources identified by Bechtel Group

Five source zones from the Bechtel Group EST contribute to hazard at CPNPP Units 3 and 4 (Table 2.5.2-202) (Figure 2.5.2-203) (References 2.5-369, 2.5-370, and 2.5-335): Texas Platform (zone BZ2), Ouachita (zone 38), Oklahoma Aulacogen (zone 39), North Great Plains (zone BZ3), and Combination (zone C04). Bechtel defined four additional zones that extended to within the site region that do not contribute to hazard (Table 2.5.2-202) (References 2.5-369, 2.5-370, and 2.5-335): Meers Fault (zone 40), El Reno (zone 65), Gulf Coast (zone BZ1), and S.E. Oklahoma (zone 55). Following is a brief discussion of the seismic source zones that contribute to hazard:

Texas Platform (zone BZ2)

The Texas Platform source zone is a large background source zone extending from eastern New Mexico into Texas (Figure 2.5.2-203). The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-202). CPNPP Units 3 and 4 are contained within the zone.

Ouachita (zone 38)

The Ouachita source zone extends from Arkansas into east Texas (Figure 2.5.2-203) and was defined to encompass the extent of the Ouachita fold belt within this region. The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-202). The closest approach of the zone to CPNPP Units 3 and 4 is 125 mi.

Oklahoma Aulacogen (zone 39)

The Oklahoma Aulacogen source zone was drawn to encompass the Oklahoma Aulacogen in Texas, Oklahoma, and New Mexico (Figure 2.5.2-203). The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-202). The closest approach of the zone to CPNPP Units 3 and 4 is 89 mi.

North Great Plains (zone BZ3)

The North Great Plains source zone is a large background zone extending over much of the central U.S. and into southern Canada (Figure 2.5.2-203). The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-202). The closest approach of the zone to CPNPP Units 3 and 4 is 89 mi.

Combination (zone C04)

Combination (zone C04) is comprised of the Oklahoma Aulacogen (zone 39) and Ouachita (zone 38) source zones. The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-202). The closest approach of the zone to CPNPP Units 3 and 4 is 89 mi.

2.5.2.2.1.2 Sources identified by Dames & Moore

Seven source zones from the Dames & Moore Group EST contribute to hazard at CPNPP Units 3 and 4 (Table 2.5.2-203) (Figure 2.5.2-204) (References 2.5-369, 2.5-370, and 2.5-335): Southern Coastal Margin (zone 20), Ouachitas Fold Belt (zone 25), Kink in Ouachita Fold Belt (zone 25a), Southern Oklahoma Aulacogen (zone 28), Default for Southern Oklahoma (zone 28b), New Mexico (zone 67) and Combinzation (zone C08). Dames & Moore defined four additional zones that extend to within the site region that do not contribute to hazard (Table 2.5.2-203) (References 2.5-369, 2.5-370, and 2.5-335): B-W-M Fault (zone 29), A/W Uplift (zone 30), Ardmore Basin (zone 32) and Anadarko Basin (zone 33). Following is a brief discussion of the seismic source zones that contributed to hazard at CPNPP Units 1 and 2 and are used in the PSHA for CPNPP Units 3 and 4:

Southern Coastal Margin (zone 20)

The South Coastal Margin source zone is a large regional zone that extends from the continental shelf off eastern Florida, along the Texas coastal plain, and into Mexico (Figure 2.5.2-204). Dames & Moore designed the zone to largely parallel the southern-rifted margin of North America, and they state that they have no tectonic basis with which to define the seismic potential of the zone. The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 83 mi.

Ouachitas Fold Belt (zone 25)

The Ouachitas Fold Belt source zone encompasses the Ouachita orogenic front extending from Arkansas through Oklahoma, Texas, and into eastern Mexico (Figure 2.5.2-204). The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 26 mi.

Kink in Ouachita Fold Belt (zone 25a)

The Kink in Ouachita Fold Belt source zone is an alternative interpretation of the Ouachitas Fold Belt (zone) representing the opinion of the Dames & Moore EST that seismicity within the fold belt may be preferentially associated with a kink in the fold belt located at the Texas-Oklahoma border (Figure 2.5.2-204). The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 75 mi.

Southern Oklahoma Aulacogen (zone 28)

The Southern Oklahoma Aulacogen source zone extends along the Texas-Oklahoma border into the Texas panhandle (Figure 2.5.2-204). The source was defined to encompass the Southern Oklahoma Aulacogen. The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 91 mi.

Default for Southern Oklahoma (zone 28b)

The Default for Southern Oklahoma Aulacogen source zone extends along the Texas-Oklahoma border into the Texas panhandle (Figure 2.5.2-204). The source is a default source zone used to represent the seismic activity of the Southern Oklahoma Aulacogen in conjunction with the Southern Oklahoma Aulacogen (zone 28) source zone. The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 70 mi.

New Mexico (zone 67)

The New Mexico source zone extends from Texas into New Mexico and part of northern Mexico (Figure 2.5.2-204). Dames & Moore describe the boundaries of the zone as being defined largely on the basis of the extent of arches and basins formed during the Paleozoic (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). CPNPP Units 3 and 4 are located within this source zone.

Combination (zone C08)

The Combination source zone (zone C08) is comprised of the Ouachitas Fold Belt (zone 25) and the Kink in Ouachitas Fold Belt (zone 25A) source zones (Figure 2.5.2-204). The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-203). The closest approach of the zone to CPNPP Units 3 and 4 is 26 miles.

2.5.2.2.1.3 Sources identified by Law Engineering

Two source zones from the Law Engineering EST contribute to hazard at CPNPP Units 3 and 4 (Table 2.5.2-204) (Figure 2.5.2-205) (References 2.5-369, 2.5-370, and 2.5-335): New Mexico-Texas Block (zone 124) and Oklahoma Aulacogen-

Arbuckle Wichita Rift (zone 26). Law Engineering defined three additional zones that extend to within the site region that do not contribute to hazard (Table 2.5.2-204) (References 2.5-369, 2.5-370, and 2.5-335): Eastern Mid-Continent (zone 119), Western Mid-Continent (zone 120) and South Coastal Block (zone 126). Following is a brief discussion of the seismic source zones that contribute to hazard:

<u>New Mexico-Texas Block (zone 124)</u>

The New Mexico-Texas Block source zone is a large areal source defined by the boundaries of the Southern Oklahoma Aulacogen, the Ouachita gravity high, and the magnetic trend of the Rio Grande Rift-Colorado Front Ranges (Reference 2.5-369). This zone encompasses the majority of Texas, excluding the Gulf Costal Plain, and extends into eastern New Mexico (Figure 2.5.2-205). The zone is characterized by an upper-bound Mmax of m_b 5.8 (Table 2.5.2-204). CPNPP Units 3 and 4 are located within this source zone.

<u>Oklahoma Aulacogen-Arbuckle Wichita Rift (zone 26)</u>

The Oklahoma Aulacogen-Arbuckle Wichita Rift source zone overlaps the Texas-Oklahoma border and extends into the Texas panhandle and New Mexico (Figure 2.5.2-205). The source zone geometry was defined to encompass the extent of the Southern Oklahoma Aulacogen. The zone is characterized by an upper-bound Mmax of m_b 6.8 (Table 2.5.2-204). The closest approach of the zone to CPNPP Units 3 and 4 is 93 mi.

2.5.2.2.1.4 Sources identified by Rondout Associates

Four source zones from the Rondout Associates EST that contribute to hazard at CPNPP Units 3 and 4 (Table 2.5.2-205) (Figure 2.5.2-206) (References 2.5-369, 2.5-370, and 2.5-335): Southern Oklahoma Aulacogen-Ouachita Mountains (zone 16), Nemaha-Anadark (zone 23), Gulf Coast to Bahamas Fracture Zone (zone 51) and Grenville Crust (zone C02). Rondout Associates defined one additional zone that extends to within the site region that does not contribute to hazard (Table 2.5.2-205) (References 2.5-369, 2.5-370, and 2.5-335): Pre-Grenville Precambrian Craton (zone 52). Following is a brief discussion of the seismic source zones that contributed to hazard:

<u>Southern Oklahoma Aulacogen-Ouachita Mountains (zone 16)</u>

The Southern Oklahoma Aulacogen-Ouachita Mountains source zone extends from Arkansas into Texas and Oklahoma along the Texas-Oklahoma border (Figure 2.5.2-206). The zone geometry was defined to encompass the Oklahoma Aulacogen (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 6.8 (Table 2.5.2-205). The closest approach of the zone to CPNPP Units 3 and 4 is 80 mi.

<u>Grenville Crust (zone C02)</u>

The Grenville Crust source zone is a set of discrete source zones that extend across the eastern and southern margin of the U.S. (Figure 2.5.2-206). The closest portion of the source zone to CPNPP Units 3 and 4 encompasses central and eastern Texas. The source zone is a background source representing all of the Grenville age crust that is not contained within a source zone based on the presence of tectonic features (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 5.8 (Table 2.5.2-205). CPNPP Units 3 and 4 are located within this source zone.

Nemaha-Anadark (zone 23)

The Nemaha-Anadark source zone is an elongated zone extending from southern to northern Oklahoma (Figure 2.5.2-206). The zone geometry was defined to encompass the intersection of possible extensions of the Humboldt fault zone and the Nemaha anticline (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 7.0 (Table 2.5.2-205). The closest approach of the zone to CPNPP Units 3 and 4 is 140 miles.

Gulf Coast to Bahamas Fracture (zone 51)

The Gulf Coast to Bahamas Fracture source zone is a large background source zone extending from the coastal plains of the Gulf of Mexico into the central Gulf of Mexico (Figure 2.5.2-206). The zone geometry was defined to represent the Paleozoic crust of the Gulf of Mexico region as distinct from that of the Appalachians (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 5.8 (Table 2.5.2-205). The closest approach of the zone to CPNPP Units 3 and 4 is 57 miles.

2.5.2.2.1.5 Sources identified by Weston Geophysical Corporation

Four source zones from the Weston Geophysical Corporation EST contribute to hazard at CPNPP Units 3 and 4 (Table 2.5.2-207) (Figure 2.5.2-208) (References 2.5-369, 2.5-370, and 2.5-335): Southwest (zone 109), Combination (zone C31), Ancestral Rockies (zone 36) and Gulf Coast (zone 107). Weston Geophysical Corporation defined one additional zone that extends to within the site region that does not contribute to hazard (References 2.5-369, 2.5-370, and 2.5-335): Delaware Basin (zone 37). Following is a brief discussion of the seismic source zones that contributed to hazard:

Southwest (zone 109)

The Southwest source zone is a large background source that extends over much of Texas, New Mexico, Colorado, and Wyoming (Figure 2.5.2-207). The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-206). CPNPP Units 3 and 4 are located within this zone.

Combination (zone C31)

The Combination (zone C31) source zone is an alternative geometry for the Southwest (zone 109) background zone that excludes the Delaware Basin in west Texas (Figure 2.5.2-207). The zone is characterized by an upper-bound Mmax of m_b 6.6 (Table 2.5.2-206). CPNPP Units 3 and 4 are located within this zone.

Ancestral Rockies (zone 36)

The Ancestral Rockies source zone extends from Arkansas, through the majority of Oklahoma, and into the Texas panhandle (Figure 2.5.2-207). The geometry of this zone was defined to encompass the extent of the Southern Oklahoma Aulacogen and associated tectonic features. The zone is characterized by an upper-bound Mmax of m_b 6.0 (Table 2.5.2-206). The closest extent of this zone to CPNPP Units 3 and 4 is 79 mi.

Gulf Coast (zone 107)

The Gulf Coast source zone is a large background source zone extending from the coastal plains of the Gulf of Mexico into the central Gulf of Mexico (Figure 2.5.2-207). The zone geometry encompasses regions for which no other source zones were defined (Reference 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 6.0 (Table 2.5.2-206). The closest approach of the zone to CPNPP Units 3 and 4 is 79 miles.

2.5.2.2.1.6 Sources identified by Woodward-Clyde Consultants

Four source zones from the Woodward-Clyde Consultants EST contributed to hazard at CPNPP Units 3 and 4 (Table 2.5.2-207) (Figure 2.5.2-208) (References 2.5-369, 2.5-370, and 2.5-335): Central U.S. Background (zone BG44), Southern Oklahoma Aulacogen (zone 46), Alternate Configuration of Southern Oklahoma Aulacogen (46a) and Southern Oklahoma Gravity Anomaly (zone 48). Woodward-Clyde Consultants defined two additional zones that extend to within the site region that do not contribute to hazard at CPNPP Units 1 and 2 (Table 2.5.2-207) (References 2.5-369, 2.5-370, and 2.5-335): Meers Fault (zone 49) and Eastern Oklahoma Seismic Zone (zone 52). Following is a brief discussion of the seismic source zones that contribute to hazard:

Central US Background (zone BG44)

The Central US Background (zone BG44) is a large areal background source centered on CPNPP Units 1 and 2. The zone is a quadrilateral shape with sides approximately 6° long, in both longitude and latitude (Figure 2.5.2-208). The zone is characterized by an upper-bound Mmax of m_b 6.5 (Table 2.5.2-207). CPNPP Units 3 and 4 are in this zone.

Southern Oklahoma Aulacogen (zone 46)

The Southern Oklahoma Aulacogen source zone extends from south-central Oklahoma along the Oklahoma-Texas border into the Texas panhandle (Figure 2.5.2-208). The zone geometry is defined to encompass the extent of the Southern Oklahoma Aulacogen. The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-207). The closest approach of the zone to CPNPP Units 3 and 4 is 100 mi.

Alternate Configuration for Southern Oklahoma Aulacogen (zone 46A)

The Alternate Configuration for Southern Oklahoma Aulacogen source zone is an alternative geometry for the Southern Oklahoma Aulacogen (zone 46) source zone that extends further to the northeast into New Mexico. The zone is characterized by an upper-bound Mmax of m_b 7.2 (Table 2.5.2-207). The closest approach of the zone to CPNPP Units 3 and 4 is 100 mi.

Southern Oklahoma Gravity Anomaly (zone 48)

The Southern Oklahoma Gravity Anomaly source zone is a northwest trending, elongated zone that extends from northern Texas into southern Oklahoma. (Figure 2.5.2-208). The zone geometry was defined to encompass the Bouguer gravity low north of the Oklahoma aulacogen (References 2.5-369). The zone is characterized by an upper-bound Mmax of m_b 7.1 (Table 2.5.2-207). The closest approach of the zone to CPNPP Units 3 and 4 is 131 miles.

2.5.2.2.2 Post-EPRI-SOG Source Characterization Studies

Since publication of the EPRI-SOG seismic source characterizations for the CEUS in 1986 (Reference 2.5-369), there have been several regional-scale source characterization studies within the CPNPP Units 3 and 4 site region. These studies include:

- A Lawrence Livermore National Laboratory (LLNL) report on the seismic hazard characterization of nuclear power plants in CEUS (NUREG/CR-5250, Vol. 1 and Vol. 5);
- A draft report prepared by Geomatrix Consultants for the U.S. Bureau of Reclamation detailing the seismotectonics of the Wichita Uplift and Oklahoma Aulacogen region (Reference 2.5-388):
- A draft report prepared by Geomatrix Consultants for the NRC on the Quaternary activity of the Meers fault (Reference 2.5-389);
- A U.S. Bureau of Reclamation PSHA study for dams in Oklahoma (Reference 2.5-390);
- A LLNL PSHA study for the Pantex nuclear weapon support facility outside Amarillo, TX (Reference 2.5-391); and

- The United States Geological Survey (USGS) National Seismic Hazard Map program source characterizations used in developing the USGS National Seismic Hazard Maps (References 2.5-339, 2.5-392, and 2.5-321).

The source characterizations used within these studies relevant to the characterization of seismic sources for CPNPP Units 3 and 4 are briefly summarized below. Source characterizations from these studies that were developed using post-EPRI-SOG research will be considered as possible revisions or additions to the EPRI-SOG model that must be considered to meet the guidelines of RG 1.208.

2.5.2.2.2.1 Lawrence Livermore National Laboratory 1989 Study

In 1988 LLNL completed a PSHA study, for the NRC, for nuclear power plants within the CEUS that was similar to the EPRI-SOG study (NUREG/CR-5250, Vol. 1). The LLNL study developed a PSHA methodology that included source characterizations and ground motion equations for the CEUS provided by independent experts. Hazard at a particular site was calculated for the source model defined by each expert using each of the ground motion relationships, and the final hazard at the site was the aggregate of all source models and ground motion relationships. As stated in RG 1.208, the resultant PSHA model is accepted by the NRC for use in determining the GMRS for modern COL applications if modifications are made to account for advances in ground motion equations and source characterizations.

The source characterizations of the 1989 LLNL study were developed by eleven independent experts resulting in eleven different source models (NUREG/CR-5250, Vol. 1). The source models were developed by the experts using geologic and geophysical data the experts compiled themselves, though at later stages of the study seismicity catalogs were provided to the experts for use with their discretion. The source models were revised through a series of feedback loops with the project organizers at LNLL that provided clarification of the project methodology and preliminary results for the source models. The final source models presented in the 1989 report volume (NUREG/CR-5250, Vol. 1) are defined by their source zone geometry, type of recurrence relationship, Mmax, and seismicity recurrence parameters, all provided by the individual experts.

The results of the 1989 LLNL study identified which source zones each expert considers the most significant contributors to hazard at CPNPP Units 1 and 2 (NUREG/CR-5250, Vol. 5). In general, these significant contributors include source zones characterizing the New Madrid region, the Oklahoma Aulacogen and Wichita Uplift, the Ouachita fold belt, and large background zones. The parameterizations of these source zones for each expert is described in Bernreuter, et al. (NUREG/CR-5250, Vol. 1) and is briefly summarized below:

- New Madrid: Probability of activity (Pa) is 1.0 for all experts' characterizations, and Mmax varies between m_b 7.0 and 8.0, depending on expert;
- Oklahoma Aulacogen and Wichita Uplift: Pa varies between 0.7 and 1.0, and Mmax varies between m_b 5.8 and 7.5, depending on expert;
- Ouachita fold belt: Pa varies between 0.6 and 0.7, and Mmax varies between m_b 5.4 and 6.3, depending on expert; and
- Background zones: Pa varies between 0.7 and 1.0, and Mmax varies between m_b 4.8 and 7.5, depending on expert.

An update to the 1989 LLNL study was completed in 1994 with the publication of NUREG-1488. The focus of this study was to reduce the uncertainty in ground motion estimates, and this was accomplished in part by having the experts reevaluate the uncertainty they reported in seismicity parameters. There were no significant changes to the above characterizations.

The geometry and seismicity parameters of these source zones identified as being significant to the hazard are broadly consistent with the EPRI-SOG source zones used as the basis for the PSHA at CPNPP Units 3 and 4 (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208). The source zones of the LLNL study do not present any new information that requires consideration for CPNPP Units 3 and 4.

2.5.2.2.2.2 Draft Report to the Bureau of Reclamation on the Wichita Uplift Region

In 1990 Geomatrix Consultants provided the U.S. Bureau of Reclamation with a draft report describing the results of a study on the seismotectonics of the Oklahoma-Wichita Uplift region in southern Oklahoma and northern Texas (Reference 2.5-388). The southernmost extent of the study region is approximately 50 mi north of CPNPP Units 3 and 4. The study evaluated faults, tectonic structures, and historical seismicity within the region to estimate the potential seismic hazard at seven dams in Oklahoma and northern Texas. Of the faults and tectonic structures investigated, only two features were determined to have potential Quaternary activity: the Meers and Criner faults. Seismic source characterizations for the Meers and Criner faults, as well as two areal source zonations, were developed as a final product of the study with the general characteristics as follows:

- Meers fault: Mmax of Ms 6.75 to 7.25 (m_b 6.5 to 6.8) with a return period on the order of 2000 to 3000 years;
- Criner fault: Mmax of Ms 6.5 to 7.0 (m_b 6.3 to 6.6) with a return period on the order of 2000 to 3000 years; and

- Source zones: two separate source zonations with three and four areal source zones, respectively. The source zone geometries, based on seismicity and tectonic structure, encompassed the region of the Southern Oklahoma Aulacogen in Oklahoma and Texas. Seismicity parameters were determined individually for each zone, and the Mmax for all zones was Ms 6.5 (m_b 6.3).

The areal sources defined by Geomatrix Consultants (Reference 2.5-388) are broadly consistent with the EPRI-SOG source zones used as the basis for the PSHA at CPNPP Units 3 and 4 (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208), and do not present any new information that requires consideration for CPNPP Units 3 and 4. However, the characterization of the Meers and Criner faults by Geomatrix Consultants (Reference 2.5-388) is based on information published after the EPRI-SOG study (Reference 2.5-369), and these fault source characterizations do require consideration for CPNPP Units 3 and 4. A summary of the information published since the EPRI-SOG study (Reference 2.5-369) on the Meers and Criner faults is presented in Subsections 2.5.1.1.4.3.6.1.2 and 2.5.1.1.4.3.6.2. Information presented in those subsections identifies the Criner fault as not capable and the Meers fault as capable. The updated source characterization of the Meers fault used in this study is presented in Subsection 2.5.2.4.2.3.2.

2.5.2.2.2.3 Draft Report (Quaternary faulting) to the NRC on the Wichita Uplift Region

In 1993 Swan, et al. (Reference 2.5-389) provided the NRC with a draft report describing the results of a study investigating Quaternary faulting along the Wichita fault system. Much of the work for the study was done in conjunction with the work described in the Geomatrix Consultants draft report to the U.S. Bureau of Reclamation (Reference 2.5-388) described in Subsection 2.5.2.2.2.2. Therefore, the conclusions of the reports are largely the same, and the Meers and Criner faults are identified as the only potentially capable faults along the Wichita Uplift. As discussed in Subsection 2.5.2.2.2.2, the conclusions of the Swan, et al. (Reference 2.5-389) study require that the Meers and Criner faults be evaluated as potential seismic sources for CPNPP Units 3 and 4. A summary of the evaluation of the capability of the faults based on information published since the EPRI-SOG study (Reference 2.5-369) is presented in Subsections 2.5.1.1.4.3.6.1.1 and 2.5.1.1.4.3.6.2. In Subsection 2.5.1.1.4.3.6.2 it is determined that the Criner fault is not a capable fault and that the Meers fault is a capable fault. The updated source characterization of the Meers fault used in this study is presented in Subsection 2.5.2.4.2.3.2.

2.5.2.2.2.4 Bureau of Reclamation PSHA Study of Dams in Oklahoma and Texas

In 1997 LaForge (Reference 2.5-390) conducted a PSHA study for seven dams in Oklahoma. The closest extent of the study area to CPNPP Units 3 and 4 is approximately 110 mi. For the study he defined three areal source zones and one

fault source, the Meers fault. The areal source zones were limited in extent to Oklahoma and were defined based on spatial seismicity patterns. LaForge (Reference 2.5-390) estimated Mmax values between Mw 6.0 and 6.5 (m_b 6.3 to 6.6) for the zones from Mmax estimates from geographically similar source zones in the EPRI-SOG (Reference 2.5-369) and 1989 LLNL (NUREG/CR-5250, Vol. 1) studies for the NRC. LaForge (Reference 2.5-390) characterized the Meers fault as capable of Mw 7.0 (m_b 6.9) earthquakes with a return period of 5000 years based on the results of Swan, et al. (Reference 2.5-389). Despite the proximity of the Criner fault to the dams analyzed in this study and the identification of the Criner fault by Swan, et al. (Reference 2.5-389) as a potentially active feature, LaForge (Reference 2.5-390) explicitly excludes the fault as a source based on work post-dating the Swan, et al. (Reference 2.5-389) study that characterizes the fault as inactive. A summary of this work is presented in Subsection 2.5.1.1.4.3.6.1.2.

The areal sources defined by LaForge (Reference 2.5-390) are broadly consistent with the EPRI-SOG source zones used as the basis for the PSHA at CPNPP Units 3 and 4 (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208). These source zones do not present any new information that requires consideration for CPNPP Units 3 and 4. As discussed in Subsection 2.5.2.2.2.2, the Meers fault characterization used in this report requires that it be evaluated as a potential seismic source for CPNPP Units 3 and 4. A summary of this evaluation based on information published since the EPRI-SOG study (Reference 2.5-369) is presented in Subsection 2.5.1.1.4.3.6.1.1. The updated source characterization of the Meers fault used in this study is presented in Subsection 2.5.2.4.2.3.2.

2.5.2.2.2.5 LLNL PSHA for Pantex Nuclear Weapons Support Facility

In 1998 Savy, et al. (Reference 2.5-391) with LLNL conducted a PSHA of the Pantex nuclear weapons support facility in Amarillo, Texas, approximately 300 mi to the northeast of CPNPP Units 3 and 4. The study region was a 10° x 10° quadrilateral centered on the Pantex site that includes eastern Colorado and New Mexico, Oklahoma, Kansas, and the majority of Texas. Within this region Savy, et al. (Reference 2.5-391) defined five areal source zones and fourteen faults largely based on the results of previous seismic source characterization studies. These sources are summarized as follows:

<u>Background Zones</u>

Savy, et al. (Reference 2.5-391) defined three background source zones (the Rocky Mountain, craton, and extended margin zones) based on zones of the same name used in the 1996 USGS National Seismic Hazard Maps (Reference 2.5-339). Savy, et al. (Reference 2.5-391) defined an additional zone to represent the Rio Grande Rift (RGR) in Texas, New Mexico, and southern Colorado. Seismicity rates for these zones are spatially uniform and were developed primarily using rates from the 1996 USGS model (Reference 2.5-339) and regional seismicity catalogs described within Savy, et al. (Reference 2.5-391).

Upper- and lower-bound Mmax values for the zones were defined as: craton Mw 6.0 and 6.75 (m_b 6.3 and 6.76); extended margin Mw 6.75 and 7.8 (m_b 6.76 and 7.4); RGR Mw 6.3 and 7.0 (m_b 6.5 and 6.9); Rocky Mountain Mw 6.0 and 6.75 (m_b 6.3 and 6.76).

Amarillo-Wichita Uplift Zone

Savy, et al. (Reference 2.5-391) defined an areal source zone representing the Amarillo-Wichita Uplift and Southern Oklahoma Aulacogen based on their opinion that the zone is capable of elevated rates of seismicity relative to surrounding areas. Savy, et al. (Reference 2.5-391) report that the geometry of the zone is defined by the bounding faults of the Uplift. Seismicity parameters are uniform within the zone and determined from observed seismicity. Savy, et al. (Reference 2.5-391) state that the zone is characterized to represent seismicity of magnitudes less than the characteristic magnitude of the Meers fault, and as such the zone has lower- and upper-bound Mmax of Mw 6.0 and 7.0 (m_b 6.3 and 6.9).

Spatially Variable Seismicity Parameter Zones

Savy, et al. (Reference 2.5-391) also defined an approximately 50 mi x 50 mi (80 km x 80 km) region around the site where seismicity parameters varied over 6.2 mi x 6.2 mi (10 km x 10 km) cells based on seismicity parameters from the 1996 USGS National Seismic Hazard Maps (Reference 2.5-339). Upper- and-lower bound Mmax values within the zones are Mw 6.0 and 6.75 (m_b 6.3 and 6.76).

Meers Fault

Savy, et al. (Reference 2.5-391) defined a discrete fault source for the Meers fault based on the work of Swan, et al. (Reference 2.5-389) and Crone and Luza (Reference 2.5-284) (see Subsection 2.5.1.1.4.3.6.1.1). Based on this work, Savy, et al. (Reference 2.5-391) use a characteristic earthquake model to represent the Meers fault with lower- and upper-bound magnitudes of Mw 6.75 and 7.25 and best estimate, upper-bound, and lower-bound return periods of 1150 years, 500,000 years, and 700 years, respectively. Savy, et al. (Reference 2.5-391) also include an alternative source model of the Meers fault that allows for the rupture to occur along an extension of the Meers fault extending the entire length of their Amarillo-Wichita Uplift zone. This alternative Meers fault source is meant to represent the possibility of Meers-like ruptures within the Southern Oklahoma Aulacogen on faults that do not yet have recognized Quaternary events. Savy, et al. (Reference 2.5-391) use a characteristic earthquake model to represent the Meers extension with lower- and upper-bound magnitudes of Mw 7.25 and 7.75 (m_b 7.1 and 7.4) and best-estimate, upper-bound, and lower-bound return periods of 500,000 years, 1,000,000 years, and 200,000 years, respectively.

Cheraw Fault

Savy, et al. (Reference 2.5-391) defined a discrete fault source for the Cheraw fault in southeast Colorado based on the work of Crone, et al. (Reference 2.5-323) (see discussion in Subsection 2.5.1.1.4.3.7.2). Savy, et al. (Reference 2.5-391) use a characteristic earthquake model to represent the Meers extension with lower- and upper-bound magnitudes of Mw 6.75 and 7.25 (m_b 6.76 and 7.1) and best-estimate, upper-bound, and lower-bound return periods of 6500 years, 500,000 years, and 3600 years, respectively.

Rio Grande Rift Faults

Savy, et al. (Reference 2.5-391) defined 14 discrete fault sources for faults within the central and eastern RGR based on a study evaluating seismic hazard at Los Alamos National Laboratory (Reference 2.5-309). Mmax and recurrence rates for the faults are based on the results of the Los Alamos report (Reference 2.5-309) and the 1996 USGS National Seismic Hazard Maps (Reference 2.5-339). The best-estimate return periods for Mw 6 (m_b 6.3) events vary between 1200 years and 10,000 years, depending on the fault, and the best-estimate characteristic magnitude varies between Mw 6.7 and 7.5 (m_b 6.7 and 7.2), depending on the fault.

The areal source zones defined within the LLNL Pantex report (Reference 2.5-391) are broadly consistent with the EPRI-SOG source zones used as the basis for the PSHA at CPNPP Units 3 and 4 (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208). For example, the LLNL zones were defined to represent tectonic features similar to those identified by many of the ESTs, and the seismicity parameters for those zones were determined from regional catalogs of seismicity. Two aspects of the areal zones that are noticeably different are:

- Mmax values for the LLNL extended margin source zone are generally higher than Mmax values for corresponding zones from the EPRI-SOG model (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208); and

- The RGR is characterized as a seismic source in the LLNL study and is not included by most ESTs in the EPRI-SOG study (Reference 2.5-369).

The Mmax value for the extended margin used is based on the USGS National Seismic Hazard Maps evaluation of Mmax values. As discussed in Subsection 2.5.2.2.2.6, the USGS characterization of CEUS seismic sources allows for large earthquakes (Mw > 7.5) within the entire region of extended crust with the goal of developing a source model capable of explaining the 1886 Mw 7.3 (m_b 7.1) Charleston earthquake (Reference 2.5-321).

During development of the EPRI-SOG model, the ESTs were aware of the 1886 Mw 7.3 (m_b 7.1) Charleston earthquake and chose to limit the possible region where earthquakes this large could occur to the Charleston area and not allow the

Mmax associated with this event to extend out to the extended margin of the Texas coastal plain. The use of a larger Mmax within the USGS model, and thus the LLNL Pantex model, primarily reflects different interpretations and opinions of the seismogenic potential of the Texas coastal plain and not new post-EPRI-SOG information on the seismogenic potential of the coastal plain. As such, the high Mmax value for the USGS extended margin source zones do not necessitate the revision of Mmax values for correlative source zones in the EPRI-SOG model.

The inclusion of the RGR in the LLNL source zones and not in the EPRI-SOG source descriptions reflects the CEUS focus of the EPRI-SOG study and the lack of information regarding the seismic potential of RGR-related faults at the time of the EPRI-SOG study. The post-EPRI-SOG information on the seismic potential for the RGR on which these source zones are based requires an evaluation for CPNPP Units 3 and 4. New information on the activity of RGR faults, presented in Subsection 2.5.1.1.4.3.7.1, is used to develop preliminary source characterizations of RGR faults (see Subsection 2.5.2.4.2.3.3) used in a screening study for seismic sources at CPNPP Units 3 and 4 (see Subsection 2.5.2.4.2.3.3.1).

The fault sources described in the Pantex report also require consideration for CPNPP Units 3 and 4. The need to consider an update to the Meers fault and RGR faults was previously mentioned. In addition to these faults, post-EPRI-SOG studies of the Cheraw fault have noted three surface rupturing events in the past 25,000 years (References 2.5-323 and 2.5-326). The results of these studies are presented in Subsection 2.5.1.1.4.3.7.2 and are used to develop a preliminary source characterization of the Cheraw faults used in a screening study for seismic sources at CPNPP Units 3 and 4 (see Subsection 2.5.2.4.2.3.4).

2.5.2.2.2.6 USGS National Seismic Hazard Maps

As part of the USGS National Seismic Hazard Mapping program, seismic hazard maps for the conterminous U.S. were produced in 1996 (Reference 2.5-339) and updated in 2002 (Reference 2.5-321) using source characterizations developed by the USGS. The USGS does not use a formal expert elicitation process and does not explicitly attempt to represent the full range of uncertainty in source characterizations. However, the source models are developed from published literature, and working groups are held to discuss source characterizations. Therefore, the USGS source characterizations can be viewed as good representations of the modern interpretation of seismic hazard posed by a given source. Aspects of the USGS source characterizations based on the 2002 model (Reference 2.5-321) relevant to CPNPP Units 3 and 4 are discussed below. It should be noted that preliminary updated source characterizations for a 2007 version of the hazard maps were released for public comment during the CPNPP Units 3 and 4 project (Reference 2.5-392). The updated characterizations provide minor changes to some of the source characterizations relevant to CPNPP Units 3 and 4, but these changes do not impact any conclusions reached regarding source models for CPNPP Units 3 and 4.

In contrast to the EPRI-SOG model (Reference 2.5-369) that incorporates many background zones and local sources, the USGS source model for the CEUS includes a small number of large areal source zones and discrete sources. Within the CPNPP Units 3 and 4 site region there are two areal source zones (the extended crust and craton) and one fault source (the Meers fault). In addition to these sources, the Cheraw fault, RGR faults, and New Madrid seismic zone are additional sources within the 2002 USGS model that are potentially pertinent to hazard calculations for CPNPP Units 3 and 4.

In the 2002 USGS model, the craton zone is characterized by an Mmax of Mw 7.0 (m_b 6.9) and the extended crust is characterized by an Mmax of Mw 7.5 (m_b 7.2) (Reference 2.5-321). In both zones seismicity recurrence parameters are determined from observed seismicity. As discussed in Subsection 2.5.2.2.2.5 with respect to the LLNL Pantex plant PSHA, Mmax values for the extended margin source zone are generally higher than Mmax values for corresponding zones in the EPRI-SOG model (Table 2.5.2-202 through Table 2.5.2-207; Figure 2.5.2-203 through Figure 2.5.2-208). However, this contrast in Mmax does not necessitate updating the EPRI-SOG Mmax values because it is not based on post-EPRI-SOG information.

The Meers and Cheraw fault source characterizations in the USGS 2002 model (Reference 2.5-339) are based on information that post-dates the EPRI-SOG study (References 2.5-389, 2.5-284, and 2.5-323, for example). As discussed above with respect to other post-EPRI-SOG source characterizations, the Meers and Cheraw faults need to be reevaluated for CPNPP Units 3 and 4 based on the post-EPRI-SOG source characterizations. The post-EPRI-SOG information from which these characterizations are derived is reviewed in Subsection 2.5.1.1.4.3.6, and the source characterizations that are developed using this information are presented in Subsection 2.5.2.4.2.3.

The 2002 USGS models (Reference 2.5-321) characterize the NMSZ using a characteristic earthquake model with Mmax values and weights of Mw 7.3 (0.15), 7.5 (0.2), 7.7 (0.5), and 8.0 (0.15) (m_b 7.1, 7.2, 7.3, and 7.5, respectively). The mean recurrence interval for characteristic earthquakes is defined as 500 years. As with the Meers and Cheraw faults, the USGS characterization of the NMSZ is based on post-EPRI-SOG research (References 2.5-393, 2.5-330, 2.5-336, and 2.5-329, for example), so the source characterization of the NMSZ needs to be reevaluated for CPNPP Units 3 and 4. The post-EPRI-SOG information on which these characterizations are based is summarized in Subsection 2.5.1.1.4.3.7.3, and the source characterizations that are developed using this information are presented in Subsection 2.5.2.4.2.3.1.

The 2002 USGS model (Reference 2.5-321) also includes 41 RGR faults. These faults are characterized using both characteristic earthquake and exponential recurrence models. Characteristic earthquake magnitudes for the faults vary between Mw 6.2 and 7.5 (m_b 6.4 and 7.2), and return periods vary between 4000 and 190,000 years. The source characterizations of the RGR faults used by the USGS are largely based on post-EPRI-SOG research (References 2.5-339 and

2.5-321) and, as discussed in Subsection 2.5.2.2.2.5, the EPRI-SOG ESTs generally did not include the RGR in their source characterizations. Therefore, RGR sources are evaluated as potential seismogenic sources for CPNPP Units 3 and 4. Background information on RGR faults sources is presented in Subsection 2.5.1.1.4.3.7.1, and the source characterizations that are developed using this information are presented in Subsection 2.5.2.4.2.3.

2.5.2.3 Correlation of Earthquake Activity with Seismic Sources

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.3 with the following.

As discussed in Subsection 2.5.2.2.1, ESTs within the EPRI-SOG project used the spatial distribution of seismicity to subdivide the CEUS into seismic source zones (Reference 2.5-369). The seismicity catalog used by the ESTs was the EPRI-SOG catalog described in Subsection 2.5.2.1.1. An updated catalog was developed for use in the CPNPP Units 3 and 4 study (see discussion in Subsection 2.5.2.1.2), and the two catalogs can be compared to assess any changes in the patterns of seismicity or if there exists any correlation between geologic structures and seismicity not identified within the EPRI-SOG study (Reference 2.5-369). Comparison of the catalogs yields the following conclusions:

- The updated seismicity catalog does not show any earthquakes of Emb ³ 3.0 within approximately 90 mi of the site. Accordingly, there are no earthquakes of Emb ³ 3.0 within 90 mi of the site that can be associated with a known geologic structure (Figure 2.5.2-201 and Figure 2.5.2-202);
- The updated seismicity catalog does show a concentration of seismicity in the Southern Oklahoma Aulacogen that has a spatial pattern consistent with the pattern observed in the EPRI-SOG catalog (Figure 2.5.2-201 and Figure 2.5.1-208). In particular, there is a west-northwest band of seismicity extending from Arkansas, through southern Oklahoma, and into the Texas panhandle. This correlation and pattern was noted by the ESTs during the EPRI-SOG study (Reference 2.5-369);
- The updated seismicity catalog does not show a pattern of seismicity different from that of the EPRI-SOG catalog that would suggest a new seismic source in addition to those included in the EPRI-SOG characterizations (Figure 2.5.2-201);
- The updated seismicity catalog does show a similar spatial distribution of earthquakes to that of the EPRI-SOG catalog, suggesting that no significant revisions to the geometry of seismic sources defined in the EPRI-SOG characterization is required (Figure 2.5.2-201); and
- The updated catalog contains two earthquakes that are larger in magnitude than some of the lower-bound Mmax values used by ESTs to characterize source zones within which these earthquakes occurred. These earthquakes are the April 14, 1995, earthquake and the January 2,

1992, earthquake (Figure 2.5.2-203 through Figure 2.5.2-208). In addition, the February 10, 2006, earthquake (not in the updated catalog but discussed in Subsection 2.5.2.1.3.1) also has a larger magnitude than the source zone that contains it (Figure 2.5.2-204). Two of these events require revisions to Mmax values for some EPRI-SOG source zones (see discussion in Subsection 2.5.2.4.2.2), and the other partially motivates the development of a source zone used in a screening study for CPNPP Units 3 and 4 (see discussion in Subsection 2.5.2.4.2.3.3).

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.4 with the following.

Subsection 2.5.2.4 describes the Probabilistic Seismic Hazard Analysis (PSHA) conducted for the Comanche Peak nuclear site. Subsection 2.5.2.4.1 discusses the basis for the PSHA, which is the 1989 EPRI study (Reference 2.5-370). This follows the procedures recommended in Regulatory Guide 1.208. Next, Subsection 2.5.2.4.2 presents investigations that were undertaken to revise seismic sources in the EPRI study. These investigations include updates to the seismicity catalog of historical earthquakes, updates to maximum magnitudes assigned to seismic sources in the 1989 EPRI study, and new seismic sources that were identified for inclusion in the seismic hazard calculations. Subsection 2.5.2.4.3 discusses new ground motion equations that were used to update the seismic hazard calculations. Subsection 2.5.2.4.4 presents the results of these revisions to the PSHA in the form of hard rock uniform hazard response spectra (UHRS) and deaggregation analyses. Next, Subsection 2.5.2.5 presents seismic wave transmission characteristics of the site. Finally, Subsection 2.5.2.6 presents horizontal and vertical ground motion response spectra (GMRS) and FIRS for various elevations.

2.5.2.4.1 1989 EPRI-SOG Probabilistic Seismic Hazard Analysis

The starting point for probabilistic seismic hazard calculations is the EPRI-SOG study that was fully documented in 1989 (Reference 2.5-370). This follows the recommendation of Regulatory Guide 1.208 (Reference 2.5-369). An underlying principle of the EPRI-SOG study is that expert opinion on alternative, competing models of earthquake occurrence (size, location, and rates of occurrence) and of ground motion amplitude and its variability should be used to weight alternative hypotheses. Interpretations of seismic sources and seismicity parameters were made in the EPRI-SOG study using the six ESTs discussed in Subsection 2.5.2.2.1: Bechtel Corporation, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants.

Seismic hazard at a site for each team's interpretation is calculated separately, and combined results are determined by weighting each team equally. The result is a family of weighted seismic hazard curves from which composite hazard curves, including the mean and fractile seismic hazard, can be derived.

The initial task in this COLA is to replicate the seismic hazard calculated for CPNPP Units 1 and 2 using the assumptions on seismic sources and ground motion equations developed in the EPRI-SOG study. This task is undertaken to ensure that seismic sources are modeled correctly for this COLA and that the software being used (Risk Engineering, Inc.'s FRISK88 software) can accurately reproduce the EPRI-SOG results.

Comparisons of hazards calculated from the EPRI-SOG study with those calculated here are shown in Tables 2.5.2-208 and 2.5.2-209, for peak ground acceleration (PGA) and 1 Hz spectral velocity, respectively. For hazards (annual frequencies of exceedance) in the range of 10^{-4} to 10^{-5} (the first two rows of numbers in each table), differences in mean hazard are less than 10%, with the 2007 calculations for this COLA showing higher (more conservative) hazards than the EPRI-SOG results. For the median and 0.85 fractile, and for higher amplitudes (lower hazards), the differences are larger, with the 2007 results generally showing larger hazards than the EPRI-SOG results. These differences are of less concern, because only mean hazards in the range of 10^{-4} to 10^{-5} are used to develop spectra recommended for seismic design.

The conclusion from these comparisons is that the EPRI-SOG hazard calculations can be reproduced within about 10% accuracy, and estimates are conservative, for mean hazards at ground motion levels corresponding to hazard levels used to recommend design spectra. For other hazards (corresponding to higher ground motions and to median and 0.85 fractile hazards), the 2007 calculations for this COLA are less consistent but are generally conservative (indicate higher hazard). This comparison validates the FRISK88 code, the representation of EPRI-SOG seismic sources, the EPRI-SOG source combinations, and the EPRI-SOG attenuation equations.

2.5.2.4.2 Revisions to 1989 EPRI-SOG Probabilistic Seismic Hazard Analysis

Several types of new information on the sources of earthquakes may require changes in inputs to PSHA, resulting in changes in the level of seismic hazard at the CPNPP Units 3 and 4 site compared to what would be calculated based on the EPRI-SOG evaluation. Seismic source characterization data and information that could affect the calculated level of seismic hazard include:

- Effects caused by an updated earthquake catalog and resulting changes in the characterization of the rate of earthquake occurrence as a function of magnitude for one or more seismic sources.
- Identification of possible new seismic sources in the site vicinity.
- Changes in the characterization of the maximum magnitude for seismic sources.

- Changes to models used to estimate strong ground shaking and its variability in the central and eastern US.

Possible changes to seismic hazard caused by changes in these areas are addressed in the following subsections.

2.5.2.4.2.1 Updated Seismicity Catalog

Subsection 2.5.2.1.2 describes the development of an updated earthquake catalog. This updated catalog documents additional earthquakes through 2006 that have occurred after the earthquake compilation for the EPRI-SOG study (which went through 1984). The impact of the new catalog information is investigated by examining the effect of the new earthquake data on earthquake recurrence estimates within a several-hundred-kilometer region around the CPNPP Units 3 and 4 site.

The effect of the updated earthquake catalog on earthquake occurrence rates in the local region around the CPNPP Units 3 and 4 site is assessed by computing earthquake recurrence parameters for two test areas shown in Figure 2.5.2-209. Test Area 1 consisted of a rectangular area encompassing seismicity in the vicinity of the site, with dimensions 4° latitude by 5.5° longitude. These dimensions are chosen to encompass historical seismicity in the vicinity of the site, and because local events within 100 km of the site generally dominate the hazard (with the exception of the New Madrid seismic zone, which is treated separately) (see Subsection 2.5.2.4.4). Test Area 2 consists of a region north of the site encompassing historical earthquakes in north Texas and Oklahoma, which shows higher historical seismicity than the region surrounding the CPNPP Units 3 and 4 site.

For both test areas, the truncated exponential recurrence model is fit to historical seismicity data using the EPRI EQPARAM program, which uses the maximum likelihood technique. Earthquake recurrence parameters are computed first using the original EPRI catalog and periods of completeness, and then using the updated catalog and extending the periods of completeness to 2006, assuming that the probability of detection for all magnitudes is unity for the time period 1985 to 2006. The resulting earthquake recurrence rates are compared in Figure 2.5.2-210 for Test Area 1 and in Figure 2.5.2-211 for Test Area 2. Both figures show that the extended earthquake catalog results in earthquake recurrence rates that are lower than rates from the original earthquake catalog.

On the basis of the comparison shown in Figures 2.5.2-210 and 2.5.2-211, it is concluded that the earthquake occurrence rate parameters developed in the EPRI-SOG study for seismic sources are conservative estimates of what would be calculated if the extended catalog were to be used to recalculate earthquake occurrence rates. As a result of this conclusion, the original EPRI-SOG earthquake rate parameters are used for EPRI-SOG seismic sources to make hazard estimates for the CPNPP Units 3 and 4 site. Treatment of earthquake rate

parameters for other seismic sources, specifically the New Madrid seismic source, is addressed in Subsection 2.5.2.4.4 below.

2.5.2.4.2.2 New Maximum Magnitude Information

Geologic and seismological data published since the EPRI-SOG study for the site region and more distal areas are summarized and discussed in Subsection 2.5.1 and Subsection 2.5.2.1.2. A review of these data has shown that there is no basis for updating the Mmax distributions of the EPRI-SOG source zones used for the PSHA at CPNPP Units 3 and 4 (Table 2.5.2-202 through 2.5.2-207), with the exception of Dames & Moore's South Coastal Margin (zone 20) and Law Engineering's New Mexico-Texas Block (zone 124). The basis for these updates is that earthquakes have occurred since the EPRI-SOG study (see discussion in Subsection 2.5.2.1 and Subsection 2.5.2.3) within these source zones that have magnitudes greater than the lower-bound Mmax magnitudes for these zones. The update to the Mmax values for these source zones consists of raising the lower-bound Mmax value for the two zones and is discussed in the following subsections.

In addition to these two earthquakes, another earthquake, the April 14, 1995, event, occurred within several source zones with lower-bound Mmax values less than the magnitude of the earthquake. This occurrence could be interpreted as justification for updating the Mmax of these EPRI-SOG source zones. However, accounting for the seismotectonic environment and seismic hazard potential reflected by this earthquake is best done through the addition of a new source zone for CPNPP Units 3 and 4. This event, the potentially affected source zones, and development of the new source zone are described in Subsection 2.5.1.1.4.3.7.1 and Subsection 2.5.2.4.2.3.3.

2.5.2.4.2.2.1 Mmax Update for Dames & Moore South Coastal Margin

The Dames & Moore South Coastal Margin (zone 20) is characterized by a Mmax distribution of m_b 5.3 (0.8) and m_b 7.2 (0.2), with weights shown in parentheses (Table 2.5.2-210). On February 10, 2006, an earthquake of magnitude Ms 5.3 (References 2.5-377 and 2.5-381) occurred within this source zone (Figure 2.5.1-204). The earthquake occurred within a region of the Gulf of Mexico with relatively poor seismograph station coverage. However, at the time of the event an ocean-bottom seismometer array was deployed near the earthquake allowing for a relatively good determination of the earthquake epicenter. The earthquake occurred well outside the extent of the updated catalog, so an Emb magnitude for the event is not listed in Table 2.5.2-201, but an Emb magnitude of 5.5 is calculated for the event using the relationship between Ms and Emb reported in Table 4-1 of EPRI (Reference 2.5-340) as described in Subsection 2.5.2.1.2. Since the Emb 5.5 magnitude is greater than the lower-bound m_b 5.3 magnitude of the zone, the Mmax distribution for the zone needs to be updated.

The methodology used by Dames & Moore in determining the Mmax distribution for the South Coastal Margin source zone is not explicitly stated in the EPRI-SOG

documentation (References 2.5-369 and 2.5-335). Given the lack of a documented methodology, an updated Mmax distribution is developed by increasing the lower-bound Mmax of the South Coastal Margin source zone to m_b 5.5 while maintaining the original weights. The updated Mmax distribution is presented in Table 2.5.2-210.

2.5.2.4.2.2.2 Mmax Update for Law Engineering New Mexico-Texas Block

The Law Engineering New Mexico-Texas Block (zone 124) is characterized by a Mmax distribution of m_b 4.9 (0.3), 5.5 (0.5), and 5.8 (0.2) with weights shown in parentheses (Table 2.5.2-210). On January 2, 1992, an earthquake with an Emb magnitude of 5.0 occurred in the southeast corner of New Mexico. This event is located well within the boundaries of the Law Engineering New Mexico-Texas Block (zone 124) (Figure 2.5.2-201 and Figure 2.5.2-205). Because the Emb magnitude of this event is greater than the lower-bound Mmax for this zone, the Mmax distribution needs to be revised.

The Law Engineering methodology for developing the New Mexico-Texas Block Mmax distribution is not explicitly stated within the EPRI-SOG study documentation (References 2.5-369 and 2.5-335). However, the 1986 volume for Law Engineering (Reference 2.5-369) does indicate that the 5.8 upper-bound Mmax is based on observations of seismicity within the zone, and that the lower-bound 4.9 is the maximum observed earthquake magnitude within the zone (EPRI, 1986). Based on these statements, the Mmax distribution is updated by increasing the lower-bound Mmax value to 5.0 and maintaining the remaining Mmax values and original weights. A summary of the updated New Mexico-Texas Block is shown in Table 2.5.2-210.

Law Engineering assigned Mmax values of 4.6 and 4.9 to the South Coastal Block Source Zone (Zone 126) (Table 2.5.2-210). The 2006 Emb 5.5 and Emb 6.1 earthquakes within the Gulf of Mexico (see Subsection 2.5.2.1.3.1) are 39 mi (63 km) and 97.6 mi (157 km) outside this zone, respectively. The Emb 6.1 earthquake was well recorded and clearly lies outside the source zone (Reference 2.5-478). The Emb 5.5 earthquake was not well recorded and attempts at relocating the event using proprietary data from ocean bottom seismographs have resulted in significant (10s of kilometers) variation in the position of the earthquake epicenter (Reference 2.5-479). Although current published locations of the Emb 5.5 earthquake locate it outside the source zone boundaries, the uncertainty in the epicentral location of the earthquake is such that it could have occurred within the source zone. The earthquake is conservatively assumed to have occurred within the South Coastal Block Zone. Because the Emb 5.5 earthquake is larger than the lower bound Mmax value of the South Coastal Block Source Zone, the Mmax distribution has been revised accordingly.

The updated Mmax values of 5.5 and 5.7 adopted here (Table 2.5.2-210) are derived using Law Engineering's methodology for developing Mmax distributions as follows (Reference 2.5-369):

- The lower bound Mmax is the magnitude of the maximum observed earthquake in the zone
- The upper bound Mmax magnitude defined by Law Engineering for regions with earthquakes occurring within 6.2 mi (10 km) of the surface is mb 5.7

Weights for the original Mmax distribution (0.9 on the lower bound Mmax and 0.1 on the upper bound Mmax) are retained in the updated Mmax distribution (Table 2.5.2-210).

2.5.2.4.2.2.3 Mmax Update for Bechtel Gulf Coast

The Bechtel Group assigned Mmax values of 5.4, 5.7, 6.0, and 6.6 to the Gulf Coast source zone (zone BZ1) (Table 2.5.2-210). Because the 2006 Emb 5.5 and Emb 6.1 earthquakes in the Gulf of Mexico occur well within this zone (Figure 2.5.2-204), and because these magnitudes are greater than the lower-bound Mmax values for the source zone, the Mmax distribution for this source zone has been updated.

The Bechtel Group's methodology for defining Mmax distributions is described within their EST volume (Reference 2.5-369) and can be applied to Zone BZ1 as follows (Table 2.5.2-210):

- The lower bound magnitude of the distribution is defined as the greater of the largest observed earthquake within the zone or m_b 5.4. For Zone BZ1, this lower-bound Mmax value is m_b 6.1 with a weight of 0.1.
- The next higher magnitude is 0.3 magnitude units greater than the minimum and is given a weight of 0.4. For Zone BZ1, this results in a Mmax value of m_b 6.4 with a weight of 0.4.
- The third magnitude is mb 6.6, interpreted by the Bechtel EST as the largest intraplate earthquake in the CEUS with specific exceptions, and is given a weight of 0.1.
- The fourth magnitude is 0.6 magnitude units above the minimum and is given a weight of 0.4. For Zone BZ1, this results in a Mmax value of mb 6.7 with a weight of 0.4.

2.5.2.4.2.2.4 Mmax Update for Rondout Gulf Coast to Bahamas Fracture Zone

Rondout Associates assigned Mmax values of 4.8, 5.5, and 5.8 to the Gulf Coast to Bahamas Fracture Zone source zone (zone 51) (Table 2.5.2-210). Because both the 2006 Emb 5.5 and Emb 6.1 earthquakes in the Gulf of Mexico occur within this zone, and because these magnitudes are greater than the lowest

Mmax values for the source zone, the Mmax distribution for this source zone has been updated.

The updated Mmax values of 6.1, 6.3, and 6.5 with weightings of 0.3, 0.55, and 0.15, respectively, used here (Table 2.5.2-210) follow from reclassifying the source zone as one capable of producing moderate earthquakes instead of the original classification of the source zone as one only capable of producing smaller than moderate earthquakes (Reference 2.5.2-369). The original Rondout Mmax distribution for moderate earthquake source zones is 5.2, 6.3, and 6.5 with weightings of 0.3, 0.55, and 0.15, respectively. The updated Mmax distribution follows this distribution with the exception of an increase in the lower bound of the distribution to 6.1 to account for the observed Emb 6.1 earthquake within this zone.

2.5.2.4.2.2.5 Mmax Update for Weston Gulf Coast

Weston Geophysical Corporation assigned Mmax values of 5.4 and 6.0 to the Gulf Coast source zone (zone 107) (Table 2.5.2-210). Both the 2006 Emb 5.5 and Emb 6.1 earthquakes in the Gulf of Mexico occur within this zone. Because these magnitudes are greater than the original Mmax values for the source zone, the Mmax distribution for this source zone has been revised.

Weston Geophysical Corporation's (Reference 2.5.2-369) methodology for defining Mmax is based on developing discrete distributions for the probability of Mmax being a particular value. For the Gulf Coast source zone, these Mmax values and probabilities determined by the Weston Geophysical Corporation EST are: 3.6 (0.04628), 4.2 (0.11982), 4.8 (0.27542), 5.4 (0.34415), 6.0 (0.16169), 6.6 (0.04461), and 7.2 (0.00553) (Reference 2.5.2-369). Conservatively applying the Weston Geophysical Corporation's methodology, this discrete probability distribution is truncated at the magnitude that is closest to, yet greater than, the maximum observed earthquake within the source zone. For this study the distribution is truncated at 6.6 because the Emb 6.1 earthquake occurred within the source zone, and the next highest discrete magnitude in the distribution is 6.6. The truncated distribution is then renormalized so that the sum of all the probabilities is 1.0. The final Mmax values are the truncated distribution, and the weights are the renormalized probabilities.

2.5.2.4.2.3 New Seismic Source Characterizations

Geologic, geophysical, and seismological information developed since the EPRI-SOG study (Reference 2.5-369) was reviewed to identify seismic sources not included in the original EPRI-SOG screening study for CPNPP Units 1 and 2 that should be evaluated to determine their potential contribution to seismic hazard at CPNPP Units 3 and 4. New seismic source characterizations are developed for four tectonic features thought to have the potential to impact seismic hazard at CPNPP Units 3 and 4. These features are the New Madrid Seismic Zone (NMSZ), the Meers fault, the Rio Grande Rift (RGR), and the Cheraw fault (Figure 2.5.2-212). The development of seismic source

characterizations for these features is described in Subsection 2.5.2.4.2.3.1 through Subsection 2.5.2.4.2.3.4 based on the post-EPRI-SOG information summarized in Subsection 2.5.1.1.4.3.6. Source characterizations developed since the EPRI-SOG study commonly use moment magnitude (Mw) to describe earthquake magnitude, whereas the EPRI-SOG study used body-wave magnitude (m_b). To allow comparisons between these magnitudes, both m_b and Mw magnitudes are reported below. To convert between the two magnitude scales, the arithmetic mean of the magnitude conversions reported in Atkinson and Boore (Reference 2.5-386), Frankel, et al. (Reference 2.5-339), and EPRI (Reference 2.5-387) are used.

2.5.2.4.2.3.1 New Madrid Seismic Zone

The NMSZ extends from southeastern Missouri to southwestern Tennessee and is located approximately 500 mi northeast of CPNPP Units 3 and 4 (Figure 2.5.2-212). The NMSZ produced a series of large-magnitude earthquakes between December 1811 and February 1812 (Reference 2.5-328). Subsection 2.5.1.1.4.3.7.3 presents a detailed discussion of the NMSZ. In brief, several post-EPRI-SOG studies demonstrate that the source parameters for geometry, Mmax, and recurrence of Mmax in the New Madrid region need to be updated to capture the current understanding of this seismic source (References 2.5-321, 2.5-328, 2.5-329, 2.5-330, 2.5-336, and 2.5-393).

The original EPRI-SOG screening study for CPNPP Units 1 and 2 did not show any New Madrid source zones from the EPRI-SOG ESTs as contributing to 99% of the hazard (Reference 2.5-370). However, with the updated geometry, Mmax values and recurrence intervals for the New Madrid source and updated ground motion attenuation relations developed for the CEUS require reevaluation of the NMSZ as a potential contributor to seismic hazard at CPNPP Units 3 and 4.

The updated New Madrid seismic source model described in the Early Site Permit (ESP) application for the Exelon Generation Company ESP site near Clinton, Illinois (Reference 2.5-395) (Figure 2.5.2-213 and Figure 2.5.2-402) and as modified for the Tennessee Valley Authority Bellefonte Nuclear Site COLA (Reference 2.5-402) is the basis for the NMSZ source model used here for CPNPP Units 3 and 4. This source model accounts for new information on recurrence intervals for large earthquakes in the New Madrid area, for recent estimates of possible earthquake sizes on each of the active faults, and for the possibility of multiple earthquake occurrences within a short period of time (earthquake clusters).

The time-dependent treatment of the NMSZ is the same as the treatment used in the Bellefonte FSAR, which used a combination of the Poisson model and a Brownian passage time (BPT) model. The Bellefonte FSAR used a cluster model for earthquake occurrences, and gave Cluster Model A a weight of 1.0 and Cluster Model B a weight of 0.0, and this interpretation is followed for the Comanche Peak FSAR.

Within this model, three faults are identified in the NMSZ, each with two alternative geometries, as follows (Figure 2.5.2-212):

Fault	Geometry
Blytheville	Blytheville arch/Bootheel lineament Blytheville arch/Blytheville fault zone
Northern	New Madrid north New Madrid north with extension
Reelfoot	Reelfoot central section Reelfoot full length

Also, earthquakes are treated as characteristic events in terms of magnitudes, with the following sets of magnitudes modeled for each fault (Reference 2.5-395):

Blytheville		Reelfoot		Northern		Weight
Mw	m_b	Mw	m_b	Mw	m_b	
7.3	7.1	7.5	7.2	7.0	6.9	0.1667
7.2	7.0	7.4	7.2	7.0	6.9	0.1667
7.2	7.0	7.4	7.2	7.2	7.0	0.0833
7.6	7.3	7.8	7.4	7.5	7.2	0.25
7.9	7.4	7.8	7.4	7.6	7.3	0.1667
7.8	7.4	7.7	7.3	7.5	7.2	0.1667

The above magnitudes represent the centers of characteristic magnitude ranges that extend ± 0.25 moment magnitude units above and below the indicated magnitude.

Seismic hazard is calculated considering the possibility of clustered earthquake occurrences. The modeling of earthquake clusters in the NMSZ has undergone considerable study, and this model will continue to evolve as further field evidence on paleo-earthquakes is found and analyzed. In the Exelon cluster model for multiple earthquake occurrences, the possibility of three clustered earthquakes is taken into account, as is the possibility of clustered earthquakes on two of the faults (but not the third), or the possibility of two faults generating a characteristic earthquake magnitude and the third fault generating a smaller magnitude. The cluster model used for CPNPP Units 3 and 4 is a conservative simplification of the Exelon model (Reference 2.5-395) in that hazard is computed assuming that all clustered events generate earthquakes on each of the three faults and that the magnitudes of those events correspond to the characteristic magnitude distribution.

Consistent with the Exelon model (Reference 2.5-395), the NMSZ faults used for CPNPP Units 3 and 4 are assumed to be vertical and to extend from the surface to 12 mi (20 km) depth, and a finite rupture model is used to represent an

extended rupture on all faults. An additional simplification was made in that only the preferred geometry of each fault is used. This is justified because of the large distance between CPNPP Units 3 and 4 and NMSZ (approximately 500 mi) and the small differences between the preferred and alternative geometries. This simplification allows efficiency in calculations while providing an accurate estimate of seismic hazard. The final model used here for the NMSZ is the same in all important aspects affecting hazard to the model used in the Tennessee Valley Authority Bellefonte Nuclear Site COLA (Reference 2.5-402).

2.5.2.4.2.3.2 Meers Fault

The Meers fault, the southern boundary of the Frontal Wichita fault system in southern Oklahoma, is approximately 180 mi from CPNPP Units 3 and 4. Two surface-rupturing earthquakes along the fault have occurred in the Holocene (Reference 2.5-278), making the Meers fault the only recognized capable fault within the Frontal Wichita fault system. The potential for Quaternary events on the Meers fault, and in particular these two Holocene events, was identified in research (see summary in Subsection 2.5.1.1.4.3.6.1.1) (References 2.5-389, 2.5-284, 2.5-296, and 2.5-293) that post-dated the development of the EPRI-SOG source models (Reference 2.5-369), and thus this Holocene activity was not taken into account in the EPRI-SOG source models or hazard calculations for CPNPP Units 1 and 2. For CPNPP Units 3 and 4 it is necessary to develop an updated source characterization of the Meers fault.

Following the guidance of RG 1.208, a seismic source characterization of the Meers fault is developed for CPNPP Units 3 and 4 using the SSHAC guidelines for a Level 2 study described in NUREG/CR-6372. The characterization of the Meers fault used for CPNPP Units 3 and 4 is developed from a thorough review of existing literature and consultation with experts familiar with the Meers fault so that the new characterization represents the legitimate range of technically supportable interpretations of the seismic capability of the Meers fault among the informed technical community. A summary of the current state of knowledge regarding the tectonics and seismic capability of the Meers fault, as determined through the literature review and elicitation of expert opinion, is presented in Subsection 2.5.1.1.4.3.6.1.1, and the source model developed from this information is presented below.

As discussed in Subsection 2.5.2.2.2.6, the USGS has developed a seismic source characterization of the Meers fault for use in the USGS National Seismic Hazard Maps. As stated in that subsection, the USGS does not use a formal expert elicitation process and does not explicitly attempt to represent the full uncertainty of source characterizations. However, the source models are developed from the range of published literature and source characterizations are discussed in regional working groups, and as such the USGS source model for the Meers fault is deemed a good base model that is modified to create the updated Meers fault characterization for CPNPP Units 3 and 4.

The USGS characterization of the Meers fault for the 2002 National Seismic Hazard Maps (Reference 2.5-321) is summarized in Table 2.5.2-206. Preliminary documentation for the 2007 National Seismic Hazard Maps (Reference 2.5-392) has the same characterization for the fault. The USGS characterization of the Meers fault is a reasonable representation of the modern state of knowledge regarding the seismic capability of the fault as described in Subsection 2.5.2.2.2.6. However, there is no epistemic uncertainty built into the USGS characterization. In particular, there is considerable uncertainty in the characteristic magnitude, characteristic return period, and fault length that is not included in the USGS source model, so these characteristics are updated for the CPNPP Units 3 and 4 source model. Any uncertainty that exists in the other fault characteristics (e.g., dip, dip direction, sense of slip) does not have a significant impact on hazard at CPNPP Units 3 and 4 due to the considerable distance between the fault and site. The updated Meers fault source model for CPNPP Units 3 and 4 is presented in Table 2.5.2-213.

2.5.2.4.2.3.2.1 Fault Location and Length

The surface trace of the Meers fault used in the updated source model is based on a simplified version of the USGS source model trace that is itself a discretized version of the fault trace from the USGS Quaternary Fault and Fold Database (Reference 2.5-278). The simplification used here (Table 2.5.2-213) uses the two endpoints of the USGS source model (Table 2.5.2-212). The additional fault trace detail provided by the two additional points in the USGS model is insignificant to calculating seismic hazard at CPNPP Units 3 and 4 given the distance between the site and fault.

The distance between the two endpoints of the fault trace is approximately 23 mi (37 km), representing the maximum expected length of the Meers fault Holocene rupture. As discussed in Subsection 2.5.1.1.4.3.6.1.1, the western 16 mi (26 km) of the fault is positively associated with the Holocene rupture, given the mapping of the trace on aerial photographs, the continuous nature of the fault scarp over those 16 mi (26 km), and the trenching studies at different locations along the fault (Figure 2.5.1-211) (References 2.5-289, 2.5-284, 2.5-278, 2.5-281, and NUREG/CR-4852). The easternmost portion of the fault scarp that extends the possible length of the Holocene scarp to 23 mi (37 km) was identified in low-sun-angle aerial photography (Figure 2.5.1-211) and is more subtle and discontinuous (NUREG/CR-4852; Reference 2.5-281). Field investigations of this easternmost extent of the scarp have not been conducted to determine if it is from the same Holocene events as is the western extent of the scarp because the area is within the U.S. Army's Fort Sill artillery range. To account for this uncertainty in the length of the Holocene surface ruptures, characteristic magnitudes for the fault are calculated using both 16 and 23 mi (26 and 37 km) as discussed in Subsection 2.5.4.2.3.2.3. However, to simplify the updated Meers fault source model, the location of the fault trace does not include this uncertainty. Not allowing for variations in the extent of fault trace in the source model is a conservative simplification because it allows short-rupture scenarios (i.e., 16-mi fault length

scenarios) to occur closer to CPNPP Units 3 and 4 than if the fault trace also included the uncertainty (Figure 2.5.1-211).

It should be noted that one researcher (Reference 2.5-289) suggests that Quaternary activity on the Meers fault extends 30 km to the northwest of the westernmost extent of the scarp shown in Figure 2.5.1-211. Cetin (Reference 2.5-289) proposes this extension based on "displaced terrace deposits of Pleistocene age, displaced, buried and/or overthickened soil horizons, fault-related colluvium deposits (colluvial wedges) found near and only on the downthrown side of the fault, active seepage near the fault, deflection of stream alignments and the land use pattern along the fault." However, as is summarized by Wheeler and Crone (Reference 2.5-397), the evidence presented by Cetin (Reference 2.5-289) for Quaternary faulting is inconclusive, has not been confirmed by other researchers who have attempted to visit the same field sites as Cetin (Reference 2.5-289), and has never been presented as peer-reviewed research. As such, this potential northwest extension of the capable Meers fault is not considered to be within the legitimate range of technically supportable interpretations.

2.5.2.4.2.3.2.2 Characteristic Magnitude

Previous studies summarized in Subsection 2.5.1.1.4.3.6.1.1 and Subsection 2.5.2.2.2 have characterized the Holocene events on the Meers fault with Mmax on the order of Mw 7.0 (m_b 6.9). Characteristic magnitudes for the updated Meers fault source model are based on using the Holocene events identified on the Meers fault as proxies for the fault's characteristic magnitude. Magnitudes for the Holocene events are estimated using the empirical relationships of Wells and Coppersmith (Reference 2.5-398) between observed earthquake magnitude and characteristics of the earthquake rupture (e.g., surface rupture length, rupture area, maximum surface displacement). For each of the empirical relationships discussed below, the "all faults" regressions of Wells and Coppersmith (Reference 2.5-398) are used to estimate characteristic magnitudes.

<u>Magnitude from Surface Rupture Length</u>

As discussed in Subsection 2.5.1.1.4.3.6.1.1, mapping of the Meers fault scarp on aerial photographs by Ramelli, et al. (NUREG/CR-4852) and other researchers (Reference 2.5-278) indicates that the scarp associated with the Holocene events is between 16 and 23 mi (26 and 37 km) long (Figure 2.5.2-202). Because of this uncertainty in the length of the Holocene surface rupture, both 16 and 23 mi (26 and 37 km) are used with the regressions of Wells and Coppersmith (Reference 2.5-398) to estimate magnitude. Using the regression between rupture length and moment magnitude for all faults, estimated characteristic event magnitudes are:

- Mw 6.7 (m_b 6.7) for a 16-mi (26-km) long rupture; and
- Mw 6.9 (m_b 6.9) for a 23-mi (37-km) long rupture.

Magnitude from Rupture Area

Rupture area for the Holocene ruptures of the Meers fault is estimated using the length of the scarp and the downdip width of the rupture, itself a function of the fault dip and depth of rupture bottom. The lengths of 16 and 23 mi (26 and 37 km) from above are used for rupture length. The dip of the Meers fault is taken from USGS source model, with an 89° dip to the southwest (Reference 2.5-321). The near-vertical orientation of the fault is supported by exposures of the fault in trenches, but the dip of the fault at depth is poorly constrained (Reference 2.5-280). The depth of the rupture bottom is taken as 9 to 12 mi (15 to 20 km) based on NUREG/CR-6034 that reports there is no indication of earthquakes occurring within Oklahoma at greater depths. Using the regressions of Wells and Coppersmith (Reference 2.5-398) between rupture area and moment magnitude for all faults results in the following values:

- Mw 6.6 (m_b 6.7) for the minimum rupture area of 9 mi x 16 mi = 144 mi^2 (15 km x 26 km = 390 km^2); and
- Mw 6.9 (m_b 6.9) for the maximum rupture area of 12 mi x 23 mi = 276 mi^2 (20 km x 37 km = 740 km^2).

Magnitude from Maximum Surface Displacement

The best estimates of surface displacement per event on the Meers fault come from the study of Swan, et al. (Reference 2.5-389) reviewed in Subsection 2.5.1.1.4.3.6.1, and these estimates are used with the regressions of Wells and Coppersmith (Reference 2.5-398) to estimate characteristic magnitudes. The regressions of Wells and Coppersmith (Reference 2.5-398) were determined using net surface displacements, and because the Meers fault exhibits oblique slip there is only one combined observation of vertical and lateral displacement with which net displacement can be determined (7.5 ft or 2.29 m per event). However, Swan, et al. (Reference 2.5-389) report a best estimate of vertical displacement at a different location that is greater than this net displacement (8.5 ft or 2.6 m per event). Both of these displacement values are used to estimate characteristic magnitudes for the Meers fault.

The regression on maximum surface displacement, and not the regression for the average surface displacement, of Wells and Coppersmith (Reference 2.5-398) is used to estimate magnitude because the average surface displacement regression is not appropriate for the displacement data available for the Meers fault. Wells and Coppersmith (Reference 2.5-398) explicitly state that the regression for maximum displacement was determined using the maximum reported displacement for an event, while the regressions for average displacement were done on faults where an average displacement was calculated from either an extensive study of the entire surface rupture or a minimum of 10 displacement measurements. The data available for the Meers fault is a maximum reported displacement and not an along-fault average.

Using the displacements described above results in the following magnitude estimates:

- Mw 7.0 (m_b 6.9) from a maximum vertical displacement of 8.5 ft (2.6 m); and
- Mw 7.0 (m_b 6.9) from a maximum net displacement of 7.5 ft (2.29 m).

Final Magnitude Distribution

The final characteristic magnitude distribution used for the Meers fault is: Mw 6.7 (m_b 6.7), Mw 6.85 (m_b 6.82), and Mw 7.0 (m_b 6.9) with weights 0.2, 0.6, and 0.2, respectively (Figure 2.5.2-259). Mw 6.7 (m_b 6.7) is chosen as the lower bound instead of Mw 6.6 (m_b 6.7) because it is not considered likely that only the 26 km of the Meers fault scarp is related to the Holocene ruptures. Mw 7.0 (m_b 6.9) is chosen as the maximum bound because it is the maximum estimated magnitude of any regression and it is roughly equivalent to other estimates of characteristic earthquake magnitude for the fault (References 2.5-389 and 2.5-321). The weighting of the distribution reflects the opinion that the best estimates of magnitude come from regressions on surface rupture length and rupture area.

2.5.2.4.2.3.2.3 Characteristic Return Period

Epistemic uncertainty in return periods for characteristic earthquakes on the Meers fault is implemented through return period branches on a logic tree (Figure 2.5.2-259). The data presented by Swan, et al. (Reference 2.5-389) on the timing of Meers earthquakes suggests that there have been two Holocene events preceded by a long period (greater than 200,000 years) of inactivity, indicating that the Meers fault exhibits clustered earthquake behavior. The initial branch of the logic tree represents uncertainty in whether or not the Meers fault is in an earthquake cluster.

Weightings of 0.9 and 0.1 are used for the logic tree branches describing the Meers fault as in an earthquake cluster or in-between earthquake clusters, respectively. High weighting on the “in earthquake cluster” conservatively reflects the observation that there is no information to suggest that the Meers fault is not in a cluster; insufficient time has elapsed since the most recent event to conclude that there is a moderate possibility that the period of increased Holocene activity has passed. Return periods for the inter-cluster branch are based on the work of Swan, et al. (Reference 2.5-389) that estimates a minimum period of inactivity prior to the Holocene ruptures of 200,000 to 500,000 years. Based on this observation, return period branches of 500,000, 350,000, and 200,000 years with weights of 0.2, 0.6, and 0.2, respectively, are used for the inter-cluster branch (Figure 2.5.2-259).

Return periods for the intra-cluster branch are based on the elapsed time since the oldest Holocene event and the observation of two earthquakes during that

time span. Assuming that the Meers fault is currently in an earthquake cluster, this method results in a reasonable estimate of the intra-cluster return period. Swan, et al. (Reference 2.5-389) report two dates to constrain the maximum age of the oldest Holocene rupture: sample PITT-0477 with a calibrated age of 3397 years B.P and sample PITT-0373 with a calibrated age of 2918 years B.P. The mean of these two ages is taken as the most-probable maximum age of the event, and half that age (1580 years) is taken as the most-probable maximum return period for intra-cluster events. Swan, et al. (Reference 2.5-389) also report four ages that they believe best constrain the minimum age of the oldest Holocene event: PITT-0370 with a calibrated age of 1942 years B.P., PITT-0369 with a calibrated age of 1610 years B.P., PITT-0378 with a calibrated age of 1912 years B.P., and PITT-0478 with a calibrated age of 2093 years B.P. The mean of these four ages is taken as the most-probable minimum age of the event, and half the age (950 years) is taken as the most-probable minimum return period for intra-cluster events.

A direct inter-event return period for the two Holocene events can also be determined from ages reported by Swan, et al. (Reference 2.5-389) as constraining the bounds of the oldest and youngest Holocene events. The return period determined using the time elapsed between the mean upper-bound age of the oldest Holocene event and the mean lower-bound age of the youngest Holocene event is 2000 years. The return period determined using the time elapsed between the mean lower-bound age of the oldest Holocene event and the mean upper-bound age of the youngest Holocene event is 300 years. The large range in return period determined using this methodology is due to the compounded uncertainty from using the dates constraining both Holocene events as opposed to just the time elapsed since the oldest event. The 300-year lower-bound return period is unrealistic since it would imply significantly more events between the oldest Holocene event and the present time than the two observed. For this reason, and because the plausible range of return periods determined from the inter-event period is captured in the return periods previously described, the inter-event period is not used to estimate return periods.

The most probable minimum and maximum return periods are both given equal weight of 0.2 in the logic tree for the return period of intra-cluster events. The remaining 0.6 weight is given to the median of the most-probable minimum and maximum return periods (1265 years) (Figure 2.5.2-259). This weighting reflects the belief that it is most likely for the intra-cluster return period to be somewhere between the minimum and maximum bounds.

2.5.2.4.2.3.2.4 PSHA Implementation of Updated Meers Fault Source

The updated source characterization for the Meers fault developed for CPNPP Units 3 and 4 is shown in Table 2.5.2-213, Figure 2.5.2-211, and Figure 2.5.2-259. This characterization is implemented in the CPNPP Units 3 and 4 PSHA model as a line source extending to 9.3 mi (15 km) depth. The possibility of ruptures extending to 20 km depth is taken into account in estimating characteristic earthquake magnitudes, but ruptures in the PSHA do not extend to 20 km. This

potential discrepancy does not affect the ground-motion estimates at CPNPP Units 3 and 4 given the large distance between the Meers fault and the site.

2.5.2.4.2.3.3 Rio Grande Rift

The RGR is a north-south-trending continental rift system recognized to extend from central Colorado through New Mexico, Texas, and into northern Mexico (References 2.5-297, 2.5-298, 2.5-299, and 2.5-300). The RGR is generally characterized by north- to north-northwest-trending grabens centered on a broad topographic high, a well-defined gravity high, elevated heat flow, and a tensile stress regime (References 2.5-300, 2.5-313, 2.5-310, and 2.5-296) (see discussion in Subsection 2.5.1.1.4.3.7.1). At the time of the EPRI-SOG study, relatively little was known about the seismogenic potential of faults within the RGR, and only the Weston EST explicitly included the RGR as a seismic source zone. Other ESTs either (1) did not extend their source model boundaries to include the RGR, or (2 included the RGR in large background source zones (Reference 2.5-369). Research post-dating the EPRI study has documented previously unrecognized late Quaternary fault activity within parts of the RGR (References 2.5-303, 2.5-309, 2.5-302, 2.5-304, 2.5-305, 2.5-306, and 2.5-307), as well as evidence that the RGR extends into southwestern Texas and northern Mexico (References 2.5-296 and 2.5-301). These post-EPRI-SOG studies indicate that the RGR is a zone of distinct and elevated tectonic activity relative to other regions at a similar distance from CPNPP Units 3 and 4. Therefore, despite the greater than 400-mi distance between the RGR and CPNPP Units 3 and 4 (Figure 2.5.2-213), RGR sources should be included in a screening study to determine their potential contribution to hazard at CPNPP Units 3 and 4.

Two independent and complementary seismic characterizations of the RGR are developed to characterize the potential contribution to hazard at CPNPP Units 3 and 4. Because of the great distance between the RGR and CPNPP Units 3 and 4 and the intent of using these sources in a screening study for CPNPP Units 3 and 4, these characterizations are simple in comparison to the source model developed for the Meers fault. The first model of the RGR represents discrete faults within the RGR that have been characterized within the USGS National Seismic Hazard Map program (Reference 2.5-321). The second model of the RGR is a point source that generates earthquakes with the bulk characteristics of fully characterized capable faults within the RGR (e.g., magnitude, recurrence rate) at the closest position of the RGR to the CPNPP Units 3 and 4 site.

2.5.2.4.2.3.3.1 RGR Fault Source Characterization

The fault source characterization of the RGR is based on a conservative simplification of the USGS representation of RGR faults in the National Seismic Hazard Maps (Reference 2.5-321). For the National Seismic Hazard Maps, the USGS characterizes the seismic behavior of 41 RGR faults (Table) (Figure 2.5.2-213). These characterizations are based on the USGS compilation of Quaternary folds and faults within the U.S. (Reference 2.5-399). As with all USGS source characterizations, a formal expert opinion elicitation process is not followed and

the characterization is not designed to represent the full uncertainty of source characterizations. However, the source models are developed from published literature, and source characterizations are discussed in regional working groups. As such, the USGS source models are a good characterization of the RGR faults for the CPNPP Units 3 and 4 screening study. It should be noted that the preliminary documentation of the 2007 update to the USGS National Seismic Hazard Maps (Reference 2.5-392) does not indicate any changes to the characterization of these faults that would significantly affect seismic hazard at CPNPP Units 3 and 4.

The USGS characterization includes alternative models of fault recurrence behavior including truncated Gutenberg-Richter and characteristic earthquake relationships. For CPNPP Units 3 and 4, the USGS characterization is simplified by assuming only a characteristic earthquake recurrence relationship parameterized by the characteristic recurrence rate and characteristic earthquake magnitude taken from the USGS parameterization of the faults (Reference 2.5-321). Uncertainty is added to the characteristic magnitudes using a magnitude distribution of ± 0.2 magnitude units about the USGS- reported magnitude with weightings of 0.2, 0.6, and 0.2 for the lowest to highest magnitudes. The surface trace of each fault is simplified from the USGS description by using only the endpoints of the fault trace (Table). Table summarizes this model. These characterizations are implemented into the CPNPP Units 3 and 4 PSHA model as vertical line sources extending to 15 km depth. Given the large distance between the RGR faults and CPNPP Units 3 and 4, details of the geometry do not have a significant impact on ground motions at the site.

2.5.2.4.2.3.3.2 RGR Point-Source Simplification

The fault source characterization of the RGR captures the potential seismic hazard at CPNPP Units 3 and 4 from only faults within the RGR that have been identified as active within the Quaternary and that have been studied in enough detail to develop a seismic source characterization of the fault. As discussed in Subsection 2.5.1.1.4.3.7.1, in addition to these faults the RGR is characterized by a larger-scale lithospheric expression (elevated topography, long-wavelength gravity anomaly, elevated heat flow, tensile stress regime, region of thinned crust and elevated mantle) (References 2.5-300, 2.5-313, 2.5-310, 2.5-296, 2.5-314, 2.5-315, 2.5-241, and 2.5-245). The observation of the extended area of the lithospheric-scale structure of the RGR compared to the surficial expression of RGR faults suggests that the processes driving Quaternary faulting and seismic activity may extend beyond, and in particular to the east of, the observed faults (Reference 2.5-316). This interpretation is supported by the April 14, 1995, earthquake (see discussion in Subsection 2.5.1.1.4.3.7.1 and Subsection 2.5.2.1.3.1).

Any potentially capable faults within this larger region of the RGR are at a minimum between approximately 300 and 400 miles (480 and 640 km) from the CPNPP Units 3 and 4 site, and are located within the southern portion of the Big Bend region of Texas. Some faults within this region have been hypothesized to

have had Quaternary activity based on limited reconnaissance level studies (e.g., mapping from aerial photos); however, there has been little to no work conducted to either confirm initial observations or develop source characterizations for the faults (e.g., recurrence rates, probability of activity, characteristic magnitudes) (References 2.5-301, 2.5-440, 2.5-441, 2.5-442, 2.5-443, 2.5-444 and 2.5-446). Given the great distance between the site and these potentially capable, yet unconfirmed and uncharacterized, faults, the point-source model was developed to determine whether a fault with the bulk characteristics of the identified, capable RGR faults at the closest distance possible to the site has any significant impact on the site hazard. If there is a significant contribution from this point-source characterization, further investigations of potentially capable RGR faults would be required. However, as discussed in Subsection 2.5.2.4.4, none of the RGR faults or the point-source contributes to the site hazard, so no additional studies were conducted.

The closest extent of the RGR to CPNPP Units 3 and 4 is determined by defining the probable easternmost extent of the lithospheric scale structure of the rift, and then determining the closest point of that line to CPNPP Units 3 and 4 (Figure 2.5.2-213). The position of the line is based on the extent of thinned crust related to the RGR (Reference 2.5-314), the relationship between topography and gravitational potential energy thought to drive RGR-related deformation (References 2.5-245, 2.5-320, 2.5-311, and 2.5-316), the extent of the region of tensile stress (References 2.5-241 and 2.5-245), and the location of RGR-related earthquakes (References 2.5-289 and 2.5-319). Essentially, the easternmost extent of each of these features roughly correlates to the distinct decrease in topography from the RGR to the Great Plains. The closest point on this line to CPNPP Units 3 and 4 is located in the Big Bend region of western Texas over 300 mi from CPNPP Units 3 and 4 (Figure 2.5.2-201).

The source characterization of the RGR point source is based on the bulk characteristics of RGR-related faults within the USGS National Seismic Hazard Map database from 2002 (Reference 2.5-321). Preliminary documentation for the 2007 update to that database does not include any changes that would significantly affect seismic hazard at CPNPP Units 3 and 4 (Reference 2.5-392). The magnitude and return period distributions for the RGR point-source model are developed by assuming the 41 characteristic magnitudes and return periods defined by the USGS for the RGR faults represent the distribution of characteristic earthquake magnitudes and return period for the RGR. As such, the observed distributions are used to derive simplified representative distributions for use in the updated characterization.

The observed characteristic magnitudes for RGR faults are shown in Table 2.5.2-214 and vary between Mw 6.1 (m_b 6.3) and Mw 7.5 (m_b 7.2), with a mean magnitude of Mw 6.9 (m_b 6.9). Approximately 10% of observed magnitudes are between Mw 6.1 and 6.5 (m_b 6.3 and 6.6), 30% are between Mw 6.5 and 6.8 (m_b 6.6 and 6.8), 40% are between Mw 6.8 and 7.1 (m_b 6.8 and 7.0), and 20% are between Mw 7.1 and 7.5 (m_b 7.0 and 7.2). The model distribution uses the

midpoints of these magnitude ranges as the magnitude and the respective percentage as the weighting. This procedure results in a model magnitude distribution of Mw 6.3 (m_b 6.5), 6.65 (m_b 6.7), 6.95 (m_b 6.88), and 7.3 (m_b 7.1) with weights of 0.1, 0.3, 0.4, and 0.2, respectively (Table 2.5.2-217).

The observed characteristic return periods for RGR faults are simply the reciprocal of the recurrence rates shown in Table 2.5.2-214 and vary between 4000 years and 188,000 years, with a mean return period of 36,000 years. Approximately 40% of the observed return periods are between 4000 and 188,000 years, 40% are between 25,000 and 50,000 years, and 20% are between 119,000 and 188,000 years. The model return period distribution is based on using the midpoints of these return period ranges as the return period and the respective percentage as the weighting. This procedure results in model return period distributions of 14,500 years, 37,500 years, and 119,000 years with weights of 0.4, 0.4, and 0.2, respectively. The model distribution does not contain the minimum return period observed in the data of approximately 4000 years. This exclusion was intentional because the 4000-year return period represents only 2.5% of the data, and including these shorter return periods with an appropriately low weighting would have little effect on seismic hazard at CPNPP Units 3 and 4.

A summary of the seismic source characterization for the RGR point source is shown in Table 2.5.2-217. Given the large distance between the RGR faults and CPNPP Units 3 and 4, details of the geometry do not have a significant impact on ground motions at the site.

2.5.2.4.2.3.4 Cheraw Fault

The Cheraw fault, located in southeastern Colorado over 500 mi from CPNPP Units 3 and 4 (Figure 2.5.2-213), has been reported as having three surface-rupturing earthquakes within the past 25,000 years (see discussion in Subsection 2.5.1.1.4.3.7.2) (References 2.5-323 and 2.5-326). While the potential for Quaternary activity on the Meers fault was identified prior to the EPRI-SOG study (Reference 2.5-322), the identification of the Cheraw fault as a capable fault did not occur until after the EPRI-SOG study (References 2.5-323 and 2.5-326). As such, none of the EPRI-SOG ESTs identified the Cheraw fault as a tectonic feature or seismogenic source. Despite the considerable distance between CPNPP Units 3 and 4 and the fault, the Cheraw fault is included in a screening study for CPNPP Units 3 and 4 because it was not included in the EPRI-SOG model and because the low level of seismicity surrounding CPNPP Units 3 and 4 may allow for earthquakes on the Cheraw fault to contribute to hazard at CPNPP Units 3 and 4.

The seismic source characterization of the Cheraw fault used here is a conservative simplification of the Cheraw fault in the 2002 USGS National Seismic Hazard Maps (Reference 2.5-321). As discussed in Subsection 2.5.2.2.2.6, the USGS seismic source characterizations do not undergo a formal expert elicitation process and do not explicitly attempt to represent the full uncertainty of source characterizations. However, the source models are developed from the range of

published literature, and source characterizations are discussed in regional working groups. As such, the USGS source model for the Cheraw fault is deemed a good representation of the potential seismic hazard contributed by the Cheraw fault.

The USGS characterization of the Cheraw fault includes alternative models of fault recurrence behavior including truncated Gutenberg-Richter and characteristic earthquake relationships. For CPNPP Units 3 and 4, the USGS characterization is simplified by assuming only a characteristic earthquake recurrence relationship parameterized by the characteristic recurrence rate and characteristic earthquake magnitude taken from the USGS parameterization of the fault (Reference 2.5-321). Uncertainty is added to the characteristic magnitude (Mw 7.0 or m_b 6.9) using a magnitude distribution of ± 0.2 Mw units about the USGS-reported magnitude with weightings of 0.2, 0.6, and 0.2 for the lowest to highest magnitude. The characteristic recurrence rate of 1.148 $x10^{-4}$ earthquakes per year (return period of 8711 years) is taken directly from the USGS model (Reference 2.5-321). The surface trace of the Cheraw fault is simplified from the USGS description of the fault by using only the endpoints of the fault trace, and the fault dip is assumed to be 90° instead of the 60° to the northwest used in the 2002 USGS hazard maps (Reference 2.5-321). These simplifications will not affect the hazard at CPNPP Units 3 and 4, given the large distance between the fault and site. Finally, a probability of activity of 1.0 is used in the characterization for CPNPP Units 3 and 4 instead of the 0.5 used in the USGS model (Reference 2.5-321) because there is conclusive evidence of Holocene fault rupture.

A summary of the seismic source characterization for the Cheraw fault is shown in Table 2.5.2-218. This characterization is implemented into the CPNPP Units 3 and 4 PSHA model as a line source extending to 15 km depth.

2.5.2.4.3 New Ground Motion Models

Ground motion models for the central and eastern US (CEUS) have evolved since the EPRI-SOG (Reference 2.5-370) study. An EPRI project was conducted to summarize knowledge about CEUS ground motions, and results were published in an EPRI report (Reference 2.5-401). These updated equations estimate median spectral acceleration and its uncertainty as a function of earthquake magnitude and distance. Epistemic uncertainty is modeled using multiple ground motion equations with weights, and multiple estimate of aleatory uncertainty, also with weights. Different sets of equations are recommended for seismic sources that represent rifted vs. non-rifted regions of the earth’s crust. Separate equations are recommended for attenuation in the stable continental region of the CEUS and for the Gulf Coast region. Equations are available for spectral frequencies at hard rock sites of 100 Hz (which is equivalent to peak ground acceleration, PGA), 25 Hz, 10 Hz, 5 Hz, 2.5 Hz, 1 Hz, and 0.5 Hz.

The aleatory uncertainties published in the EPRI model (Reference 2.5-401) were re-examined by Abrahamson and Bommer (Reference 2.5-403), because it was

thought that the aleatory uncertainties in the 2004 EPRI report (Reference 2.5-401) were too large, resulting in over-estimates of seismic hazard. The EPRI (Reference 2.5-403) study recommends a revised set of aleatory uncertainties and weights that can be used to replace the original EPRI (Reference 2.5-401) aleatory uncertainties.

To correctly model the damageability of small magnitude earthquakes to engineered facilities, the Cumulative Absolute Velocity (CAV) model of Hardy, et al. (Reference 2.5-404) was used. The CAV model in effect filters out the fraction of small magnitude earthquakes that will not cause damage and includes in the hazard calculations only those ground motions with CAV values greater than 0.16 g-sec. The filter that is used is based on empirical ground motion records and depends on ground motion amplitude, earthquake magnitude, duration of motion (which in turn depends on earthquake magnitude), and shear-wave velocity in the top 30 m at the site. The ground motions for frequencies other than 100 Hz are assumed to be correlated with the ground motions at 100 Hz, so that the filtering is consistent from frequency to frequency.

In summary the ground motion model used in the seismic hazard calculations for CPNPP Units 3 and 4 consisted of the median equations from EPRI (Reference 2.5-401) combined with the updated aleatory uncertainties of the EPRI study (Reference 2.5-404). The CAV filter (Reference 2.5-404) was applied to account for the damageability of small-magnitude earthquake ground motions.

2.5.2.4.4 Updated Probabilistic Seismic Hazard Analysis and Deaggregation

The seismic hazard at the CPNPP Units 3 and 4 site was investigated with the changes described in Subsections 2.5.2.4.2 through 2.5.2.4.3 to seismic sources, seismicity parameters, maximum magnitudes, and ground motion equations. The initial investigation was made for hard rock conditions, followed by the incorporation of site-specific conditions at the CPNPP Units 3 and 4 site.

A probabilistic seismic hazard analysis (PSHA) consists of calculating annual frequencies of exceeding various ground motion amplitudes for all possible earthquakes that are hypothesized in a region. The seismic sources specify the rates of occurrence of earthquakes as a function of magnitude and location, and the ground motion model estimates the distribution of ground motions at the site for each event. Multiple weighted hypotheses on seismic sources, earthquake rates of occurrence, and ground motions (characterized by the median ground motion amplitude and its uncertainty) result in multiple weighted seismic hazard curves. The calculation is made separately for each of the six EPRI teams (as described in Subsection 2.5.2.4.1), and the seismic hazard distributions for the teams are combined, weighting each team equally. This combination gives the overall mean and distribution of rock seismic hazard at the site. The effects of local site conditions on seismic ground motions are taken into account as described below.

A preliminary calculation of rock seismic hazard was made with the EPRI-SOG sources plus the Meers fault and New Madrid faults, using the EPRI ground motion equations (Reference 2.5-401) with the EPRI aleatory uncertainties (Reference 2.5-403) and no CAV filter. Sensitivity studies indicated that of the faults identified in Subsection 2.5.2.4.2.3, only the New Madrid and Meers faults contributed significantly to the hazard. The other faults discussed in Subsection 2.5.2.4.2.3 (the Rio Grande Rift faults and the Cheraw fault) did not contribute 1% of the total hazard for 10 Hz and 1 Hz spectral acceleration. The preliminary calculation of hazard was done for the purpose of deaggregating the hazard. The CAV filter was not used for this analysis because the CAV filter depends on site amplitude and shear-wave velocity in the top 30 meters from the surface. The reason was that incoming seismic waves that might produce low-amplitude rock motions and be removed by the CAV filter, might also be amplified by local soil conditions, producing higher amplitudes on soil that would not be removed by the CAV filter.

Figures 2.5.2-215 through 2.5.2-221 show total rock hazard as the mean, 15th, 50th, and 85th fractile curves for the EPRI-SOG sources plus the Meers fault and New Madrid faults, using the EPRI ground motion equations (Reference 2.5-401) with the EPRI aleatory uncertainties (Reference 2.5-403) and no CAV filter. The total mean and fractile rock hazard curves are shown for all sources. In addition, the mean hazard from the New Madrid faults is shown (this is included in the total curves). The Meers fault and New Madrid faults dominate the hazard for frequencies of 5 Hz and lower, and contribute a significant part of the hazard for 10 Hz amplitudes and higher. One of the characteristics of the hazard curves at low spectral frequencies (2.5 Hz and lower) is that the mean rock hazard curves exceeded the 85th fractile at high ground motion amplitudes. This exceedance occurs because the New Madrid seismic source dominates the hazard, and is caused by a few EPRI ground motion equations (Reference 2.5-401) indicating relatively high hazards for the large distance between the New Madrid seismic source and the CPNPP Units 3 and 4 site.

Figure 2.5.2-222 shows the mean and median 10^{-4} and 10^{-5} UHRS for hard rock conditions, based on the seven ground motion frequencies for which ground motion estimates are available. Numerical values for the mean UHRS are shown in Table 2.5.2-219.

The seismic hazard was deaggregated following the guidelines of Regulatory Guide 1.208 (USNRC, 2007). Specifically, the mean contributions to seismic hazard for 5 Hz and 10 Hz hazards were deaggregated by magnitude and distance for the mean 10^{-4} ground motions at 5 Hz and 10 Hz, and these deaggregations were combined. Figure 2.5.2-223 shows this combined deaggregation. Similar deaggregations of the mean hazard were performed for 1 and 2.5 Hz spectral accelerations (Figure 2.5.2-224). Deaggregations of the mean hazard for 10^{-5} and 10^{-6} ground motions are shown in Figures 2.5.2-225 through 2.5.2-228. Deaggregation of the mean seismic hazard is recommended in Regulatory Guide 1.206 (USNRC, 2007). The contribution of the New Madrid

source to seismic hazard is plotted in the deaggregation figures in the last distance interval, which represents 400+ km; the New Madrid source is actually about 870 km from the Comanche Peak site.

Figures 2.5.2-223 through 2.5.2-228 include the contribution to hazard by e, which is the number of logarithmic standard deviations that the applicable ground motion (10^{-4}, 10^{-5}, or 10^{-6}) is above the logarithmic mean. These figures indicate that the largest contribution to hazard for 10^{-4} and 10^{-5} ground motions comes from values between 0 and 2 standard deviations above the mean, which is a common result.

The deaggregation plots in Figures 2.5.2-223 through 2.5.2-228 for 10^{-4} and 10^{-5} ground motions indicate that the Meers fault and New Madrid faults have major contributions to seismic hazard at the Comanche Peak site. For 10^{-4} annual frequency of exceedance, these sources are the largest contributors to seismic hazard for both 5 and 10 Hz (Figure 2.5.2-223) and 1 and 2.5 Hz (Figure 2.5.2-224). For an annual frequency of 10^{-5}, the Meers fault and New Madrid faults are also dominant contributors to seismic hazard, even for high frequencies (Figures 2.5.2-225 and 2.5.2-226). For an annual frequency of 10^{-6}, most of the hazard at high frequencies comes from local sources (Figure 2.5.2-227), while low frequencies still have a dominant contributions from the New Madrid faults (Figure 2.5.2-228). All of these observations are confirmed qualitatively in Figures 2.5.2-217 through 2.5.2-220, which compare the hazard from the Meers fault and the New Madrid faults to the hazard from all sources for 10, 5, 2.5, and 1 Hz.

Table 2.5.2-220 summarizes the mean magnitude and distance resulting from these deaggregations, for all contributions to hazard and for contributions with distances exceeding 100 km. For the 1 and 2.5 Hz results, contributions from events with R>100 km exceed 5% of the total hazard. As a result, following the guidance of RG1.208, the controlling earthquake for low-frequency ground motions was selected from the R>100 km calculation, and the controlling earthquake for high-frequency ground motions was selected from the overall calculation. The values of Mw and R selected in this way are shown in shaded cells in Table 2.5.2-220.

Tables 2.5.2-221 through 2.5.2-226 document the deaggregation of seismic hazard for the following deaggregations: 10^{-4} high frequencies, 10^{-4} low frequencies, 10^{-5} high frequencies, 10^{-5} low frequencies, 10^{-6} high frequencies, and 10^{-6} low frequencies.

Smooth rock UHRS were developed from the UHRS amplitudes in Table 2.5.2-219, using controlling earthquake Mw and R values shown in Table 2.5.2-220 and using the hard rock spectral shapes for CEUS earthquake ground motions recommended in NUREG/CR-6728. Separate spectral shapes were developed for high frequencies (HF) and low frequencies (LF). In order to accurately reflect the UHRS values calculated by the PSHA as shown in Table 2.5.2-220, the HF

spectral shape was anchored to the UHRS values from Table 2.5.2-220 at 100 Hz, 25 Hz, 10 Hz, and 5 Hz. In between these frequencies, the spectrum was calculated using shapes anchored to the next higher and lower frequency and weighting those shapes. The weighting was based on the inverse logarithmic difference between the intermediate frequency and the next higher or lower frequency. This technique provided a smooth, realistic spectral shape at these intermediate frequencies. Below 5 Hz, the HF shape was extrapolated from 5 Hz.

For the LF spectral shape a similar procedure was used except that the LF spectral shape was anchored to the UHRS values at all seven ground motion frequencies for which hazard calculations were made (100 Hz, 25 Hz, 10 Hz, 5 Hz, 2.5 Hz, 1 Hz, and 0.5 Hz). Anchoring the LF spectral shape to all frequencies was necessary because otherwise the LF spectral shape exceeded the HF spectral shape at high frequencies. The use of a LF shape with amplitudes higher than the HF UHRS amplitudes would not be appropriate because this would overdrive the soil column. Anchoring the LF spectrum to the UHRS amplitudes at all frequencies ensures that appropriate ground motions are represented. The lack of fit of the LF spectral shape to the HF UHRS amplitudes results from distant, large earthquakes that contribute to seismic hazard at this site, with ground motion ε values greater than unity. In these cases, the spectral shapes of NUREG/CR-6728 are not appropriate and the LF spectrum needs to be anchored to the HF UHRS amplitudes.

Figures 2.5.2-229 through 2.5.2-231 show the smooth horizontal HF and LF UHRS calculated in this way for 10^{-4}, 10^{-5}, and 10^{-6} annual frequencies of exceedance, respectively. As mentioned previously, these spectra accurately reflect the UHRS amplitudes in Table 2.5.2-219 that were calculated for the seven spectral frequencies at which PSHA calculations were done. Because the HF and LF spectra were scaled to the same high-frequency amplitudes, they are very similar at high frequencies and differ only for frequencies below 5 Hz. As a result of these similarities, a broad-banded spectrum was used as input to site response calculations, using the envelope of the HF and LF spectra shown in Figures 2.5.2-229 through 2.5.2-231.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.5 with the following.

The subsurface conditions necessary to predict and model the seismic wave transmission characteristics for CPNPP Units 3 and 4 were determined from both site-specific and regional data. These data included both stratigraphic and representative shear and compressional wave measurements that were used to develop the site profile and are summarized in Table 2.5.2-227. A detailed discussion of the data and methodology for developing the stratigraphy and corresponding dynamic properties used to define the dynamic profile for the site is provided in Subsection 2.5.4.4.2.2.

The profile is divided into the shallow profile (surface to about 500 ft) and the deep profile (about 500 ft to “basement”). The shallow profile represents depth to which extensive characterization has been performed. The lateral and vertical control on the subsurface strata (layering) was defined primarily on lithology and material properties. The velocity measurements in the shallow profile have been developed from 15 suspension logs from borings drilled as part of the foundation exploration described in Subsection 2.5.4.4.2.1.

The foundation basemats of all seismic Catergory Istructures will be founded on a limestone unit (denoted as Layer C in Subsection 2.5.4). Excavation to Layer C will remove the shallower units (layers A, B1, and B2) and, where the top of Layer C is below the bottom of the elevation, fill concrete will be placed to achieve the bottom of basemat elevation. The average thickness of Layer C is greater than 60 ft and dips less than 1°. The average shear wave velocity of Layer C is greater than 5800 ft/sec, as determined from the 15 suspension log borings. Profiles for development of the GMRS and FIRS are detailed in Subsection 2.5.2.6 and provide the criteria for exclusion or inclusion of specific layers including fill concrete and compacted fill.

The deep profile was characterized from regional wells and maps. Strata that define the deep profile are based primarily on lithology and stratigraphic surfaces projected to the CPNPP site to estimate the elevation. Velocity data for the deep profile was limited to only a few wells and consisted primarily of compressional wave velocities except where shear wave velocity data was available from a single well as discussed in the following section on uncertainties. Basement was defined as the depth at which a shear wave velocity of 9200 ft/sec and greater was achieved. Basement was therefore defined as the top of the Ellenburger limestone located at a depth of about 5300 ft at the site. The Ellenburger is a regionally extensive unit with an estimated shear wave velocity of nearly 11,000 ft/sec.

2.5.2.5.1 Aleatory and Epistemic Uncertainty

The shallow profile has been extensively characterized from over 150 geotechnical borings and geologic mapping of the area. The profile has been stratified based on vertical changes in lithology that can be mapped laterally from boring to boring. Standard deviations for the top of each shallow profile layer are less than 2 ft for the upper 200 ft of the profile. The standard deviation for the layers defining the shallow profile from about 200 ft to about 500 ft range from about 1 to 5 ft. Velocity data for the shallow profile acquired from 15 suspension borings demonstrated a strong correlation between the layering and places where simulated down-hole travel time gradient “breaks” occurred.

The deep profile was developed from regional wells and results in a higher uncertainty in both the layering (stratigraphy) and velocity measurements. Shear wave velocity measurements were available from a single well located about 6 mi from the site and were limited to the Barnett Shale (a shale unit at a depth of about 5000 ft) for a total depth interval of about 4000 ft (about 5000 ft depth to

about 9000 ft depth). This data was used to develop a linear extrapolation to estimate shear wave velocity from available pressure wave velocities from other wells to complete the deep profile. Thus, the epistemic uncertainty for the deep profile is much greater than for the shallow profile. See Subsection 2.5.4.4.2.2 for detailed discussion.

The deep profile lacks a statistical basis for estimating a robust standard deviation for all layer velocities. The coefficient of variation (CoV=standard deviation/mean) calculated as 31% for the Atoka formation demonstrated the highest CoV for all deep profile layers. Therefore, the variability in velocity was calculated at 31% for all deep profile layers. The velocity range for the shallow profile was defined as 25% of the mean velocity of each layer. Subsection 2.5.4.4.2.2 provides a detailed discussion of the data and methodology for development of the dynamic profile.

Table 2.5.2-227 summarizes the layer properties including depth, thickness, velocities and assigned variabilities based on the aleatory and epistemic uncertainties discussed.

2.5.2.5.2 Description of Site Response Analysis

The site response analysis was conducted in three steps that are common to analyses of this type. First, the site geology and geotechnical properties were reviewed and used to generate multiple synthetic profiles of site characteristics. Second, sets of rock spectra were selected to represent rock ground motions corresponding to mean annual exceedence frequencies of 10^{-4}, 10^{-5}, and 10^{-6}. Finally, site response was calculated using an equivalent-linear technique, using the multiple synthetic profile and the sets of rock spectra representing input motions. These three steps are described in detail in the following sections.

2.5.2.5.2.1 Generation of Synthetic Profiles

To account for the epistemic and aleatory uncertainties in the site's dynamic properties, 60 synthetic profiles were generated using the stochastic model developed by Toro (Reference 2.5-432), with some modifications to account for the conditions at the Comanche Peak site. These synthetic profiles represent the site column from the top of the bedrock to the elevations where the GMRS and the various FIRS are defined (see Subsection 2.5.2.6). Bedrock is defined as having a shear-wave velocity of 9,200 fps, in order to achieve consistency with the 2004 EPRI attenuation equations used for the rock hazard calculations (Reference 2.5-401). For each site column, this stochastic model uses as inputs the following quantities: (1) the median shear-wave velocity profile, which is equal to the base-case profile given in Table 2.5.2-227; (2) the standard deviation of ln(Vs) (the natural logarithm of the shear-wave velocity) as a function of depth, which is calculated from the values in Table 2.5.2-227; (3) the correlation coefficient between ln(Vs) in adjacent layers, which is taken from generic results for rock in Toro (Reference 2.5-432). Layer thickness was not randomized because the site's stratigraphy is very uniform.

The correlation coefficient between ln(Vs) in adjacent layers is estimated using the inter-layer correlation model from Toro (Reference 2.5-432) for USGS category A. In the log-normal randomization model used to calculate the synthetic Vs for each layer, it is possible for the synthetic Vs in the deeper formations to be greater than 9,200 fps. When this happens for a certain synthetic profile, the randomization scheme sets that Vs to 9,200 fps and defines the corresponding depth to be the depth to bedrock for that synthetic profile.

Figure 2.5.2-240 illustrates the Vs value for the first 10 synthetic profiles for the GMRS/FIRS1 site column. Figure 2.5.2-241 compares the median of these 60 Vs profiles to the Vs ±1 sigma Variability values given in Table 2.5.2-227, indicating excellent agreement. The difference in the mean +sigma values below 800 m is a consequence of imposing the 9200 fps upper bound dictated by the bedrock Vs(see above). Figures 2.5-242 and 2.5-243 show analogous results for top portion the FIRS4 site column.

The best-estimate values for the damping ratio and for the stiffness degradation (G/Gmax) are given in Table 2.5.2-227. Except for the fill at the top of the FIRS4 soil column, materials are assumed to behave linearly (strain-independent), with constant damping and G/Gmax=1. The uncertainty in damping is specified as 35%, (following the generic values in EPRI, Reference 2.5-387) and the uncertainty in G/Gmax for fill is specified as 15% at $3x10^{-3}$% strain (following the generic values given by Costantino, Reference 2.5-433). The correlation coefficient between ln(G/Gmax) and ln(damping) in the fill is specified as -0.75. This implies that in synthetic profiles where the fill has higher than average G/Gmax, the fill tends to have lower than average damping. The degradation and damping properties are treated as fully correlated among layers in the same geological unit, but independent between different units. Figure 2.5.2-244 shows the damping ratios for the Strawn formation in the 60 synthetic profiles corresponding to FIRS1. Similarly, Figure 2.5.2-245 shows the G/Gmax and damping ratios for the 60 synthetic profiles corresponding to FIRS4. A sensitivity study that evaluates the effect of using strain-dependent shear-modulus degradation (G/Gmax) and damping ratio, instead of using constant shear-modulus degradation (G/Gmax =1) and constant damping ratio. Results from this study indicate that the spectra at the top of the profile obtained with the constant material properties are slightly higher than those obtained with strain-dependent properties. The profile with constant material properties was used to develop all FIRS (GMRS/FIRS1, FIRS2, FIRS2, FIRS4, and FIRS4_CoV50), as presented in Subsection 2.5.2.6, and to develop the inputs for the SSI analysis in Subsection 3.7.2.

Each set of 60 synthetic profiles, consisting of Vs and unit weight vs. depth, depth to bedrock, stiffness, and damping curves, is used to calculate and quantify site response and its uncertainty, as described below.

2.5.2.5.2.2 Selection of Rock Input Motions

Rock input motions were selected for input to the site response calculations using the seismic hazard results presented in Subsection 2.5.2. Uniform hazard response spectra (UHRS) for rock conditions corresponding to mean annual exceedence frequencies of 10^{-4}, 10^{-5}, and 10^{-6} were used. The base spectrum for each mean annual exceedence frequency was a broad-banded (BB) spectrum, because deaggregation and fitting of high-and low-frequency (HF and LF) spectra indicated the same high-frequency amplitudes. These spectra are plotted in Figures 2.5.2-229 through 2.5.2-231 and are given in tabular form in Table 2.5.2-219. The development of these spectra is documented in Subsection 2.5.2.4.4. The effect of choosing a broad-banded spectrum was investigated by also computing response to the 10^{-4} HF spectrum, and comparing that response to the 10^{-4} BB spectrum, as described in the next subsection.

2.5.2.5.2.3 Site Response Calculations

The site response calculations for Comanche Peak were performed using the Random Vibration Theory (RVT) approach. In many respects, the inputs and assumptions are the same for an RVT analysis and for a time-history based analysis (e.g., an analysis with the program SHAKE, Reference 2.5-434). Both the RVT and time-history (SHAKE, Reference 2.5-434) procedures use a horizontally-layered half-space representation of the site and use an equivalent-linear representation of dynamic response to vertically propagating shear waves. Starting from the same inputs (in the form of response spectra), both procedures will lead to similar estimates of site response (see, for example, Rathje and Ozbey, Reference 2.5-435). The main advantage of the RVT approach is that it does not require the spectral matching of multiple time histories to a given rock response spectrum. Instead, the RVT approach uses a probabilistic representation of the ensemble of all input motions corresponding to that given response spectrum and then calculates the response spectrum of the ensemble of dynamic responses.

Site-response calculalions were performed for the three broad-banded (BB) bedrock motions, and for the 10^{-4} HF motion,as described in the previous section.

In addition to the rock response spectra, the RVT site-response calculations require the following inputs: (1) the strong-motion duration associated with each rock spectrum; and (2) the equivalent-strain ratio to use in the eqivalent-linear calculations (this input is required for both the time-history and RVT approaches) and depends on magnitude. The duration is calculated from the de-aggregation results in Subsection 2.5.2.4.4 (Table 2.5.2-220), using standard seismological relations between magnitude, seismic moment, corner frequency, and duration (see, for example, Rathje and Ozboy, Reference 2.5-435) and using stress-drop and crustal Vs values typical of the eastern United States. The effective strain ratio is calculated using the expression (M-1)/10 (Reference 2.5-434). Values smaller than 0.5 or greater than 0.65 were brought into the 0.5-0.65 range, which

is the range recommended by Kramer (Reference 2.5-436). The calculated values of duration and effective strain ratio are given in Table 2.5.2-230.

For each site column and each rock-motion input, separate site response calculations were performed for the corresponding 60 synthetic profiles. These results for each combination of input motion and site column were then used to calculate the logarithmic mean and standard deviation of the amplification factor. Results for the various site columns, and for the 10^{-4}, 10^{-5}, and 10^{-6} BB inputs, are given in Figures 2.5.2-233 and 2.5.2-235 through 2.5.2-238. Tabular results are provided in Tables 2.5.2-231 through 2.5.2-235.

Figure 2.5.2-253 and Figure 2.5.2-254 present the peak strain in the upper 500 ft of the GMRS/FIRS1 soil column for the $1x10^{-4}$ and $1x10^{-5}$ broad band (BB) spectra, respectively. The maximum value of the logarithmic-mean strain (over the 60 synthetic profiles) in the entire GMRS/FIRS1 profile for the $1x10^{-4}$ spectrum is approximately 0.0035% ad occurs at a depth of approximately 390 ft in the profile. The maximum value of the logarithmic-mean strain in the entire GMRS/FIRS1 profile for the $1x10^{-5}$ spectrum is approximately 0.0075% and also occurs at a depth of approximately 390 ft in the profile.

Figure 2.5.2-255 and Figure 2.5.2-256 present the peak strain in the upper 50 ft of the FIRS4 soil column for the $1x10^{-4}$ broad-band (BB) spectra, respectively. As described in FSAR Subsection 2.5.2.6, the FIRS4 site profile consists of compacted fill overlying the stiff limestone that is the outcrop of the GMRS/FIRS1 profile. As such, the peak strains within most of the FIRS4 profile are similar to the peak strains within the GMRS/FIRS1 profile with the exception of peak strains within the fill (i.e., the upper 40 ft).

Therefore, Figure 2.5.2-255 and Figure 2.5.2-256 only show the peak strains within the upper 50 ft of the FIRS4 profile. The maximum value of the logarithmic-mean strain in the FIRS4 profile for the $1x10^{-4}$ spectrum is approximately 0.006% and occurs at depths of approximately 17 and 37 ft in the profile. The maximum value of the logarithmic-mean strain in the FIRS4 profile for the $1x10^{-5}$ spectrum is approximately 0.016% and also occurs at depths of approximately 17 and 37 ft in the profile.

The logarithmic mean value of the peak strain in the fill is approximately 0.03% for the 10^{-6} inputs.

In addition, Figure 2.5.2-246 compares the median amplification factors obtained for GMRS/FIRS1 site column using the 10^{-4} HF and BB rock inputs. Although Figure 2.5.2-246 shows that the BB spectrum gives larger amplification factors for frequencies above 3 Hz, the effect of this difference on the 10^{-4} site hazard will be negligible because most of the 10^{-4} hazard at all frequencies comes from distant events (see Figures 2.5.2-223 and 2.5.2-224). These distant events will generate

a BB rock spectrum. The effect of a difference in amplification factors at 10^{-5} would be somewhat larger (and would result in lower mean site spectra) because roughly 40% of the 10^{-5} hazard comes from local, small-magnitude events (see Figures 2.5.2-225 and 2.5.2-226). As a result, use of the BB amplification factors for all magnitude-distance combinations in the soil-hazard calculations (Subsection 2.5.2.6.1.1) yields slightly conservantive hazard results at 10^{-5}, resulting in slightly conservative estimates of the design spectrum.

2.5.2.6 Ground Motion and Site Response Analysis

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.6 with the following.

Four FIRS have been identified for the CPNPP Units 3 and 4 and are calculated for both the Safe Shutdown Earthquake (SSE) and Operating Basis Earthquake (OBE) where OBE=(1/3)SSE. The SSE is the envelope of the GMRS and the minimum earthquake requirements of 10 CFR 50 Appendix S, based on the shape of the Certified Site Design Response Spectra (CSDRS) scaled down to a PGA of 0.1 g. The CSDRS is itself a modified RG 1.60 shape formed by shifting the control points at 9 Hz and 33 Hz to 12 Hz and 50 Hz, respectively.

2.5.2.6.1 Ground Motion Response Spectrum (GMRS)

All category 1 structures as well as the Turbine Building will be founded directly on a stiff limestone (Layer C) at elevation 782 ft. Thus the GMRS/FIRS1 (referred to hereafter as GMRS) represents the top of stiff limestone (Layer C) at, or slightly below, foundation basemat elevation for the following safety-related and seismic Category II structures:

- Reactor Building
- Ultimate Heat Sink
- Turbine Building
- Auxiliary Building
- Essential Service Water Pipe Tunnel
- Power Source Fuel Storage Vaults
- East and West Power Source Buildings

In some cases, slight amounts of over-excavation will be required below the planned foundation subgrade elevations to reach the stiff limestone (Layer C). In these cases, a relatively thin layer of fill concrete will be placed on the cleaned limestone sub-excavation and extended to the foundation subgrade elevation.

The thickness of the fill concrete will potentially range from about 0 ft to less than 2 ft.

Ground motion response spectra (GMRS) were calculated for horizontal and vertical motion by the methods discussed below.

2.5.2.6.1.1 Horizontal GMRS Spectrum

A seismic hazard calculation was made using the site amplification factors for the GMRS elevation, which is elevation 782 ft (top of Layer C). Figure 2.5.2-233 shows the median amplification factor (AF) and logarithmic standard deviation of AF for this elevation, using broad-banded input motions (the envelope of the spectra in Figures 2.5.2-229 through 2.5.2-231). This calculation was made at the seven spectral frequencies at which ground motion equations were available from the 2004 EPRI study (Reference 2.5-401) (100 Hz, 25 Hz, 5 Hz, 2.5 Hz, 1 Hz, and 0.5 Hz).

The seismic hazard for horizontal motion was calculated by integrating the horizontal amplification factors shown in Figure 2.5.2-233 with the rock hazard and applying the CAV filter. This corresponds to Approach 3 in NUREG/CR-6769.

The horizontal GMRS was developed from the horizontal UHRS using the approach described in ASCE/SEI Standard 43-05 (Reference 2.5-371) and Regulatory Guide 1.208. The ASCE/SEI Standard 43-05 (Reference 2.5-371) approach defines the GMRS using the site-specific UHRS, which is defined for Seismic Design Category SDC-5 at a mean 10^{-4} annual frequency of exceedance. The procedure for computing the GMRS is as follows.

For each spectral frequency at which the UHRS is defined, a slope factor A_R is determined from:

$$A_R = SA(10^{-5})/SA(10^{-4}) \quad \text{(Equation 5)}$$

where $SA(10^{-4})$ is the spectral acceleration SA at a mean UHRS exceedance frequency of 10^{-4}/yr (and similarly for $SA(10^{-5})$). A design factor (DF) is defined based on A_R, which reflects the slope of the mean hazard curve between 10^{-4} and 10^{-5} mean annual frequencies of exceedance. The DF at each spectral frequency is given by:

$$DF = 0.6(A_R)^{0.80} \quad \text{(Equation 6)}$$

and

$$GMRS = \max[SA(10^{-4}) \times \max(1, DF), 0.45 \times SA(10^{-5})] \quad \text{(Equation 7)}$$

The derivation of DF is described in detail in the Commentary to ASCE/SEI Standard 43-05 and in Regulatory Guide 1.208.

For the CPNPP Units 3 and 4 site, the horizontal hazard curves for GMRS elevation roll over at low amplitudes to an annual frequency of exceedance less than 10^{-4}. This means that the frequency of damaging ground motions is less than 10^{-4}. Under these conditions, the GMRS is calculated from Equation 7 above as 0.45 x SA(10^{-5}). Table 2.5.2-228 shows the 10^{-5} ground motion at the seven spectral frequencies for which ground motion equations are available, and shows the GMRS calculated as 0.45 x SA(10^{-5}).

Figure 2.5.2-234 shows the horizontal GMRS spectrum taken from Table 2.5.2-228, plotted with the horizontal DCD spectrum. This figure shows that the GMRS at the seven spectral frequencies at which ground motion equations were available from the 2004 EPRI study (Reference 2.5-401) is enveloped by the DCD spectrum.

The horizontal 10^{-5} and GMRS spectra were calculated at 39 frequencies between 0.1 Hz and 100 Hz for the GMRS elevation. This spectral frequency range encompasses all the energy of the rock ground motions for earthquakes in the Central and Eastern United States and meets the requirements in Subsection 3.4 “Hazard Assessment” in item C “Regulatory Position” of Regulatory Guide 1.208. The natural frequency of the GMRS soil column is 0.29 Hz. Because of the very flat appearance of the spectra at the seven spectral frequencies at which hazard calculations were made, log-log interpolation between available hazard values was used, with the exception of the following frequency ranges.

<u>1 Hz to 5Hz:</u> Within this frequeney range, a peak in site spectra occurs at 2.5 Hz, reflecting a site amplification at about 2 Hz. To reflect this amplification, the 10^{-5} spectral amplitude at 2.5 Hz was broadened using rock spectral shapes from NUREG/CR-6728 and using the broad-banded values of M=7.5 and R=650 km for 10^{-5} (on which the site amplification calculations were based). This is an acceptable approximation given that the rock spectrum is decreasing between 2.5 and 1 Hz.

<u>0.5 Hz to 0.1 Hz:</u> Below 0.5 Hz, the site-specific spectral shape determined during site amplification calculations was used to extrapolate to 0.1 Hz. This spectral shape was determined from the 10^{-5} surface spectrum at the GMRS elevation, using the 10^{-5} rock input motion. This spectral shape between 0.5 Hz and 0.1 Hz was used to extrapolate the GMRS from 0.5 Hz to 0.1 Hz. The GMRS shape at long periods is thereby consistent with the site-specific amplification calculation for the GMRS elevation.

The horizontal GMRS and 10^{-5} spectra are plotted in Figure 2.5.2-247, and the numerical values of the spectra are shown in Table 2.5.2-236.

The smooth horizontal GMRS spectrum is plotted in Figure 2.5.2-257 along with the respective DCD spectrum. This figure shows that the GMRS spectrum is enveloped by the DCD.

2.5.2.6.1.2 Vertical GMRS Spectrum

Vertical motions at the CPNPP Units 3 and 4 site are addressed by reviewing results in NUREG/CR-6728 for V/H ratios at deep soil sites, for both the western US (WUS) and the CEUS. Example results presented in the NUREG/CR-6728 indicate that for earthquakes >40 km from a deep soil site, V/H ratios are expected to be less than unity for all frequencies (Figures J-31 and J-32 in Appendix J of the NUREG/CR-6728). For the 10^{-5} ground motion, expected distances from deaggregation are greater than 100 km (Table 2.5.2-220). Any exceedance of unity occurs for high frequencies (>10 Hz) for short source-to-site distances. Also, for ground motions with peak horizontal accelerations <0.2g, the recommended V/H ratios for hard rock conditions are less than unity; see Table 4-5 of the NUREG/CR-6728. The conclusion is that V/H ratios for the CPNPP Units 3 and 4 site will be less than unity for all spectral frequencies. Therefore, the vertical GMRS will be below the horizontal GMRS shown in Figure 2.5.2-234.

Figure 2.5.2-234 shows that the horizontal DCD spectrum exceeds the horizontal GMRS. The vertical DCD spectrum equals or does not exceed the horizontal DCD spectrum for frequencies above 3.5 Hz. The conclusion is that the vertical DCD spectrum will also exceed the vertical GMRS. Under this condition, the DCD minimum vertical design motion will govern the vertical response, just as the DCD minimum horizontal design motion will govern the horizontal response.

Vertical GMRS and FIRS spectra were developed using vertical-to-horizontal (V/H) ratios. NUREG/CR-6728 and RG 1.60 indicate proposed V/H ratios for design spectra for nuclear facilities, and these V/H ratios are plotted in Figure 2.5.2-252. The V/H ratios in Figure 2.5.2-252 taken from NUGREG/CR-6728 (the blue curve) are recommended for hard sites in the CEUS. The Comanche Peak site is a deep, soft-rock site with shale and limestone near the surface having shear-wave velocities of about 2600 fps, and the V/H ratios for this site condition will be similar to those for hard roick sites.

Based on these comparisons, it is concluded that the applicable V/H ratios at the Comanche Peak site will be ≤ 1.0 at all spectral frequencies between 100 Hz and 0.1 Hz. As a conservative assumption, the V /H ratio is assumed to be equal to the V/H ratio from RG 1.60. This assumption is also plotted in Figure 2.5.2-252. The vertical GMRS spectrum resulting from this assumption is presented in Table 2.5.2-236.

The smooth vertical GMRS spectrum is plotted in Figure 2.5.2-258 along with the respective DCD spectrum. This figure shows that the GMRS spectrum is enveloped by the DCD.

2.5.2.6.2 Foundation Input Response Spectrum

Site response analyses were conducted for an additional four cases (FIRS 2, FIRS 3, FIRS 4_CoV30, and FIRS 4_CoV50) to consider foundation input response spectra for specific conditions different from the GMRS elevation. These four cases are as follows:

FIRS 2 - Set at elevation 787 ft.

This FIRS represents generic site response conditions for structures resting on fill concrete layer in which the fill concrete thickness and horizontal extent away from the edge of the foundation is significant and thus modeled as a horizontally infinite layer.

- FIRS 2 analysis demonstrates that the response at the top of the fill concrete remains well below the minimum earthquake and does not apply to any specific structure.

The FIRS 2 profile consists of 5 ft of fill concrete placed over a sub-excavated stiff limestone (Layer C) surface at elevation 782 ft. Fill concrete with compressive strength ranging from 2,500 psi to 4,400 psi is considered by using a mean shear wave velocity of 6800 fps with a range of +/- 500 fps. See Table 2.5.2-227 for properties used for FIRS 2 analysis. Note that the site-specific soil-structure interaction analyses described in Subsection 3.7.2 model the fill concrete under the category 1 foundations as part of the structural model.

FIRS 3 - Set at Plant Grade elevation 822 ft.

The FIRS 3 profile considers the ground surface seismic response in areas of the site where cutting of the native soil is required to reach final Plant Grade elevation 822 ft.

- FIRS 3 analysis demonstrates that the response at Plant Grade elevation in regions of the site with native soil remains below the minimum earthquake. It does not represent the foundation subgrade elevation for any safety-related facilities identified, but could accommodate possible future shallow (at-grade) facilities.

The profile consists of stiff limestone at elevation 782 ft and overlying shale (Glen Rose Layer B1 and B2) and interbedded limestone/shale (Glen Rose Layer A) to Plant Grade elevation 822 ft. See Table 2.5.2-227 for properties used for FIRS 2 analysis.

FIRS 4 - Set at Plant Grade elevation 822 ft:

- FIRS 4 analysis demonstrates that the response of engineered compacted backfill at Plant Grade elevation remains below the minimum earthquake.

The elevations of FIRS 4 and FIRS 3 are identical, but this profile consists of sub-excavation to stiff limestone at elevation 782 ft, and backfilling to Plant Grade with cohesionless engineered fill to Plant Grade elevation 822 ft. Assumed shear wave velocity and shear modulus/damping properties for the fill are estimated based on a specified range of cohesionless fill materials, and reported properties for similar compacted fill materials. Ranges of values representing best estimates, and lower and upper bounding values, are provided in Table 2.5.2-227. Degradation curves are provided in Figure 2.5.2-232. FIRS 4 consists of two different cases (FIRS 4_CoV30 and FIRS4_CoV50) to provide a wide variability on shear wave velocities estimated for the cohesionless compacted fill.

FIRS4_CoV30: elevation 822 ft. The elevation for FIRS 4 is the same as for FIRS 3, but the profile consists of sub-excavation to stiff limestone at elevation 782 ft, and backfilling to plant grade with cohesionless engineered compacted fill.

FIRS4_CoV50: elevation 822 ft. This profile is the same as for FIRS 4 except it uses a coefficient of variation (CoV) of 50% (instead of 30%) for the Vs of the fill material.

Figures 2.5.2-235 through 2.5.2-238 show median amplification factors and logarithmic standard deviations for these four FIRS cases, for the 10^{-4}, 10^{-5}, and 10^{-6} broadband input motions.

The seismic hazard for each FIRS case was calculated by integrating the horizontal amplification factors shown in Figures 2.5.2-235 through 2.5.2-238 with the rock hazard and applying the CAV filter. This is an analogous calculation to the calculation of hazard for the GMRS elevation. For all FIRS cases the hazard curves at low amplitudes rolled over to an annual frequency of exceedance that was less than 10^{-4}. As was the case for the GMRS, the FIRS spectra were calculated using the 10^{-5} UHRS and applying the factor from Eq. 2.5.2-3; i.e., FIRS = 0.45 × SA(10^{-5}).

Figure 2.5.2-239 plots the four horizontal FIRS and compares them to the horizontal minimum DCD spectrum. The minimum DCD spectrum envelops all four FIRS, down to frequencies of 0.5 Hz. Values of the horizontal 10^{-5} UHRS and FIRS are shown in Table 2.5.2-229 for the seven spectral frequencies.

Smooth horizontal spectra for the four FIRS conditions (FIRS2, FIRS3, FIRS4, and FIRS4-CoV50) were calculated in a manner similar to the way in which the smooth GMRS was calculated, as described in Section 2.5.2.6.1.1. Note that the FIRS3 spectra have peaks at about 2.5 Hz and 10 Hz, and that the FIRS4 and FIRS4-CoV50 spectra have peaks at about 1.5 Hz and 5 Hz. These peaks were broadened in an approximate way similar to the procedure used for the GMRS.

The horizontal 10^{-5} and FIRS spectra are plotted in Figures 2.5.2-248 through 2.5.2-251. Table 2.5.2-237 shows the numerical values for the 10^{-5} and FIRS spectra.

For vertical FIRS motions, the same considerations used for the GMRS were used for the FIRS. That is, as a conservative assumption the V/H ratio for the FIRS spectra is assumed to be equal to the V/H ratio from RG 1.60.

The smooth horizontal and vertical FIRS spectra are plotted in Figures 2.5.2-257 and 2.5.2-258, respectively, along with the respective DCD spectrum. These figures show that the FIRS spectra are enveloped by the DCD.

2.5.3 Surface Faulting

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.3 with the following.

This subsection evaluates the potential for tectonic and non-tectonic surface deformation at the CPNPP Units 3 and 4 site (CPNPP Units 3 and 4). Information contained in Subsection 2.5.3 was developed in accordance with Regulatory Guide (RG) 1.165, and is intended to demonstrate compliance with 10 CFR 100.23. RG 1.165 contains guidance on characterizing seismic sources, and defines a "capable tectonic source" 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.

This subsection contains information on:

- Potential surface deformation associated with capable tectonic sources
- Potential surface deformation associated with non-tectonic processes, such as collapse structures (karst collapse), subsurface salt migration (salt domes), volcanism, and man-induced deformation (e.g., mining collapse and subsidence due to fluid withdrawal)

There are no capable faults and there is no potential for non-tectonic fault rupture within the 25-mi-radius CPNPP Units 3 and 4 site vicinity. Similarly, there is no potential for tectonic or non-tectonic deformation in the 5-mi-radius site area or the 0.6-mi-radius site. The following subsections contain the data, observations, and references to support these conclusions.

2.5.3.1 Geological, Seismological, and Geophysical Investigations

An extensive body of information regarding the potential for surface faulting is available for the CPNPP Units 3 and 4 site and is documented in several primary sources:

- Geologic mapping published by the U.S. Geological Survey (USGS), the state of Texas, and other researchers (Reference 2.5-228)
- Articles published by various researchers in refereed journals and field trip guidebooks
- Seismicity data compiled and analyzed in published journal articles, EPRI's seismic hazard methodology (Reference 2.5-369), and the updated seismicity catalog (Subsection 2.5.2)
- Previous site investigations performed for the final safety analysis for CPNPP Units 1 and 2 (Reference 2.5-201)

In addition to reviewing this existing information, the following investigations were performed to assess the potential for tectonic and non-tectonic deformation within the 5-mi-radius CPNPP Units 3 and 4 site area:

- Compilation and review of existing data and literature, with emphasis on reports published since the CPNPP Units 1 and 2 FSAR (Reference 2.5-201) and EPRI studies (Reference 2.5-369)
- Interpretation of aerial photography and remote sensing imagery
- Field and aerial reconnaissance
- Review of pre- and post-EPRI-SOG seismicity

2.5.3.1.1 Previous Site Investigations

The results of previous geology and seismology investigations at CPNPP are summarized in Subsection 2.5.3 of the CPNPP Units 1 and 2 FSAR (Reference 2.5-201) with the simple statement:

> "No evidence of surface faulting was found within five miles of the site."

In Subsection 2.5.1, it is indicated that no faults, shear zones, or anomalies were found within 5 miles of the site.

2.5.3.1.2 Regional and Local Geological Studies

The USGS has compiled information related to all known Quaternary faults, liquefaction features, and possible tectonic features in the CEUS (References 2.5-236 and 2.5-405). These compilations do not show any Quaternary tectonic faults or tectonic features within a 25-mi or 5-mi radius of the CPNPP Units 3 and 4 site. Additionally, compilation of local mapping does not show any surface faults within the 25-mi radius (Figure 2.5.1-216; Reference 2.5-406).

2.5.3.2 Geological Evidence, or Absence of Evidence, for Surface Deformation

As shown on Figure 2.5.1-216, no surface bedrock faults have been mapped within the 25-mi-radius CPNPP Units 3 and 4 site vicinity. Almost the entire site vicinity is located in the Grand Prairie physiographic province, a tectonically stable region underlain by thick continental crust (Figure 2.5.1-201). This region is characterized by low rates of seismicity, and no seismicity in the updated catalog has been found greater than 50 mi from the site (Subsection 2.5.2.1.3). The only structures at the surface within the 25-, 5-, and 0.6-mi radii are sedimentary in nature (i.e., unconformities). Bedding is nearly horizontal (dips <1°) at the 25-, 5- and 0.6-mi radii scale (Figures 2.5.1-214, 2.5.1-215, 2.5.1-216, 2.5.1-222, and 2.5.1-223; see discussion in Subsection 2.5.4.4). Two exceptions to this are seen within the 5-mi radius, where bedding locally exhibits a ~5° dip (Figures 2.5.1-

226b and 2.5.1-227). However, these features are minimal in extent, and while their cause is unresolved, no evidence indicates that these outcrop-scale, sedimentary thickness variations signal any sort of hazard for CPNPP Units 3 and 4.

Initial inspections of stereo-pair black and white, ~1:20,000 scale, aerial photographs from the 1940s yielded linear features or lineaments to be investigated in the field (Figure 2.5.3-201). These features were classified as vegetation, stream, tonal, and topographic. The mapped linear features are randomly distributed and variably oriented within the 5-mi radius of the site (Figure 2.5.3-201). The accessible features within the site area were investigated, and most of these lineaments were not identifiable in the modern landscape. Occasionally, a linear feature could be identified as a fence line or an outcrop of bedrock along a paleo-drainage. None of the lineaments investigated indicated any tectonic or geologic disruptions. A discussion of the lineament analysis is provided in the following subsection.

2.5.3.2.1 Lineament Analysis and Ground Surveys

An evaluation of the presence of geologic structures (i.e., faults and folds) expressed at the ground surface within the 5-mi radius of the CPNPP site was performed using aerial photography, satellite imagery and ground surveys (i.e, field reconnaissance). Results of this evaluation were used to focus further field reconnaissance and mapping activities. Satellite imagery of the area surrounding the site indicated that much of the surface has been modified by residential development, agricultural and ranching activities. Thus historical black and white aerial photography was assembled including USGS 1958 1:62,500 obliques and, 1948 and 1949 1:20,000 stereo pairs from the Texas Natural Resources Information System (TNRIS) covering the 5-mi area surrounding the site. The 1948 and 1949 photos were noted to be of good quality and minimal distortion and indicated less surface modification from anthropogenic activities compared to the 1958 photos which provided much less contrast. The 1948 and 1949 photo set was used for a detailed evaluation of surface lineaments.

The photographs were analyzed to identify surface features of linear to sub-linear expression of possible fault off-sets or fold hinge-lines or limbs manifested topographically as ridge lines or stream segments. The stereo pairs were indexed and tiled according to the master index file provided by the TNRIS. For the analysis, the photographs were evaluated for the following feature classifications and were cataloged for mapping and further evaluation through field reconnaissance:

1. Vegetation lineament: Interpreted features associated with noted changes in vegetation type, density or distribution. The features may result from anthropogenic activity (agricultural or land development) or be associated with changes in the soil type due to near-surface structure. This class of features dominated and those that could readily be attributed to anthropogenic activity were not individually classified.

2. Stream lineament: Linear reaches of streams and rivers over some distance that departed from the more typical meandering nature. These features may indicate local incision due to structural changes to local base level or be a local control on the river course due to the interception of vertical change in the stratigraphy.

3. Topographic lineament: Linear expression in the surface topography such as ridge lines and valleys. These features may indicate structure being accentuated by erosional processes.

4. Tonal Contrast lineament: Distinguishable variation in color tone (black-white scale) which differs from adjacent features. These features may result from shading along ridge lines or from the photographic development process.

Any features that could not be grouped into one of the above classifications were noted as unclassified. Linear features readily identified as anthropogenic, or an artifact of the photographic development, were not classified due to the number and density of such features present in each aerial photograph.

A total of 184 potential lineaments was identified within the surrounding 5-mi radius of the CPNPP site. Of these, 118 were classified as vegetation lineaments, 29 tonal lineaments and 37 were unclassified. Also, several stream lineaments were noted and were further evaluated using the aerial photography as well as field reconnaissance.

The most significant stream lineament can be noted on Figure 2.5.1-217 along the Brazos River just over 5-mi east of the CPNPP site. This straight reach of river also aligns with the upstream reach of the meander to the north. Further evaluation of the aerial photography did not indicate any vegetation, tonal or topographic lineaments that could be projected between these noted reaches in the Brazos River. This section of the Brazos River and other noted stream lineaments, along with the 37 unclassified lineaments, were further evaluated with field reconnaissance.

The field reconnaissance of the 37 unclassified lineaments noted extensive modification to the ground surface since the 1940’s photography as well as significant changes in vegetation cover and access restrictions. None of the unclassified lineaments that could be accessed could be confirmed from field observations and are deemed artifacts in the photography. The stream lineaments such as the Brazos River were confirmed to be controlled by local stratigraphy. This was concluded from the lateral continuity of bedding that could mapped on the aerial photography as well as field observations of outcrops along the Brazos River that indicated undeformed, horizontal bedding.

2.5.3.3 Correlation of Earthquakes with Capable Tectonic Sources

There is no seismicity within the 25-mi-radius site vicinity and therefore there is no spatial correlation of earthquake epicenters with known or postulated faults, other tectonic features, or other geomorphic features (Figure 2.5.2-201). As part of this COL application, the EPRI earthquake catalog was updated to incorporate southern United States earthquakes that occurred between 1985 and 2006 (see Subsection 2.5.2.1.2. The updated earthquake catalog contains no earthquakes with body wave magnitude (m_b) ≥3.0 with more than 50 mi of the CPNPP Units 3 and 4 site.

2.5.3.4 Ages of Most Recent Deformation

No faults or tectonic deformation has been identified at the surface within 25 mi of the CPNPP Units 3 and 4 site. The region in fact has experienced only sedimentation and erosion since the Permian Period, the last time of faulting or uplift in the area. The only disruptions to completely planar bedding are the two localized, probably sedimentary, features described in Subsection 2.5.1.2.4.1 and Subsection 2.5.1.2.4.2. These features were most likely developed in the Cretaceous Period.

2.5.3.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Sources

There are no tectonic bedrock faults within the 5-mi-radius site area. Consequently, it is concluded that there is no correlation of geologic structures in the site area to regional, capable tectonic sources.

2.5.3.6 Characterization of Capable Tectonic Sources

On the basis of data presented in Subsection 2.5.1 and previous discussions in Subsection 2.5.3.4, there are no capable tectonic sources within 5 mi of the CPNPP Units 3 and 4 site.

2.5.3.7 Designation of Zones of Quaternary Deformation in the Site Region

There are no zones of Quaternary deformation associated with tectonic faults requiring detailed investigation within the 5-mi-radius site area. A review and interpretation of aerial photography and available geotechnical boring logs, coupled with aerial and field reconnaissance, identified no possible Quaternary deformation in the site area.

2.5.3.8 Potential for Tectonic or Non-Tectonic Deformation at the Site

The potential for surface deformation aside from faulting was also investigated at the CPNPP site. This included tectonic non-fault deformation, such as folding, and non-tectonic deformation such as glacial rebound, ground collapse, volcanic

intrusions, salt movement, and anthropogenic activities. None of these potential hazards pose a threat to deforming the surface near the CPNPP site.

2.5.3.8.1 Potential for Tectonic Deformation at the Site

There is no potential for tectonic surface deformation at the site. There are no capable tectonic faults within the 25-mi-radius site vicinity.

2.5.3.8.2 Potential for Non-Tectonic Deformation

The potential for non-tectonic deformation at the site is negligible. The site is far from any geologic, non-tectonic sources of deformation such as salt domes or volcanic intrusions. It has been concluded that anthropogenic activities occurring near the site do not pose a hazard for surface deformation.

2.5.3.8.2.1 Potential Sources of Geologic Deformation

There is no evidence of non-tectonic deformation at the CPNPP 3 and 4 site in the form of glacially induced faulting, collapse structures, salt migration, or volcanic intrusion. There are no documented examples of glacially induced faulting in the 200-mi-radius site region. No piercement-type salt domes are located within the 5-mi-radius site area. The nearest salt dome is about 105 mi east in the East Texas Basin. No new data indicate a salt dome is located closer to the site. The CPNPP 3 and 4 site is founded on Glen Rose Formation limestone, a limestone not favorable for dissolution or karst-related hazard (CPNPP Units 1 and 2 FSAR). Topographic maps, aerial photographs, and rock samples within the area do not reveal any karst formation or associated collapse or subsidence. The only potential exceptions to this are the warping of the Cretaceous units in two locations, which could be related to minimal karst processes. No new evidence of sinkholes or solution cavities was found within the 5-mi-radius of the site area since the initial CPNPP Units 1 and 2 FSAR, which also concluded that "solutioning activity in the limestones beneath the site does not exist" (CPNPP Units 1 and 2 FSAR). The Texas Grand Prairie is part of a stable continental region, and no Tertiary or Quaternary volcanic activity is found within the 200-mi-radius region. The youngest documented magmatic activity in the site region is Mesozoic in age, and is spatially associated with the Balcones fault zone (Reference 2.5-215) at a distance of about 100 mi from CPNPP Units 3 and 4.

2.5.3.8.2.2 Potential Sources of Human-Related Deformation

There is no hazard due to human activity at the CPNPP Units 3 and 4 site. There are no mining activities within the site area that may produce man-induced surface collapse. Mining activities were summarized from a review of the Texas Mining and Reclamation Association and the Texas Bureau of Economic Geology as well as screening the 2006 satellite imagery of the site vicinity of any evidence of mining activities. No subsurface mining activities were identified and only one surface aggregate (sand and gravel) mine was identified within the 5 mile radius of the site. The surface mining activities are open excavation, strip mining of

Paluxy sand. The Squaw Creek Quarry is located just north of the SCR, but is not a source of significant surface deformation.

However, there are many oil and gas production-related activities within the area, which may include types of water or fluid removal and injection. Relevant to surface deformation at the site is the effect of fluid extraction on surface subsidence. These activities and their associated hazards were discussed in detail in Subsection 2.5.1.2.5.10.

2.5.4 Stability of Subsurface Materials and Foundations

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4 with the following.

In conformance with Regulatory Guide (RG) 1.206, this subsection presents information on the properties and stability of surficial soils and underlying rock formations (geotechnical site characterization) that may affect the nuclear power plant facilities, under both static and dynamic conditions. Data evaluation and analyses are presented to demonstrate that the site is stable and free of significant geologic or geotechnical hazards under static or seismic conditions that could adversely affect stability and function of the Comanche Peak Nuclear Power Plant (CPNPP) Units 3 and 4 safety-related (seismic category I and II) plant components. Geophysical and geotechnical data integrated with the site geologic model and plant foundation layout show foundation interface conditions, and support development of site dynamic profiles for the Ground Motion Response Spectra (GMRS) and Foundation Input Response Spectra (FIRS) for seismic category I and other structures. Geotechnical analyses demonstrate that the site geotechnical and foundation conditions are enveloped by the US-APWR DCD Standard Design criteria, and that no unusually adverse geotechnical conditions will be encountered during plant construction.

A map showing locations of the CPNPP Units 3 and 4 relative to the existing CPNPP Units 1 and 2 is presented on Figure 2.5.4-201. Engineering geologic and geotechnical investigations and analyses were performed by William Lettis & Associates Inc. (Walnut Creek, California), Fugro West, Inc. (Tustin, California), and Fugro Consultants, Inc. (Houston, Texas).

The information presented in this subsection was developed using existing data from the investigation performed for the CPNPP Units 1 and 2 Final Safety Analysis Report (FSAR; Reference 2.5-201), as well as new data generated from field investigations for CPNPP Units 3 and 4, completed between late 2006 and mid 2007. Subsurface conditions of the site are characterized by geologic reconnaissance mapping, exploratory drilling and borehole testing, geophysical investigations, and laboratory tests conducted on soil and rock samples in compliance with NRC Regulatory Guides 1.132 and 1.138.

Figures 2.5.4-202 and 2.5.4-203 show exploration locations described in the following subsections. The geotechnical exploration, laboratory testing program and supporting analysis clearly demonstrate the geologic lateral and vertical variability within the CPNPP Units 3 and 4 footprint areas, and show that the site conforms to a relatively “uniform” site condition.

The following subsections discuss the subsurface conditions and the properties of subsurface materials. The subsurface is stratified into layers with distinctive properties defined from the field and laboratory data. These layers are correlated to geologic formations and are referred to as engineering Layers A through I (shallowest to deepest, respectively). These engineering layers are part of the Glen Rose and Twin Mountain formations that overly the Mineral Wells Formation.

Of particular note is engineering Layer C, which is a massive limestone layer within the Glen Rose Formation. This limestone layer provides the foundation bearing layer for all seismic category I and II structures. The following discussions will refer to these engineering layers and respective geologic formations interchangeably as appropriate. A detailed discussion of the engineering stratigraphy is provided in Subsection 2.5.4.3.1.

2.5.4.1 Geologic Features

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.1 with the following.

This subsection discusses site-specific geologic processes, materials, and conditions relative to the potential for foundation instability or adverse performance at the CPNPP Units 3 and 4 site. Subsections 2.5.1 and 2.5.3 provide background discussion regarding the geologic setting and regional framework, tectonic setting and history, and potential geologic hazards that are referenced or expounded upon in this section. The focus of the following discussion is a site-specific evaluation of these geologic conditions and features within the CPNPP Units 3 and 4 plant foundation and influence zone pertaining to the following issues:

- Geologic Stress Conditions and Structure
- Adverse Mineralogy and Zones of Weathering or Alteration
- Karst or Zones of Dissolution
- Tectonic Ground Failure (Paleoearthquake)
- Landslides and Slope Stability
- Non-Tectonic Deformation and Volcanism
- Groundwater Conditions/Withdrawal
- Man-Induced Activity

2.5.4.1.1 Geologic Stress Conditions and Structure

The CPNPP site is located within a stable continent area (Subsection 2.5.3) with relatively low stress conditions and no active structural deformation within the 25 mi radius around the site. Late Cretaceous bedrock of the Glen Rose and Twin Mountain formations that underlie the site and locally crop out at the surface is nearly horizontal bedded and undeformed. Individual rock strata have been confidently identified and traced through the plant site, and extend to correlative elevations in boring logs and construction excavation photographs from the CPNPP Units 1 and 2, located about 2000 ft to the east. Figures 2.5.4-204 and 2.5.4-205 show the uniform stratigraphy and correlation of individual rock strata.

Resistant markers beds of the Glen Rose Formation that crop out at the surface follow topographic contour lines, and are continuous and undeformed throughout the plant area, as observed during field mapping and inspection of historic and modern aerial photographs and satellite images. The continuity and uniform, nearly horizontal condition of the bedrock provide evidence of a lack of active structural deformation or localized stress since the late Cretaceous.

As discussed in Subsection 2.5.1, the regional primary tectonic stress is horizontal compression oriented in a northeast-southwest direction. This stress is relatively low (in comparison with active tectonic regions) and is distributed without localized concentrations that could be problematic with respect to geologic or excavation stability (e.g., creep, heave). Possible local vertical or horizontal stresses related to erosion or other “unloading” processes should be low, based on relatively low erosion rates, gentle topography, and lack of past Pleistocene continental glaciations at this latitude. Since the Cretaceous rocks at the CPNPP site have been buried only shallowly, the bedrock has equilibrated to surficial temperatures and inter-granular stress and has had considerable time to be relieved. Therefore, unrelieved thermal stress is negligible at the site. Similarly, the slight burial of rocks at the site, along with long near-surface residence times, suggests that residual stresses resulting from the removal of overburden would be minimal at the site. Based on this low stress condition, no special hazard or engineering mitigation is expected to be necessary to control possible floor heave or lateral instability of deep excavations at the site.

The CPNPP Units 1 and 2 FSAR discusses rock stress relief measurements associated with general plant site excavation recorded in two extensometers. Maximum stress relief of 0.02 in was measured by the extensometers during deep excavations (about 30 ft to 60 ft) into upper Glen Rose Formation strata that are laterally contiguous with the rock strata that will be excavated for the CPNPP Units 3 and 4 plant site and seismic category I and II foundations. No occurrences of high stress or stress-induced instability are described, and construction photographs of foundation excavations show that unsupported vertical cuts in the rock approximately 40 ft to 60 ft high performed well without significant spalling or instability. Similar performance is expected during excavation at the CPNPP Units 3 and 4 plant site.

2.5.4.1.2 Adverse Mineralogy, Zones of Weathering, or Weak Materials

Subsection 2.5.4.2 provides a description of the geologic and engineering stratigraphy referenced in this subsection and defined in Subsection 2.5.4.3. Excavation of the CPNPP Units 3 and 4 site to plant yard grade elevation 822 ft completely strips surficial soils and a surficial weathered rock zone (generally several feet thick) from the footprint areas of the safety-related plant facilities, exposing the upper limestone beds of Glen Rose Formation engineering Layer A (Figures 2.5.4-209, 2.5.4-210, and 2.5.4-211, Subsection 2.5.4.3). Additional excavation to the targeted average foundation elevation of 782 ft for safety-related plant structures provides foundation support in confined rock at the top of the massive limestone of Glen Rose Formation engineering Layer C. Foundation

excavation side-cuts are made in the Glen Rose Layer A limestone, and shale of Layer B (generally 40 ft thick). At and below foundation elevation, the Glen Rose Formation engineering Layers C through F consist principally of massive and competent limestone, to a depth of about 150 ft to 160 ft below foundation level. These rock strata exhibit typical Rock Quality Designation (RQD) and percent recovery of greater than 90 percent (Figure 2.5.4-240). RQD measures core recovery percentage, which incorporates only pieces that are greater than 4 inches in length. Visual examination of rock core, petrographic classification, and geotechnical index testing indicates that these deeper Glen Rose Formation limestone strata do not contain zones or beds with adverse mineralogical conditions, and are sound, competent, and stable for foundation support. Thin shale beds are interspersed within the Glen Rose Formation Layers C through F, but represent a small total percentage of the rock, and are confined by thicker beds of sound limestone.

Below the Glen Rose Formation, interbedded limestone, shale, and sandstone of the Twin Mountain Formation (engineering Layers G through I, Subsection 2.5.4.3) extend to a depth of about 380 ft to 400 ft below plant foundation grade. No indications of unusual or adverse mineralogy were found in rock cores, or indicated by petrographic and geotechnical index testing in these strata. Some discontinuous and relatively thin (about several inches) coal occurrences are noted in some intervals of the Twin Mountain Formation, and localized zones or thin beds of poorly cemented sandstone and soft shale are also encountered. However, these materials represent a small percent of the total thickness of the Twin Mountain Formation, which otherwise consists of massive and relatively competent rock. The localized coal occurrences and poorly cemented sandstone and weak shale strata are separated from the plant foundation grade by the thick sequence of competent Glen Rose Formation limestone that extends through the main zone of foundation influence. Therefore, these materials and strata do not adversely affect the performance or stability of the plant foundations.

Shale beds in the lower portion of Glen Rose Formation Layers A and B are potential low strength interfaces that daylight into the excavation slopes. These shale strata are horizontal to subhorizontal, a geometry favorable for stability (Subsection 2.5.5). However, shale strata are considerably weaker than bounding, stiff limestone strata, and may undergo softening and surficial slaking at the excavation cut interface. They are therefore conservatively assumed to represent possible sliding surfaces or detachment zones for rock masses in the excavations. Subsection 2.5.4.5 discusses engineering analyses and design approaches to address this issue. The design approach of maintaining stable deep excavations (e.g., rock bolts) is a typical construction procedure for bedded rock formations. Because the excavations are backfilled after construction, shale beds do not pose a hazard or performance issue for the stability and performance of CPNPP Units 3 and 4 in the as-built condition. The extensive network of exploratory borings made throughout the plant footprints (Figure 2.5.4-202), identified no adverse shale beds or weak/altered zones that could impact foundation stability issues for the seismic category I or II foundations are encountered at, or immediately below, the foundation subgrade elevations.

In addition to the extensive vertical core borings drilled beneath the structure footprints (Figure 2.5.4-202), four inclined core borings (B2009I and B2011I under Unit 3 reactor building, and B1009I and B1011I under Unit 4 reactor building) and a crossing network of geophysical seismic refraction lines were incorporated in the exploration program to verify the absence of steep or vertical rock structures and shear/altered zones. Borehole optical and acoustic surveys in select core borings provided continuous logs of the borehole walls, and compared fracturing in recovered core against in situ rock mass conditions. These explorations verified that the Glen Rose and Twin Mountain formations rock is tight, sound, and free of significant shears or altered zones that could potentially impact foundation or excavation stability.

Reconnaissance of exposures surrounding the site, a review of aerial photography, and examination of borings drilled as part of the geotechnical investigation showed no zones of unusual deep enhanced weathering or structural weaknesses, such as fractures or joint sets in-filled with weak material. Outcrop exposures of the Glen Rose Formation, and CPNPP Units 1 and 2 construction excavation photographs, show that the rock mass is tight and does not contain continuous joint systems or dense fracture zones. Random and dispersed minor joints/fractures typically are constrained within individual horizontal beds and do not exhibit shearing or displacements of strata. They also typically have interlocked rock-to-rock contacts without in-fillings of weak clay or sheared gouge. Furthermore, neither the petrographic analysis of samples acquired from core borings drilled as part of the geotechnical investigation nor the samples taken from exposures surrounding the site indicate any secondary alteration of minerals that weaken the rock mass.

2.5.4.1.3 Karst or Zones of Dissolution

Subsection 2.5.1 presents an evaluation of karst or limestone dissolution potential. No references of features indicative of karst development in the site vicinity (25 mi) are found in the literature. The CPNPP Units 1 and 2 FSAR concludes that "solutioning activity in the limestone rocks beneath the site does not exist" (Reference 2.5-201), and construction photographs of tall (40 ft to 60 ft high) and extensive vertical excavations in Glen Rose Formation bedrock do not show any significant solutioning or karst features. No evidence of old or new karst or significant dissolution features is found in the site area (5 mi radius) and site (0.6 mi radius) for CPNPP Units 3 and 4 by review of aerial photographs and field reconnaissance mapping, including extensive outcrop exposures of the Glen Rose Formation.

There is no evidence of large or interconnected solution voids or karst development through the Glen Rose Formation limestone section. Geotechnical boring logs produced from the dense pattern of vertical and inclined rock core borings made throughout the plant site (Figure 2.5.4-202), water pressure packer testing, surface and borehole geophysical surveys, and borehole televiewer surveys indicate that the rock mass is tight and impermeable. Petrographic analysis on limestone core samples indicates that these rocks have a

considerable fines (clay/silt) component, reducing the potential for development of major dissolutioning (Subsection 2.5.4.2). Small, discontinuous solution vugs were observed in some recovered rock core intervals, but these typically are in-filled with silt and do not interconnect to form large cavities or voids. A thorough review of the geotechnical boring logs and the actual recovered cores did not indicate any significant zones of solutioning.

The findings indicate that karst and dissolutioning are not significant hazards for CPNPP Units 3 and 4.

2.5.4.1.4 Tectonic Ground Failure (Paleoearthquake)

Subsection 2.5.3 describes evaluation of potential tectonic surface deformation. No Quaternary faults, liquefaction features, or possible tectonic features have been identified within the site vicinity (25 mi radius) by the U.S. Geological Survey, the compilation of local mappings, or aerial photograph analysis and field reconnaissance within the site area (5 mi radius) for the CPNPP Units 3 and 4 investigations. Site subsurface explorations demonstrate that competent Glen Rose Formation bedrock occurs at shallow depths throughout the plant area. This rock is stable and not subject to earthquake-induced ground failure from liquefaction, lateral spreading, or lurching. This information indicates no potential hazard related to seismically induced ground failure to the CPNPP Units 3 and 4 safety-related facilities that are founded in sound Glen Rose Formation bedrock.

2.5.4.1.5 Landslides and Slope Stability

The existing terrain at the CPNPP Units 3 and 4 site has relatively gentle slopes, with the exception of the localized slope that descends to SCR along the north margin of the site. The footprint envelopes for the plant power blocks are set back from the top of the reservoir slopes at a minimum distance of 150 ft to 200 ft. No evidence of past significant landsliding is identified in the site (0.6 mi radius) by aerial photograph evaluation or field reconnaissance mapping (Subsections 2.5.1 and 2.5.3). Intact outcropping strata of Glen Rose Formation bedrock are visible tracing along topographic contour in the area of the reservoir slope on pre-reservoir and modern aerial photographs. Discrete bedrock strata of the Glen Rose Formation can be correlated between borings along the north margin of the plant site near the top of the reservoir slope. This suggests that the bedrock has not been displaced by past landsliding.

Localized surficial erosion and raveling have occurred in fill and/or native colluvial soils on the reservoir slopes. This is a surficial condition that does not present a significant slope stability hazard to the CPNPP Units 3 and 4 plant site.

Shale layers in the upper parts of the Glen Rose Formation Layers A and B (Subsection 2.5.4.3) daylight in the reservoir slopes (Figures 2.5.4-210, 2.5.4-211, 2.5.5-202, and 2.5.5-203). These beds could act as potentially weaker zones, especially if softened by perched groundwater conditions. The shale beds could serve as sliding surfaces for rock sliding in temporarily excavated slopes for the

Ultimate Heat Sink (UHS) structures along the top of the reservoir slope. Engineering analysis for this potential condition is presented in Subsection 2.5.5, and shows an adequate factor of safety.

Thick, undocumented fill was placed in former topographic swale areas north and east of the CPNPP Units 3 and 4 power block footprints (Figure 2.5.4-212). The fill extends to the margin of SCR, and is in hydraulic communication with the reservoir. As a result, groundwater occurs as a perched condition in the swale fill at higher elevations than encountered in the bedrock surrounding the in-filled swale areas. Fill in the eastern swale area has undergone differential settlement, indicated by ground cracks and depressed areas. Stability analysis of the swale fill areas is included in Subsection 2.5.5 and demonstrates an adequate factor of safety against this failure mode. Isopach contour maps showing the elevation of the top of Glen Rose Formation bedrock (Figure 2.5.4-213), Glen Rose Formation Layer C foundation strata (Figure 2.5.4-214), and material exposed at plant grade of elevation 822 ft (Figure 2.5.4-215) show that the swale fill is largely stripped from the plant areas by site grading. Safety-related plant structures are supported by foundations bearing into the competent Glen Rose Formation Layer C limestone below plant grade. As a result, any swale fill that may remain around the perimeter of the plant site does not affect the stability or performance of plant safety-related facilities.

2.5.4.1.6 Non-Tectonic Surface Deformation and Volcanism

Subsection 2.5.3 discusses potential sources of non-tectonic deformation and regional volcanic conditions. The potential for non-tectonic deformation from regional ground collapse, salt migration, glacial rebound, and volcanic processes is negligible. No evidence of deformation from these mechanisms is documented in the site region (200 mi radius). Pleistocene continental glaciations did not extend southward to the latitude of the site region. Thick continental crust and shallow bedrock occur in the site vicinity (25 mi radius), and layers of sedimentary basins and thick regional sequences of collapsible weak sediments do not occur within the site vicinity. No piercement-type salt domes are located within the site area (5 mi radius), and the nearest salt dome is located about 105 miles to the east. No Tertiary or Quaternary volcanic activity occurs within the site region, and the youngest regional magmatic activity is Mesozoic in age and located about 100 miles south.

Therefore, these geologic processes do not present a hazard to the CPNPP Units 3 and 4.

2.5.4.1.7 Groundwater Conditions/Withdrawal

Subsection 2.5.1 summarizes potential issues related to groundwater withdrawal from aquifers beneath the site. Subsection 2.4.12 discusses site groundwater conditions, aquifers, and local and regional groundwater resources and usage. The primary drainage in the site area is SCR, an artificial impoundment of Squaw Creek. The pool elevation of SCR is 775 ft, approximately 47 ft below post-

construction plant grade elevation of 822 ft. The measured data from the monitoring wells within the CPNPP Units 3 and 4 area suggest that the piezometric levels range between about elevation 775 ft and 858 ft, with a number of wells remaining dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site. Historical groundwater levels from a few observation wells in Somervell County around the plant site suggest that the groundwater levels in the area range between about elevation 600 ft and 760 ft (Reference 2.5-201).

Geologic strata underlying the CPNPP Units 3 and 4 plant site are within the measured groundwater and SCR pool elevations, and are cemented limestone and shale of the Glen Rose Formation. The induration and low compressibility of these rock materials makes them resistant to possible groundwater drawdown and fluctuation-induced settlement. Groundwater withdrawal is therefore not considered to be a significant potential hazard at the site. Subsection 2.5.4.6 provides a discussion of groundwater control measures for plant foundation excavation.

2.5.4.1.8 Man-Induced Activity

Subsection 2.5.1 discusses potential man-induced activities in the site area (5 mi) and site (0.6 mi), and their impacts on the CPNPP Units 3 and 4. No significant surface or subsurface mining operations exist within the site area, except for shallow aggregate mines. These mining operations do not pose a hazard to the site.

Significant natural gas resources have been identified in the site area, including production of gas in the Barnett Shale that has significantly increased in the past several years. Subsection 2.5.1.2.5.10 provides a detailed evaluation of the potential impacts of oil and gas production and related activities (e.g., exploration). In summary, these activities are determined to have no significant adverse impact on the stability and operation of the CPNPP Units 3 and 4.

2.5.4.2 Properties of Subsurface Materials

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.2 with the following.

This subsection presents a summary of the field investigation, sampling, laboratory testing, and subsurface material properties at the CPNPP Units 3 and 4 site. Figure 2.5.4-202 shows exploration locations in the plant power block area. The geotechnical investigation program and procedures for field investigations and laboratory tests conform to RG 1.132, Site Investigations for Foundations of Nuclear Power Plants and RG 1.138, Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants.

Information from published literature, regional and local maps, and historical information from exploration activities completed for the CPNPP Units 1 and 2 were all used to guide development of the field exploration program. The exploration program included multiple methods of exploration utilizing both traditional and state of the practice methods of subsurface exploration and in situ testing. Multiple independent, testing methods were used to measure and evaluate critical site characteristics such as seismic wave velocity. Soil and rock sampling conformed to the guidelines for spacing, depth, and sample frequency provided in the RG 1.132. Borings in the seismic category I and power block foundation areas were extended deeply into sound rock materials below foundation subgrade (typically 50 ft to 100 ft, and greater, below subgrade). Specific drill depths have been determined based on relationships between foundation width and depth of influence (e.g., Boussinesq pressure distribution diagrams), regulatory criteria (e.g., RG 1.132), and elevations of major geologic layer contacts. Geophysical testing included both surface and borehole geophysical methods. Selected borings extended far below foundation influence zones to screen for potential geologic hazards (e.g., karst, shears), and to obtain borehole geophysical surveys through possible Soil Structure Interaction (SSI) and ground motion site response influence zones. These deeper borings included both vertical and inclined borings to depths of between 200 ft and 550 ft below ground surface.

Geologists and engineers documented and logged samples of site materials obtained during the exploration work, and preserved them in the field for further analysis and laboratory testing. Details regarding the field exploration and laboratory testing programs are provided in Subsections 2.5.4.2.1 and 2.5.4.2.2, respectively. Field exploration and laboratory test data were integrated with available historical site data and published information to develop a comprehensive geologic model that includes geologic cross sections and subsurface contour maps for each major geologic stratum through the plant foundation influence zone. The number and density of explorations permit confident evaluation of geologic lateral and vertical variability within the CPNPP Units 3 and 4 footprint area, and show that the site conforms to a relatively “uniform” site condition. Laboratory testing and integration of field borehole test data (e.g., seismic velocity, water pressure “packer” testing, pressuremeter modulus tests) provide characterization of the engineering properties of site materials for geotechnical analyses (e.g., bearing capacity, settlement, slope stability) and comparison of site characteristics against the US-APWR Standard Design criteria.

These test data document that the site characteristics are consistent with the US-APWR Standard Design geotechnical parameters.

2.5.4.2.1 Exploration

The subsurface exploration for the CPNPP Units 3 and 4 consists of soil and rock borings, installation of ground-water monitoring wells, field packer tests, surface geophysical surveys, cone penetration test (CPT) soundings, geotechnical test pit

excavations, and borehole in situ and geophysical testing. The number and type of explorations performed for the CPNPP Units 3 and 4 are summarized in the following tables.

- Summary of Exploratory Borings (Table 2.5.4-201)
- Summary of Test Pits (Table 2.5.4-202)
- Summary of Monitoring Wells (Table 2.5.4-203)
- Summary of CPT Soundings (Table 2.5.4-204)
- Summary of Seismic Refraction Survey Lines (Table 2.5.4-205)
- Summary of In Situ Packer Tests (Table 2.5.4-206)
- Summary of Borehole Geophysical Testing (Table 2.5.4-207)
- Summary of In Situ Pressuremeter Testing (Table 2.5.4-208)

2.5.4.2.1.1 Exploratory Borehole Drilling and Sampling

Exploratory borehole drilling and sampling were performed using conventional geotechnical drill rigs mounted on trucks or tracked vehicles. Specific equipment used at each borehole was recorded on the boring logs. The boring locations are shown on Figure 2.5.4-202, and boring location coordinates, ground surface elevations, and depths are summarized in Table 2.5.4-201. A total of 141 geotechnical exploratory boreholes, advanced to depths ranging between 40 ft and 550 ft (below pre-graded ground surface), provide comprehensive characterization of subsurface conditions under the CPNPP Units 3 and 4 safety-related structures. Geophysical surveys and in-hole testing (e.g., seismic velocity, Pressuremeter modulus, and water pressure “packer”) were performed in select geotechnical borings to measure in situ soil and rock engineering properties.

Geotechnical borings were supplemented with 20 Monitoring Well stratigraphic core borings drilled to depths of between 15 ft and 100 ft (typical 100 ft depth) that were distributed within and around the plant power block areas. A subset of the Monitoring Wells within the immediate power block area is shown on Figure 2.5.4-202.

A total of 103 shallow Cone Penetrometer Test (CPT) soundings at 42 locations throughout the power block areas (Figure 2.5.4-202) facilitate evaluation of in situ properties of surficial soil and fill overlying rock, and help refine the elevation of the top of rock.

Surface geophysical surveys (e.g., seismic refraction) were performed at selected locations in the plant power block area to further refine the interface locations of shallow soil and fill material, independently measure in situ engineering properties

in soil and shallow rock, screen the site for possible large solution features in the bedrock surface, and facilitate extrapolation and integration of borehole data. Figure 2.5.4-202 shows the locations of geophysical surface surveys.

Field boring logs, daily field reports, and other field records were maintained by a rig geologist assigned to each drill rig. Soil materials are classified in conformance with ASTM D2487, Standard Classification of Soils for Engineering Purposes and D2488, Standard Practice for Description and Identification of Soils. Rock materials are classified in conformance with Brown, E.T. (Reference 2.5-419); Deere, D.U. (Reference 2.5-407); and Dunham, R.J. (Reference 2.5-418) suggested methods. All field geologists and engineers were trained under the project Quality Assurance (QA) Program. Data Collection Plans and borehole-specific Work Instructions provide QA control for the exploration locations, depths, techniques, sampling, and data collection (e.g., classification and logging). Senior geologists and engineers reviewed all field data collection activities and performed independent review of classification and logging operations.

Borings for geotechnical purposes were advanced in soil using Hollow Stem Auger (HSA) drilling techniques and equipment until auger refusal was encountered. Standard Penetration Test (SPT) drive samples were typically obtained on 2.5-foot intervals in soil materials, beginning at about 1 ft below ground surface. All SPT samplers consisted of 18-inch-long standard unlined split barrel drive samplers, and were in good condition. Auger stems had a nominal outside diameter of 8 in and an inside diameter of 4.5 in. Drive sampling by SPT method was conducted with automatic trip hammers with a weight of 140 pounds and a drop of 30 in per ASTM D1586. Blow counts were measured for three consecutive 6-inch driving intervals. In zones with high blow counts, driving was terminated at a count of 50 blows in any 6-inch interval and the actual penetration distance was recorded. Blow counts were recorded independently by rig geologists and drillers and immediately noted on the borehole logs.

After recovery, rig geologists selected representative portions of each SPT sample and placed each in one or more labeled glass jars with sealed, lined caps. All samples were immediately assigned alphanumeric sample identifications, photographed, described, and recorded on field logs. Use of thin-wall, 3-inch inside diameter Shelby tube samplers (ASTM D1587) was also tried for obtaining undisturbed soil samples. However, due to sampler damage caused by the presence of gravel, large-size particles, and very stiff to hard soil conditions, the attempts were not successful.

Field SPT energy measurements were made for each drill rig on select exploratory borings and recorded during sampling at several different intervals. The ratio of average measured energy to the theoretical potential energy of the SPT system is the energy transfer ratio (ETR). The ETR range of automatic hammers used at the CPNPP site ranges between approximately 66 percent and 92 percent of the theoretical potential energy, with an average value of 82 percent. These ETR values are within the range of typical values for automatic

hammers (Reference 2.5-407), and are factored in SPT-based engineering classification and analyses.

Once refusal was encountered, a steel or PVC casing was set if soil was present. The holes were then advanced using wire-line rock coring equipment and procedures as described in ASTM D2113. A five- or ten-foot long NQ or HQ size core barrel was used for all rock coring, and was typically advanced in 5-ft core runs. After each core run, recovered rock core was examined and classified by the geologist, photographed, and placed on PVC trays within wood core boxes. Core boxes were labeled on the outer and inner panels for identification. In addition to geologic classification, RQD and percent recovery were measured and recorded for each core run. During drilling, the geologist noted drilling response, drill return water characteristics and percent circulation (water "take"), and rotation and down feed pressures applied by the drill rig. Discrete samples of rock core were cut from the recovered core run by the geologist, wrapped in bubble wrap, and placed in PVC sample tubes for laboratory testing. These PVC core samples were stored in cardboard boxes along with the remaining rock cores, with the exception of samples that were sent to the laboratory for testing. All samples were assigned alphanumeric sample identifications, photographed, described, and recorded on field borehole logs.

Permanent PVC casing was installed and grouted in place in select boreholes for downhole geophysical surveys. PVC casing for this purpose was 4-in diameter riser pipe grouted in place using a cement bentonite grout mix to provide a consistent seal between the casing and the surrounding soil and rock.

At the completion of drilling or in situ borehole testing, all boreholes were backfilled using a cement-bentonite grout. The grout was placed by pumping through a tremie pipe from the bottom of the hole. The borings with grouted-in PVC casings used for geophysical tests were over-drilled to remove the PVC before backfilling with cement-bentonite grout.

Copies of the exploration records are provided in the following supporting documents:

- Boring logs: Boring Logs Data Report
- Test pit logs: Geologic Test Pit Report
- CPT logs: CPT Sounding Data Report

2.5.4.2.1.2 Monitoring Wells

Geologic stratigraphic profiles were established at each monitoring well (described in Subsection 2.4.12) location by drilling and sampling a continuous "profile" core boring using the same general methodology and equipment as for the geotechnical core borings. Each profile boring was drilled to a depth of 100 ft and was located within about 15 ft from the monitoring well holes. The locations of

groundwater monitoring wells constructed as part of the CPNPP exploration are shown on Figure 2.5.4-202, and the location coordinates, ground surface elevations, and depths for all monitoring wells (including those outside of the power block areas) are summarized in Table 2.5.4-203. In order to obtain continuous sampling in soils overlying rock for the monitoring wells, CME soil core barrels that fit within HSA sections were used through the upper residual and fill soils. Upon CME/auger refusal, continuous wireline rock coring with NQ/HQ equipment was used to complete borings in rock to the target depth. The lithologic descriptions provided in the exploratory boring logs were evaluated by the hydrogeologist to develop the monitoring well design (e.g., screen interval) on a case-by-case basis. A detailed discussion of the monitoring wells is provided in Subsection 2.4.12. Although the focus of the monitoring well profile core borings was to assess the groundwater conditions and facilitate well design, they also provide important supplemental subsurface data that provide control for cross sections and extrapolation of the geologic model outside of the power block area.

2.5.4.2.1.3 Geologic Test Pits

Three geologic test pits (A, B, and C) were excavated and logged as part of the field exploration along the margin of a filled swale east of CPNPP Unit 3 (Figures 2.5.4-202 and 2.5.4-212). Test pit locations were targeted to help define the margin of the swale fill, and were guided by information from geotechnical borings, field reconnaissance, and review of historic construction aerial photographs. Table 2.5.4-202 provides summaries of the test pit location coordinates, surface elevations, orientations, dimensions, and depths. Specific objectives of the excavation program included:

- Define the western edge of a backfilled valley located in an old construction parking lot
- Identify contacts between artificial fill lifts
- Identify the contact between artificial fill and residual soil
- Identify and describe any in situ soil horizons
- Use this information to check refraction survey lines performed across the area

Observations from the three test pits confirmed the western limit of artificial fill in the swale area east of CPNPP Unit 3, and allowed evaluation of the condition of existing fill. Artificial fill was distinguished from native soil based on the presence of disturbed nature and matrix suspended, angular gravels or rock pieces (up to boulder size), relatively poor sorting, and a lack of bedding. Test pit exposures suggest that the swale fill is uncontrolled and heterogeneous, and exhibits variable and unpredictable properties.

2.5.4.2.1.4 Cone Penetration Test Soundings

Cone Penetration Test (CPT) soundings were performed to provide near continuous subsurface profiles through soil and fill at selected locations. The CPT locations are shown on Figure 2.5.4-202, and location coordinates, ground surface elevations, and depths are summarized in Table 2.5.4-204. CPT data were used to evaluate the depth to bedrock and the in situ engineering characteristics of the subsurface soils. Select CPT soundings were located near or adjacent to geotechnical borings to provide correlation of logging, and also were in places distributed along surface geophysical lines to provide data correlation and control.

The CPT soundings were performed at 42 locations to depths ranging between about 0.5 ft and 37 ft below the ground surface. When shallow refusal was encountered, the CPT sounding was repeated several times by moving the equipment about 5 ft to10 ft in order to attempt deeper penetration. A total of 103 CPT soundings were performed at the 42 designated locations. Shear wave velocity measurements were also made using a seismic cone at 37 CPT locations.

The cone penetrometer used at the CPNPP Units 3 and 4 site was mounted on a 25-ton truck and consisted of a 1 ½ -inch-diameter rod with a 15 cm^2 (2 1/3 in^2), 60-degree-apex-angle cone tip per ASTM D5778. The cone was equipped with electronic load cells that measured both point resistance and frictional resistance between the soils and the cylinder side of the cone. The seismic cone was also equipped with geophones near its tip to allow for the measurement of soil shear wave velocity.

CPT data are presented in a graphical format versus depth and include:

- Cone tip resistance plot in tons/ft^2 (tsf)
- Friction sleeve resistance plot in tsf
- Pore water pressure plot in tsf
- Friction ratio plot in percent

The following data interpretations were performed for each location where seismic CPT data were collected:

- Average arrival time vs. travel distance
- Shear wave interval velocity vs. depth
- Dynamic (small-strain) shear modulus vs. depth

The CPT logs, along with data collected during the seismic testing, and the calculated and interpreted results are presented in the CPT Sounding Data Report.

2.5.4.2.1.5 In Situ Packer Testing

In situ double packer tests ("straddle packers") were performed in Glen Rose Formation bedrock in 6 boreholes at depths ranging between 20 ft and 175 ft. A summary of the locations, depths, and permeability results of the in situ packer tests is provided in Table 2.5.4-206. All tests included 5-ft long isolated borehole intervals. The objective of the packer test investigation was to measure the in situ transmissivity of the Glen Rose Formation engineering stratigraphic Layers A through E within the plant power block area. The selection of the test intervals was based upon review of the boring logs and consideration of 1) stratigraphy, contacts, discontinuities, and weathering, and 2) the plant yard grade (822 ft) and foundation excavation depths. The testing intervals were targeted to identify the average and variations in transmissivity, both laterally and vertically, in rock within the foundation and excavation influence zones. Field packer testing was in general accordance with the requirements of ASTM D4630, and incorporated the Lugeon and U.S. Bureau of Reclamation procedures (Table 2.5.4-206). In all, 40 packer tests (inflations) were performed within six boreholes. Table 2.5.4-206 and Figure 2.5.4-224 summarize the results from borehole packer testing.

All tested intervals exhibited very low transmissivity (practically impervious), with the highest measured value of about 6.38×10^{-7} cm/s. Each of the tested Glen Rose Formation layers exhibited similar ranges in transmissivity without clear differentiation by layer. Packer test results indicate that in situ Glen Rose Formation rock (including rock mass joints and other features) exhibits low transmissivity and is in a "tight" condition. As discussed in Subsection 2.5.4.1.7, the true groundwater table at the plant area is anticipated to be below an elevation of about 760 ft (about 62 ft below plant yard grade), and below anticipated foundation excavations. Even in the event that perched groundwater theoretically develops at higher elevations in the rock mass, inflows into foundation excavations would be slow and easily controllable by standard excavation dewatering procedures (e.g., sumps and pumps). Additional information and details regarding packer testing and results is provided in the Field Packer Test Results Report.

2.5.4.2.1.6 In Situ Pressuremeter Tests

Pressuremeter tests (PMT) were performed in seven boreholes distributed throughout the CPNPP Units 3 and 4 power block area to evaluate in situ rock mass deformation moduli (Young's modulus) in vertical profiles through the likely foundation influence zone. Data obtained from PMTs, in combination with data obtained using other in situ and laboratory tests, form the basis for selection of the design modulus values.

A total of 62 tests were performed, of which 59 produced useful data. Borings were selected for pressuremeter tests based on evaluation of the borehole conditions, boring location respective to the structure footprints, and the overall lateral and vertical coverage of the site area and subsurface conditions. Specific intervals for testing were targeted based on the lithology.

Each PMT test began by lowering a single-cell pressuremeter device into a stable borehole section and inflating the cell membrane to deform adjacent material at a prescribed set of intervals. Three electronic displacement sensors, spaced 120 degrees apart, recorded displacements of the borehole wall in response to induced pressure.

Pressure was increased manually in small steps until the pressure was in the range of 400 to 500 psi. Subsequently, the pressure was reduced to about 50 percent of the maximum past pressure, and then increased again. The resulting unload-reload loop can be used to interpret the elastic behavior of the material (materials with linear elastic characteristics exhibit weak hysteretic behavior in that the plot of the reloading path closely follows the unloading path).

The pressure was then increased in steps to about 900 psi before completing a second unload-reload cycle. If the disturbance is small during the pressuremeter insertion, the slope of the loops will tend to be parallel. If disturbance is present, the first loop usually has a flatter slope than the second loop, and in most tests a third unload-reload loop was conducted.

PMT tests in the limestone were not carried out to the device’s pressure limit, because otherwise the resultant deformation of the nearby weak layers of shale could have caused the membrane to rupture. Limiting the pressure resulted in loss of only one membrane in 62 tests.

A summary of the locations, depths, and results of the in situ PMT tests is provided in Table 2.5.4-208. Detailed descriptions of the PMT methodology and results are provided in the Summary of the Pressuremeter Testing report.

2.5.4.2.1.7 Geophysical Surveys

Multiple surface and borehole geophysical test methods were used to evaluate site stratigraphy, screen for potential geologic hazards (e.g., karst), estimate engineering properties of in situ soil and rock mass, and help extrapolate information between borings for development of cross sections and a 3D geologic model of the CPNPP Units 3 and 4 power block area. The following geophysical techniques were performed:

- Surface Refraction Surveys by Fugro West Inc.
- Borehole Suspension P-S Logging by Geovision Inc.
- Borehole Down-Hole Seismic Logging by Geovision Inc.

- Borehole Acoustic Televiewer Logging by Geovision Inc.
- Borehole Combined Caliper, Gamma, Resistivity and Spontaneous Potential Logging by Geovision Inc.

Integration of these varied techniques permits independent measurement of critical site characteristics (e.g., seismic wave velocity), and produces a robust data set to evaluate potential lateral and vertical variability of engineering properties throughout the power block area. Integration of geophysical survey data is described in Subsection 2.5.4.4.

The locations of seismic refraction lines are shown on Figure 2.5.4-202, and the location coordinates, ground surface elevations, and line lengths are summarized in Table 2.5.4-205. Figure 2.5.4-203 shows the borehole locations where down-hole geophysical tests were performed, and Table 2.5.4-207 provides a summary of borehole geophysical survey tests. Test procedures and results of the surface and borehole geophysical tests are summarized in the following subsections.

2.5.4.2.1.7.1 Seismic Refraction Survey

Seismic refraction surveys were configured as a series of crossing straight lines, or isolated lines at different azimuth orientations, that extended between control boreholes, test pits, and CPT soundings. A total of 15 separate straight-line surveys cross the power block area, with a cumulative footage of about 8,350 lineal ft. Specific parameters for each seismic refraction line are provided in Table 2.5.4-205. The orientations of seismic refraction lines were varied to screen the site for possible geologic features with varying geometries. A dense array of crossing lines (Lines 7 through 10) spanned the in-filled swale area east of CPNPP Unit 3, with the objective to delineate the extent and geometry of swale fill with respect to the plant power block.

Surface refraction surveys were designed, performed, and analyzed by registered geophysicists. Each of the 15 seismic lines consisted of one or more 24-channel spreads, with a geophone spacing ranging between 10 ft and 15 ft. For each spread, both off-end and interior shots were used. Signals were produced using a propelled weight drop source (R.T. Clark Companies Inc.), propelling an 80 lb source through a 14- to 17-inch drop onto a strike plate. A 24-channel Geometrics Inc. seismograph was used to collect compressional-wave (P-wave) data from repeated and stacked impacts. Recordings were obtained by 10-Hz geophones in a vertical configuration for P-wave measurements, and horizontal configuration for shear-wave (S-wave) measurements.

Raw seismic refraction data were downloaded to a laptop computer and first arrivals (first-break picks) were selected and used to create plots of arrival time versus distance. A Time-Term analysis (delay-time) method was used to create a velocity section for each refraction line employing a combination of linear least squares and delay-time analyses to invert the first-arrivals.

Surface refraction results were checked against data developed by control borings, test pits, and CPT soundings, and used to develop initial 2-D layered velocity models. These velocity models were then evaluated to identify geologic contacts, determine the surface extent of existing fill, and evaluate the top of the rock profile to help identify any possible large dissolution features or shears. In most areas, the seismic refraction surveys were not able to confidently image the bedrock surface. Where possible, the results from the refraction surveys are incorporated in the site engineering stratigraphy and geologic model described in Subsection 2.5.4.3. Detailed descriptions of the seismic refraction survey and results are provided in the Seismic Refraction Survey Report.

2.5.4.2.1.7.2 Suspension P-S Velocity Survey

Borehole Suspension P-S logging was performed in 15 select borings by Geovision Inc., using an OYO Corporation Model 170 commercial probe. The P-S logging equipment obtains discrete P-wave and S-wave seismic velocity measurements in a borehole using a down-hole source, and is a current industry standard method for nuclear site characterization. Seismic velocity profiles developed using the P-S surveys provided the primary data source to characterize the seismic wave transmission characteristics of the site, and to characterize the site according to the US-APWR Key Site Parameters (DCD Table 2.0-1).

P-S logging was performed in select geotechnical HQ boreholes without casing, and using consistent vertical measurement intervals of 1.6 ft. Boreholes were selected to provide complete spatial coverage throughout the CPNPP Units 3 and 4 seismic category I facilities footprints, and general and power block area, as shown on Figure 2.5.4-203 and summarized in Table 2.5.4-207. Surveyed holes included the reactor center points for both of the CPNPP Units 3 and 4, a deep boring midway between the two reactor centerpoints, and distributed around the power block and UHS areas. These include the deepest geotechnical core borings that extended to depths of between about 300 ft and 550 ft.

For the P-S surveys, the down-hole source generated a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave was converted to P- and S-waves in the surrounding soil and rock that were detected by two separate receivers at fixed distances above the source. A field check of measured velocity was performed by comparing calculated velocities between the source and receivers, and independently between the two receivers. Acoustic televiewer and caliper surveys, described in following subsections, were used to determine the borehole dimensions and vertical deviation to help verify the quality of the imaged borehole and evaluate possible impacts from deviations in borehole diameter or inclination. Initial depth-velocity plots produced by the P-S surveys were plotted at a common scale and compared against the borehole geologic stratigraphy, RQD/percent recovery, and other collected geophysical data (e.g., gamma) to develop correlations between velocity layers and geologic/rock mass conditions. This process included field review of core samples between project geologists and Geovision, Inc.

personnel, and comparison of velocities measured in discrete shale and limestone marker beds in the Glen Rose Formation.

Excellent agreement was found between velocity layers and geologic stratigraphy (engineering layers presented in Subsection 2.5.4.3), including differentiation of lower-velocity shale intervals as thin as about 1 ft. Individual velocity profiles from successive borings were also indexed by elevation and key marker strata, and found to be very similar across the entire CPNPP Units 3 and 4 site, providing documentation of a high level of lateral uniformity in seismic velocity.

Figures 2.5.4-206, 2.5.4-207, 2.5.4-208, 2.4.5-209, 2.5.4-210, and 2.5.4-211 show correlations between P-S Suspension velocity profiles, the site stratigraphy, and other geophysical and rock mass parameters. Interpreted seismic wave velocity profiles define the vertical variations in P-wave and S-wave velocity through the site geologic stratigraphy. Summary velocities by principal geologic strata are presented in Subsection 2.5.4.4.2.1. An average composite S-wave velocity for the rock mass extending from plant yard grade to a depth of about 530 ft is in the range of about 4000 to 4500 fps. This corresponds to a “firm rock” condition, according to the US-APWR Key Site Parameters (DCD Table 2.0-1). Detailed descriptions of the borehole Suspension P-S logging and results are provided in the Borehole Geophysical Logging Report.

2.5.4.2.1.7.3 Down-Hole Velocity Survey

Down-hole seismic velocity surveys were performed in the two reactor center point borings, B-1000 and B-2000 (Figure 2.5.4-202), to provide independent borehole measurements (with respect to P-S surveys) of this critical site parameter. The down-hole technique consists of a single borehole geophone that is clamped to the inside of a PVC casing grouted into the borehole after completion of other geophysical techniques that use an uncased hole. The borehole geophone is lowered to the bottom of the casing and progressively raised and set at 2.5 ft to 5 ft vertical intervals for discrete measurements. These surveys were performed by Geovision Inc. using a Geostuff BHG-3, three-component borehole geophone that orients the geophone parallel to the axis of excitation at the surface. This orientation ensures that received signals are of maximum amplitude. The S-wave signals were generated by blows from a sledge hammer weighing approximately 16 lb against the ends of a wooden plank on smooth and level ground, with ends situated equidistant from the hole, and anchored by placing it under the wheels of a truck. The plank is struck alternately on either end and stacked to facilitate identification of the S-wave arrivals. The P-wave signals were generated by vertical blows to a metal plate placed on smoothed and level ground. The P-wave and S-wave velocities were calculated based on measured wave travel, times and distances between the source and receiver for each depth interval.

A Geometrics Geode seismograph is used to collect recorded data from the geophone. Reliable interpretation of the down-hole surface source extended to a depth of about 135 ft in boring B-1000, and 144 ft in boring B-2000.

Collected P- and S-wave data from the down-hole surveys were used to develop a layered travel time-depth and layered velocity model for each surveyed boring to a depth of about 145 ft. These interpreted models were compared to borehole stratigraphy and P-S Suspension velocity profiles, and were found to be in good agreement. The down-hole P-wave measurements were within about 10 percent of the P-S Suspension measurements and the down-hole S-wave measurements were within about 20 percent of the P-S Suspension measurements. Considering the site layering (limestone and shale) and signal to noise ratio, this agreement between independent methods was considered reasonable. The down-hole data therefore provide an important independent verification of the seismic velocity profiles developed by the P-S Suspension surveys. Detailed descriptions of the down-hole seismic velocity survey and results are provided in the Borehole Geophysical Logging Report.

2.5.4.2.1.7.4 Acoustical Televiewer

Acoustic televiewer logging was conducted in 30 boreholes to obtain in situ images of borehole walls, obtain deviation data, and measure geometric orientations (strike and dip) of bedding and rock mass discontinuities. The acoustical images are especially useful in evaluating the possible presence of dissolution features in limestone beds, evaluating the tightness of bedding planes, comparing in situ discontinuity density against recovered core fractures (differentiate mechanical fracturing), and evaluating areas of no or poor core recovery. These logs were also used to compare lithological characteristics noted from the core logs to refine stratigraphic contacts (see engineering layers discussed in Subsection 2.5.4.3) and compare depth control.

Table 2.5.4-207 lists borings that were imaged with the acoustical televiewer. The acoustical televiewer surveys were performed by Geovision Inc., using a Robertson Geologging Ltd. High Resolution Acoustic Televiewer (HiRAT) probe. This probe is 7.58 ft long and 1.9 inches in diameter, and is fitted with upper and lower four-band centralizers. This system produces images of the boring wall based upon the amplitude and travel time of an ultrasonic beam reflected from the formation wall. The strength of the reflected signal depends primarily upon the impedance contrast of the boring fluid and the boring wall formation. The probe contains a fluxgate magnetometer to monitor magnetic north and strike/dip orientations of geologic features, and a three-axis accelerometer to measure borehole verticality. A Robertson Geologging Ltd. Micrologger II collected data acquired by the HiRAT and stored them on a hard disk for processing. Borehole data sets are compiled to develop images of the borehole wall.

Acoustical logs included in borehole summary plots (e.g., Figure 2.5.4-206), were reviewed and compared to stratigraphic logging and other geophysical borehole data.

Acoustical images clearly show distinct interbedded limestone and shale strata that match the stratigraphic profiles developed by examination of recovered core and velocity variations indicated by the P-S suspension velocity surveys. The

images confirm that the in situ rock mass and bedding planes are tight without significant open joints, voids, or dissolution features. The dominant in situ rock mass structure is bedding, which is nearly horizontal. Few natural joints are observed in the borehole walls, supporting the geologist interpretations that most fractures in recovered core are mechanical breaks, and RQD is high. Borehole deviations measured by the acoustical televiewer show that the borings typically are quite vertical. Detailed descriptions of Acoustical survey, results and images are provided in the Borehole Geophysical Logging Report.

2.5.4.2.1.7.5 Caliper, Natural Gamma, Resistivity and Spontaneous Potential

Composite borehole index surveys were performed by Geovision Inc. in select core borings using the following suite of test methods: caliper, natural gamma, resistivity, and spontaneous potential. These techniques obtain common index parameters to evaluate general borehole quality, and help define stratigraphy. The combination of gamma, resistivity, and spontaneous potential measurements can provide unique “fingerprint” signatures for different stratigraphic layers that are useful to trace stratigraphy between borings, and refine vertical borehole profiles. Table 2.5.4-207 indicates the core borings that were imaged by the combined geophysical index surveys.

The surveys were developed following guidance in ASTM D5753 and performed in un-cased or partially cased borings filled with bentonite or polymer based drilling mud. Combined geophysical index surveys were performed in a bottom-up sequence. The probes were lowered to the bottom of the boring, where the caliper legs were opened, or probe activated, and data collection was initiated. The probes were then returned to the surface at a rate of 10 ft/min, collecting data continuously at 0.05-ft intervals.

Caliper data were collected using a Robertson Geologging, Ltd. Model 3ACS three-leg caliper probe following ASTM D6167 procedure. The probe was capable of measuring boring diameters ranging between 1.6 in and 16 in. Continuous caliper measurements are presented as profiles of diameter versus depth.

Natural gamma, resistivity, and spontaneous potential data were collected by Geovision Inc. using the same Robertson Geologging, Ltd. Combination ELXG electric log probe. This probe measures Single Point Resistance (SPR), short normal (16 in) resistivity, long normal (64 in) resistivity, Spontaneous Potential (SP) and natural gamma. Probe signals are collected by a Robertson Geologging, Ltd. Micrologger II data collector.

The resistivity measurements were obtained by driving an alternating current into the formation from a probe electrode. The current returned via the logging cable. To ensure adequate penetration of the formation, the logging cable was insulated for approximately 30 ft from the cablehead. Voltages were measured between the 16 in and 64 in electrodes and a remote earth connection at surface, as noted below:

- SPR: The current flowing to the cable was measured along with the voltage at the SPR electrode. The voltage divided by current gave the resistance.

- SP: This was the DC bias of the 16 in electrode with respect to the voltage return at the surface (ground stake).

Natural gamma surveys measured minute amounts of gamma radiation emitted by the geologic formation, and was a qualitative measurement useful for picking transitions between lithologic layers. Gamma logging followed the procedure described in ASTM D6274, Conducting Borehole Geophysical Logging - Gamma. Prior to logging, a field calibration test was performed by applying fixed resistance values across the probe electrodes, as well as a 100 millivolt signal across the SP electrodes, and recording the resultant output of the system.

Digital data were reviewed by the geophysicist in the field, and plotted onto vertical log forms. Caliper surveys showed that the diameter of borehole walls generally did not vary significantly, suggesting a relatively competent and tight rock mass. Some zones of erosion were identified where drilling fill eroded into the formation rock around the drill bit, especially in shaley intervals and zones of weakly cemented sandstone (e.g., upper sandstones in the Twin Mountain Formation). No caverns, “blow out” zones, or large solution cavities were identified by the caliper surveys.

The resistivity and SP measurements in the borehole wall differentiate geologic strata on the basis on differences in resistance and DC bias induced by different mineralogies, cementation, and pore water chemistry. In general, shaley strata produce distinctive peaks in the resistance and SP plots, and higher gamma radiation. The geophysical index profiles show general trends that are consistent with the engineering stratigraphy established by rock core classification, and interpreted seismic velocity layers interpreted from the P-S suspension logging (e.g., Figure 2.5.4-206). Detailed descriptions of Caliper, Natural Gamma, Resistivity and SP surveys, results, data tables and log sheets are provided in the Borehole Geophysical Logging Report.

2.5.4.2.2 Laboratory Testing

Laboratory tests were performed on disturbed soil samples, and both disturbed and relatively undisturbed rock cores obtained during the field exploration program. Tests were performed in general accordance with ASTM or other applicable standards. Laboratory tests performed are listed below:

- Moisture content and density

- Atterberg limits

- Particle-size distribution

- Specific gravity
- Organic content
- Carbonate content
- Slake durability
- Petrographic examination
- X-Ray diffraction
- Consolidated-drained direct shear
- Consolidated-undrained triaxial compression with pore pressure measurements
- Consolidated-undrained triaxial compression without pore pressure measurements
- Unconsolidated-undrained triaxial compression
- Unconfined compression of rock
- Point load strength index of rock
- Laboratory shear wave velocity
- One-dimensional swell or settlement potential
- One-dimensional consolidation

The Glen Rose Formation within the plant foundation influence zones (upper 200 ft) was specifically targeted for geotechnical laboratory testing. Laboratory test results are generally reported by specific depth, boring, and engineering layer on laboratory test sheets, summary tables, and plots. Detailed descriptions of laboratory tests, procedures, and results are provided in the Laboratory Test Data Report.

2.5.4.2.2.1 Sample Control

During the field exploration program, a secured warehouse building within the CPNPP site was selected and designated as the controlled access storage facility for the soil and rock samples. Samples from each drill hole were transported daily from the field to the sample storage facility by the rig geologists and stored in designated areas. SPT samples were placed in glass jars and sealed using a moisture-tight lid. CME continuous soil samples were placed in either wooden core boxes or heavy-waxed cardboard boxes. Undisturbed tube samples were

sealed on both ends in the field using plastic packers overlaid with wax, covered with plastic caps, and sealed with duct tape. Rock core samples were placed in wooden core boxes equipped with PVC trays. Selected pieces of rock core specimens for laboratory testing were wrapped in plastic and secured in glass jars that were sealed using moisture-tight lids. All samples in jars and boxes were labeled with proper identification at the time of collection, and labels were copied into the sample inventory records upon arrival in the storage area.

Field boring logs and records were reviewed by the project geologists and engineers, and samples were identified and prepared for possible laboratory testing. These samples were reviewed by senior and principal geologists and engineers, and specific specimens were identified for laboratory testing. The transport and receipt of samples from field storage to the Fugro Laboratory in Houston, Texas is documented by Chain-of-Custody forms, and performed according to QA program Work Instructions. All sample handling and transportation on-site and to the laboratory was carried out in accordance with ASTM D4220.

Upon arrival at the laboratory, samples were assigned to labeled boxes and stored in secured, climate-controlled storage rooms. Samples were always kept wrapped in bubble wrap and/or tin foil within the climate controlled storage rooms in the laboratory facility to preserve original moisture content. After test assignments, specific samples were examined and retrieved for testing. Appropriate portions of the samples were taken to complete the assigned tests. After sample examination or testing, the unused portions were properly re-sealed, placed back into their individual jars, and stored in the laboratory climate-controlled storage rooms.

A subset of samples was selected for testing at other facilities (Spectrum Petrographics, University of Washington, and University of Texas at Austin) for non-safety-related applications. These samples were placed in labeled sample jars with moisture-tight lids and shipped under chain of custody to the designated testing laboratory.

2.5.4.2.2.2 Laboratory Testing Procedures

2.5.4.2.2.2.1 Moisture Content and Density

Moisture content and density were determined in general accordance with ASTM D2216 and D2166 Section 6. A test specimen was dried in an oven at a temperature of 110° ± 5°C to a constant mass. The loss of mass was considered water mass, and moisture content was calculated based on water mass and the mass of the dried specimen. The density was determined by means of the direct measurement of the dimensions and mass of the specimen. Moisture content, total, and dry unit weight test data are plotted versus sample elevations on Figures 2.5.4-218, 2.5.4-219, and 2.5.4-220, respectively, and summarized in Table 2.5.4-210. The distribution of tested samples focused on the Glen Rose and Twin Mountain formations throughout the plant foundation influence zone, and

included sufficient sampling of each discrete engineering layer (Subsection 2.5.4.3) to evaluate potential lateral and vertical variability in properties.

2.5.4.2.2.2.2 Atterberg Limits

Liquid and plastic limits of selected specimens were determined in accordance with ASTM D4318. The samples were prepared by removing any material retained on No. 40 Sieve (425 mm). For shale, samples of approximately 150 g to 250 g were selected and pulverized through a grinder into powder form. Then water was added to the sample up to about the liquid limit point. The wet sample was pushed through a No. 40 Sieve, bagged, and left overnight to slake before being tested. The liquid limit was determined by ASTM D4318 Method A. Atterberg limits test results are summarized in Table 2.5.4-209, and the plasticity chart is shown on Figure 2.5.4-222. Atterberg limits help refine the classification of a soil, and are useful indices related to mechanical properties of fine-grained soil and rock material.

2.5.4.2.2.2.3 Particle-Size Distribution

Particle-size distribution (gradation) of soils using sieve analysis and hydrometer technique was performed in accordance with ASTM D6913 and D422. The grain-size distribution of coarse-grained soils (particle sizes larger than 75 mm) is determined directly by sieve analysis, while the distribution of fine-grained soils (particle sizes smaller than 75 mm) is determined indirectly using hydrometer analysis. The grain-size distribution of mixed soils is determined by combined sieve and hydrometer analyses. Sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve. The hydrometer analysis is based on Stokes' law, which relates the terminal velocity of a sphere falling freely through a fluid to its diameter. The hydrometer is used to determine the percentages of clay and silt in the sample based on precipitation rates through a water column. The grain-size distribution of a sample aids in the engineering classification, and is useful for correlation of typical engineering properties. Particle-size distribution test results are provided in the Laboratory Test Data Report.

2.5.4.2.2.2.4 Specific Gravity

The specific gravity of specimens was determined in accordance with ASTM D854. This test method determines the specific gravity of soil solids that are finer than 4.75 mm (No. 4 sieve), by means of a water pycnometer. The specific gravity of solids (soil or rock) is the ratio of the weight in air of a given volume of sample at a stated temperature to the weight in air of an equal volume of distilled water at the same temperature. Specific gravity test results are plotted versus sample elevation on Figure 2.5.4-221 and summarized in Table 2.5.4-209. Rock sample porosities were calculated based on the specific gravity data and dry unit weight, and are plotted versus elevation on Figure 2.5.4-221. Porosity estimates are used to help evaluate permeability and potential interconnections of small dissolution features.

2.5.4.2.2.2.5 Organic Content

The organic content of specimens was determined in general accordance with ASTM D2974, with the exception that the moisture content was determined by using ASTM D2216. Organic Content test results are provided in the Laboratory Test Data Report.

2.5.4.2.2.2.6 Carbonate Content

The carbonate content of specimens was determined in accordance with ASTM D4373. The carbonate content (calcite equivalent) of soil is determined by treating a 1 g dried soil specimen with hydrochloric acid in an enclosed reaction cylinder. The pressure of carbon dioxide (CO_2) gas generated by the reaction between the acid and carbonate fraction of the specimen is used to determine the calcite equivalent of the specimen. Results are in terms of calcite equivalent as a percentage because different carbonate species cover a wide range of percent calcite equivalent. For example, 100 percent dolomite would be expected to yield a 108.6 percent calcite equivalent. Table 2.5.4-212 presents a summary of the carbonate content test results. Carbonate content is a useful index to evaluate the potential for dissolution and karstic development in limestone or limey rock sequences. Typically, well-developed dissolution and karst requires high carbonate content, generally about 90 to 95 percent or greater carbonate purity. Carbonate content test results are considered in the evaluation of dissolution and karst potential in Subsections 2.5.1 and 2.5.4.1.

2.5.4.2.2.2.7 Slake Durability

Slake durability tests were conducted in accordance with ASTM D4644. Slake durability index is the percentage of dry mass retained for a collection of shale pieces on a 2.00 mm (No. 10) sieve after two cycles of oven drying and 10 minutes of soaking in water with a standard tumbling and abrasion action. Slake durability test results are summarized on Table 2.5.4-211. Slake durability is used to evaluate the potential for degradation of shale or claystone rock layers upon exposure to the atmosphere (e.g., excavation faces) both under temporary construction and long-term conditions. These test results are considered in the evaluation of the performance of graded slopes, excavation walls, and foundation subgrades, as discussed in Subsection 2.5.4.5.

2.5.4.2.2.2.8 Petrographic Examination

Petrographic and photomicrographic analyses were performed by Spectrum Petrographics Inc. of Vancouver, Washington. These tests are typically performed on a standard thin section stained for K-feldspar plus combined carbonates, and had a permanent coverglass. Polished thin sections are used when opaque mineral analysis is required. Petrographic analysis includes: 1) rock name and interpreted protolith; 2) visually estimated modal mineralogy; 3) primary textures and structures; 4) secondary textures and structures; and 5) relative timing of formational events with emphasis on alteration and paragenesis. Standard Digital

Photomicrography includes: 1) a 3X macro photo of the chip's polished surface; and 2) one or more photomicrographs at 28X to 568X to document typical appearance and important features. Petrographic examination results are summarized in Table 2.5.4-213. Petrographic analyses are used for basic classification of rock layers, and evaluation of potential variability of mineralogy within discrete rock strata. Petrographic classification also is a useful index for comparison of basic rock properties and identification of possible adverse mineralogies, as described in Subsections 2.5.1 and 2.5.4.1.

2.5.4.2.2.2.9 X-Ray Diffraction

X-ray diffraction analyses were performed by Portland State University for clay-size fraction and bulk samples. The clay-size fraction was mounted in an oriented way onto a glass slide. This glass slide was X-rayed using a Philips Expert PW3040 theta-theta diffractometer. The X-ray diffraction patterns were analyzed both in air-dried and glycol-solvated and heated samples to evaluate changes in crystal structure ("d-spacing" and "peak intensity"). The results were used to identify the clay mineralogy and percent clay mineral. For bulk samples, whole rock specimens were ground to form a derivative sample powder smaller than 63 micrometers and packed into a random powder sample holder. The random powder was X-rayed using a Philips Expert PW3040 theta-theta diffractometer. The minerals were identified based on the d-spacing of the random powder pattern. Results of the X-Ray Diffraction analyses are summarized in Table 2.5.4-214. The X-Ray Diffraction results were used to identify potentially adverse mineralogy, and to help refine mineralogical and petrologic classifications.

2.5.4.2.2.2.10 Consolidated-Drained Direct Shear

Direct shear tests were conducted in accordance with ASTM D3080 and ASTM D5607. Samples were treated as soils when practical, in accordance with ASTM D3080. For samples that did not fit securely into the ring, grouting was used to help secure the samples, and the resulting samples were tested according to ASTM D5607 for rock specimens. Three tests were conducted on each sample under varying normal loads to develop a shear strength envelope. When possible, different specimens of the same material were used for each pre-consolidation pressure. If three similar, undisturbed specimens were not available, the tests were conducted as "staged" tests on a single specimen, resulting in fully-softened (ultimate) or near-residual shear strength values at the three consolidation pressures. Direct shear strength tests for shale specimens, which were tested horizontally (along bedding), are presented in Table 2.5.4-221 and graphically summarized on Figure 2.5.4-235. The results from direct shear tests were used to help evaluate the shear strength of the shale material in the rock mass.

2.5.4.2.2.2.11 Consolidated-Undrained Triaxial Compression with Pore Pressure Measurements

Consolidated-Undrained (CU) triaxial compression tests with pore water pressure measurement were performed in accordance with ASTM D4767 on relatively

undisturbed shale and lightly cemented sandstone specimens derived from intact rock core intervals. These samples were carefully extracted from the recovered core, wrapped in plastic, and placed in sealed jars to maintain moisture content. In the laboratory, the specimens were encased in rubber membranes and fully saturated by applying backpressure prior to the consolidation stage. Pore water in the samples was permitted to drain during the isotropic consolidation stage, but no drainage was allowed during the axial loading phase. Failure is assumed to occur when the specimens reach the maximum deviator stress, or an axial strain of 15 percent, whichever develops first. Axial load, axial displacement, chamber pressure, and excess hydrostatic pressure generated during the shearing phase are measured and recorded. The results provide total stress strength parameters if pore pressure is neglected. Effective stress strength parameters are obtained by subtracting measured pore water pressures from the total stress results. Results of the CU triaxial tests are summarized in Table 2.5.4-215, and ultimate shear strength values for shale specimens are graphically presented on Figure 2.5.4-235. These tests are performed on vertically oriented samples consistent with the orientation of the samples in situ. Therefore, they provide across-bedding shear strength of sedimentary rock specimens. The results from these tests are used to estimate in situ shear strength under certain loading conditions for slope stability and foundation analyses presented in Subsections 2.5.5 and 2.5.4.10.

2.5.4.2.2.2.12 Consolidated-Undrained Triaxial Compression without Pore Pressure Measurements

Isotropically Consolidated-Undrained (CU) triaxial compression tests without pore water pressure measurement were performed in accordance with ASTM D7012 Method A and ASTM D4767 on relatively undisturbed shale, limestone, and sandstone specimens collected and prepared as discussed above in Subsection 2.5.4.2.2.2.11. Specimens are trimmed in accordance with ASTM D4543 for rock core specimens. Some very soft rock specimens (e.g., soft shale, weakly cemented sandstone) are trimmed using soil-trimming techniques. The specimens are encased in rubber membrane and permitted to drain during the consolidation phase to permit consolidation under the confining pressure. No drainage is allowed during the shearing phase. Failure is assumed to occur when the specimens reach the maximum deviator stress, or an axial strain of 15 percent, whichever occurs first. Axial load, axial displacement, and chamber pressure during the shearing phase are measured and recorded. Pore pressures are not measured or recorded during shearing. The results are used to evaluate the total stress strength parameters of weak rocks, such as shale. Results of the CU triaxial tests without pore water pressure measurement for both the peak and ultimate shear strength are summarized in Table 2.5.4-216 and Table 2.5.4-220. Results are also graphically presented in a Mohr-Coulomb format on Figures 2.5.4-232, 2.5.4-233, and 2.5.4-234 for limestone peak strength, shale peak strength, and shale ultimate strength, respectively. The results from these tests are used to estimate in situ shear strength under certain loading conditions for slope stability and foundation analyses, presented in Subsections 2.5.5 and 2.5.4.10.

2.5.4.2.2.2.13 Unconsolidated-Undrained Triaxial Compression

Unconsolidated-Undrained (UU) triaxial compression tests were performed in accordance with ASTM D7012 Method A on relatively undisturbed shale, limestone, and sandstone specimens. UU triaxial compression tests were performed in a manner similar to CU triaxial compression tests without pore water pressure measurements, except that no drainage and consolidation were permitted to occur during the application of confining pressure. Results of the UU triaxial tests are summarized in Table 2.5.4-217. The graphical representation of results is also included in the same plots for CU tests (Figure 2.5.4-232 for peak strength of limestone, and Figures 2.5.4-233 and 2.5.4-234 for peak and ultimate strengths of shale, respectively). The results from these tests are used to estimate in situ shear strength under certain loading conditions for slope stability and foundation analyses, presented in Subsections 2.5.5 and 2.5.4.10.

2.5.4.2.2.2.14 Unconfined Compression of Rock

Unconfined Compression (UC) tests were performed in accordance with ASTM D7012 Methods C or D on relatively undisturbed and intact core samples of shale, limestone, and sandstone. In Method C, vertical displacement measurements are obtained based on the movement of the bottom platen during testing. These measurements incorporate movements related to equipment flex, with the result that strain cannot be used for modulus determination without correction. In Method D, the axial deformations are measured using a Linear Variable Displacement Transducer (LVDT)-Jacket Device (attach and support system) along with the platen movement, while the lateral deformations are determined using a "Chain-LVDT" Device (change in circumference system). This procedure permits obtaining estimates of materials modulus. Poisson’s ratio of the specimens is obtained by measuring both axial and radial deformations during testing. Results of UC tests are summarized in Table 2.5.4-218 and Table 2.5.4-220, and are plotted versus elevations on Figures 2.5.4-226 and 2.5.4-227. The UC tests provide basic strength measurements of intact rock, and are useful strength properties for mechanical behavior of rock strata that are relatively competent and do not undergo significant changes under excavation procedures. The UC test results also are used as input for the Hoek-Brown criterion for evaluation of in situ rock mass strength and deformation characteristics.

ASTM D7012 procedure requires that all specimens contain a Length (L) to Diameter (D) ratio of at least 2. However, there are published data (Reference 2.5-423) that allow one to correct strength data from tests in which L/D ratio is less than 2. As indicated on Table 2.5.4-226, there were several samples that contained L/D ratios ranging between 1.5 and 2. The procedure in Reference 2.5-423 was used to estimate the correction factors and evaluate the results of the samples with L/D less than 2.

2.5.4.2.2.2.15 Point Load Strength Index of Rock

Point Load Strength Index (PLI) tests were conducted in accordance with ASTM D5731. These tests are performed using GEOTAC equipment configured to take readings close together in order to estimate the peak load. Diametral and axial PLI tests can be rapidly performed on relatively undisturbed core specimens of sandstone and limestone, or core fragments, and are typically indexed to UC tests on the same or similar adjacent rock core specimens to develop a site-specific correlation. Results from PLI tests are summarized in Table 2.5.4-219 and shown graphically versus depth on Figure 2.5.4-228. Correlations between PLI test results and UC test results are plotted versus elevation on Figure 2.5.4-229. The PLI tests provide additional index and correlation measurements of rock core to help assess vertical and lateral variability in mechanical properties throughout the site.

2.5.4.2.2.2.16 Laboratory-Based Shear Wave Velocity

Laboratory measurements of shear wave (S-wave) velocity on relatively undisturbed specimens of shale, sandstone, and limestone specimens were conducted in accordance with ASTM D2845. The velocities are measured in terms of travel time and the distance waves traveled through the rock specimen. A summary of test sample locations and measured S-wave velocity values is provided in Table 2.5.4-224, and a summary plot versus elevation is presented on Figure 2.5.4-238. The laboratory-measured S-wave velocities are representative of intact, limited-length core specimens, and do not account for scale effects or rock mass features (e.g., joints and bedding planes) that typically result in lower in situ velocities.

These laboratory measurements are not used to develop the site velocity profile. The site velocity profile is based on in situ geophysical measurements described in Subsections 2.5.4.3 and 2.5.4.4. The laboratory measurements do provide a useful index property to evaluate possible variability inherent in intact rock cores, and to evaluate the degree of disturbance in a laboratory test sample.

2.5.4.2.2.2.17 One-Dimensional Swell or Settlement Potential

Swell or settlement potential tests were conducted in accordance with ASTM D4546 Method C on relatively undisturbed, intact core intervals of shale. During testing, vertical load increments are applied to keep the sample from swelling after the specimen is inundated in water. A consolidation test is subsequently performed on the saturated sample, in accordance with ASTM D2435. A summary of swell test sample locations and results is presented in Table 2.5.4-223. Comparisons between the swell and consolidation measurements provide a reference to evaluate potential stress-relief (rebound) and/or saturation-induced swell in shale strata, and to estimate post-construction settlements, discussed in Subsections 2.5.4.5 and 2.5.4.10.

2.5.4.2.2.2.18 One-Dimensional Consolidation

One-dimensional consolidation tests were performed in accordance with ASTM D2435 Method B on relatively undisturbed core specimens of shale. Consolidation is the process of gradual transfer of an applied pressure from the pore water to the soil structure as pore water is squeezed out of the voids. Low-density or highly porous shale strata may exhibit a potential for consolidation settlement, commonly expressed as a re-consolidation upon applied foundation loading, following some level of stress-relief rebound in the floor of an excavation. For the consolidation test, a laterally confined specimen is subjected to successively increasing vertical pressure, allowing for free drainage from both the top and bottom surfaces. The samples are inundated shortly after application of seating pressure and loads are applied to contain the swelling. A summary of the test sample locations and results is provided in Table 2.5.4-222. The results from the one-dimensional consolidation tests are used to evaluate potential settlement of weak shale beds, discussed in Subsection 2.5.4.10.

2.5.4.2.2.2.19 Rock Specimen Preparation

All rock core specimens were prepared in accordance with ASTM D4543. This standard outlines the procedure and methods for laboratory specimen preparation and determination of the length and diameter of rock core specimens and the conformance of the dimensions with established standards. Because the dimension, shape, and surface tolerances of rock core specimens are important in determining rock properties of intact specimens, great care is exercised when preparing core samples for strength testing. The prepared cores are measured to determine the straightness of the specimen’s cylindrical side, flatness of its ends, parallelism of the end platens, and perpendicularity of end surfaces to the specimen axis.

2.5.4.2.3 Material Properties

As described in Subsection 2.5.4.3, the CPNPP Units 3 and 4 site is underlain by shallow bedrock comprised of the following main geologic formations, in order of increasing depth: the late Cretaceous Glen Rose Formation limestone and shale (engineering Layers A through F) to an approximate elevation of 620 ft; the late Cretaceous Twin Mountain Formation sandstone, shale, and limestone (engineering Layers G through I) between approximately elevations 620 ft and 390 ft; and the Upper Paleozoic Mineral Wells Formation indurated shale and sandstone below elevation 390 ft. Based on the dimensions, loads, and embedment depths of the seismic category I and II structures, the main zone of foundation influence occurs within Glen Rose Formation engineering Layer C, which consists primarily of competent, massive limestone at and below foundation subgrade elevation 782 ft.

Laboratory testing included multiple samples of each engineering layer and comprised a complete section through the three main geologic formations. Limited test results are also provided for surficial residual soil and localized

undocumented fill that mantle the bedrock, but are stripped away from seismic category I and II structure footprints during mass grading to provide a complete characterization of materials occurring at the site.

Laboratory test samples and results were correlated with specific site geological layers and engineering stratigraphy that are described in Subsection 2.5.4.3 and summarized on Figures 2.5.4-204 and 2.5.4-205. For the purposes of general discussion, the Glen Rose Formation and Twin Mountain Formation limestone and shale layers were lumped together as "limestone" or "shale," respectively, and the Twin Mountain Formation sandstone strata were lumped together as "sandstone." The in situ characteristics and laboratory test results for each primary rock type (e.g., limestone) fall within a relatively tight range, irrespective of specific engineering layer, and thus are assigned the general description rather than specific engineering layer of derivation for simplification. However, laboratory test results are reported by specific depth, boring, and engineering layer on laboratory test sheets and summary tables and plots (Subsection 2.5.4.2.2).

2.5.4.2.3.1 Index Properties

2.5.4.2.3.1.1 Moisture Content

Typical ranges of measured water content from selected rock samples are summarized below, and are presented in Table 2.5.4-209 and on Figure 2.5.4-218.

Limestone: Water contents measured from limestone samples ranged from about 1 to 19 percent, with an average value of about 5 percent. Low moisture contents in the limestone reflect a low porosity and dense condition.

Shale: Water contents measured from shale samples ranged from about 6 to 23 percent. The average measured water content for shale was about 15 percent.

Interbedded Limestone and Shale: Samples that exhibited characteristics of limestone and shale (e.g., shaley limestone, calcareous shale), or that occurred along stratigraphic contacts between limestone and shale strata, ranged from about 9 to16 percent, with an average value of about 12 percent. These moisture contents were intermediate between the values for relatively pure limestone and shale, as expected.

Sandstone: Water contents measured from sandstone samples ranged from about 8 to 19 percent. The average measured water content for sandstone was about 13 percent.

2.5.4.2.3.1.2 Unit Weight

The measured total and dry unit weights of limestone, shale, and sandstone samples are summarized below, and are presented in Table 2.5.4-209 and plotted versus elevation on Figures 2.5.4-219 and 2.5.4-220.

Limestone: Measured dry unit weights of limestone samples ranged from approximately 119 pounds per cubic foot (pcf) to 160 pcf, with an average value of about 149 pcf. Measured total unit weights of limestone samples ranged from 136 pcf to 165 pcf, with an average value of about 156 pcf. These results indicate that the limestone is relatively massive and dense, with dry unit weights in the typical range for concrete. As a result, the limestone-concrete interface at the base of foundations will not represent a significant density contrast.

Shale: Measured dry unit weights of shale samples ranged from 99 pcf to 147 pcf, with an average value of about 120 pcf. Measured total unit weights of shale samples ranged from 118 pcf to 169 pcf, with an average value of about 137 pcf. These results show that the shale is relatively dense and indurated in situ.

Interbedded Limestone and Shale: Dry unit weights of interbedded limestone and shale samples ranged between 114 pcf and 137 pcf, with an average value of 127 pcf. Measured total unit weights ranged from 132 pcf to 154 pcf, with an average value of about 142 pcf. Similar to the discussion above of moisture content, the unit weights of interbedded limestone and shale samples were intermediate between those of a relatively pure limestone and shale, as was expected.

Sandstone: Measured dry unit weights of sandstone samples ranged from 105 pcf to 134 pcf, with an average value of about 121 pcf. Measured total unit weights of sandstone samples ranged from 124 pcf to 147 pcf, with an average value of about 136 pcf.

2.5.4.2.3.1.3 Porosity

Typical ranges of calculated porosity values for soil, limestone, shale, and sandstone specimens can be summarized as follows, and are presented in Table 2.5.4-209 and plotted versus elevation on Figure 2.5.4-221.

Limestone: Estimated porosity values of limestone samples ranged from 5 to 30 percent, with an average of about 12 percent.

Shale: The porosity of shale samples was estimated to range from 13 to 42 percent, with an average value of 30 percent.

Interbedded Limestone and Shale: Porosities for interbedded limestone and shale samples were estimated to range from 19 to 33 percent. The average porosity for interbedded rock samples was about 26 percent.

Sandstone: Estimated porosity of sandstone samples ranged from 19 to 37 percent, with an average value of 27 percent.

2.5.4.2.3.1.4 Atterberg Limits

Atterberg limits tests were performed on samples of residual clayey soils and shale layers within the Glen Rose Formation. Test results are summarized below, and are presented in Table 2.5.4-209 and on Figure 2.5.4-222.

Soil Samples: Liquid limits (LL) of tested soil samples ranged between 25 and 60 percent, with an average value of 36 percent. The plasticity index (PI) ranged between 9 and 43 percent, with an average value of about 21 percent. The results of the Atterberg limits tests on soil samples indicated predominately Lean Clay (CL), with only three samples classified as Fat Clay (CH).

Shale: LL tests from shale samples ranged from 27 to 71 percent, with an average value of 46 percent. The PI within these samples ranged from 14 to 48, with an average value of 28. The results of the Atterberg Limits tests on shale samples indicate mainly Lean Clay (CL), with some samples classified as Fat Clay (CH).

2.5.4.2.3.1.5 Specific Gravity

Specific gravity tests were performed on selected samples of limestone, shale, and sandstone. Specific gravity values are summarized as follows and are presented in Table 2.5.4-209 and plotted versus elevation on Figure 2.5.4-223.

Limestone: The specific gravities measured from limestone samples varied between 2.69 and 2.72, with an average value of about 2.71.

Shale: The specific gravities measured from shale samples ranged from 2.74 to 2.78, with an average value of about 2.76.

Sandstone: The one specific gravity test performed on a sandstone sample had a measured value of 2.65.

Specific gravity for tested rock materials are within typical published ranges for calcite and dolomite (2.7 to 2.8), which are primary components of the limestone, and quartz (2.65), which is the primary component of the sandstone (Reference 2.5-416).

2.5.4.2.3.1.6 Slake Durability

Slake durability tests were performed on selected samples of limestone and shale. The test results are summarized below, and are presented in Table 2.5.4-211.

Limestone: Slake durability indices of selected limestone samples ranged from 91 to 98 percent, with an average value of 95 percent. According to the Slake Durability Classification of Gamble (Reference 2.5-408), the measured slake durability values indicate that the limestone samples are medium to highly durable.

Shale: Slake durability indices of selected shale samples range from 0.2 to 83 percent, with an average value of about 50 percent. Based on the Slake Durability Classification of Gamble (Reference 2.5-408), measured slake durability values indicate that the shale samples have low durability. These results suggest that shale strata potentially are subject to slaking and raveling when confining pressure is removed (e.g., in excavated slopes).

2.5.4.2.3.2 Static Properties

Static strength and deformation properties of the subsurface materials were determined in the laboratory by means of the following types of engineering property tests:

- Unconfined compression tests
- Triaxial compression tests
- Point load strength index tests
- Direct shear strength tests
- One-dimensional consolidation tests
- One-dimensional swell tests

2.5.4.2.3.2.1 Unconfined Compressive Strength

UC tests were performed on selected samples of limestone, shale, and sandstone between elevations 830 ft to 660 ft. Test results are presented in Table 2.5.4-218, and are plotted versus elevation on Figure 2.5.4-226 (UC tests) and Figure 2.5.4-227 (all tests). The results are summarized below.

Limestone: UC strengths measured from limestone samples ranged from about 73 tons per square foot (tsf) to 812 tsf, with an average value of about 299 tsf. The Tangent Modulus at 50 percent of the failure load (q_u) ranged from about 310 kips per square inch (ksi) to 7,300 ksi, with an average value of about 1,900 ksi. This rock is characterized as weak to moderately strong (Reference 2.5-416). UC test results on limestone are used as an input parameter along with other rock-mass parameters (e.g., GSI [Geologic Strength Index], mi [Hoek-Brown constant]) for the Hoek and Brown criterion to estimate the in situ strength of the limestone rock mass (References 2.5-409, 2.5-410, and 2.5-411). The range of rock-mass shear strength results interpreted from the UC test data is shown on Figure 2.5.4-237.

Shale: UC strengths measured from shale samples ranged from 13 tsf to 104 tsf, with an average compressive strength of about 75 tsf. The Tangent Modulus at 50 percent of failure load (q_u) ranged from about 5 ksi to 310 ksi, with an average value of about 190 ksi. UC tests results classify the shale as a very weak to weak rock (Reference 2.5-416).

Sandstone: Only one UC strength test was performed on a sample classified as weakly cemented sandstone. The measured unconfined compressive strength value was 10 tsf, correlating with very weak rock. The weakly cemented sandstone occurs at depths far below the foundation bottom elevations, and only occasionally occurs within otherwise massive and hard sandstone. The weakly cemented sandstone was observed at only a few locations within lenses that were normally less than 2 ft thick.

The UC test results from the limestone samples were used to estimate the range of rock-mass shear strength values.

2.5.4.2.3.2.2 Shear Strength from Triaxial Consolidated-Undrained Tests

CU triaxial compression strength tests were performed on selected limestone, shale, and sandstone samples ranging in elevation from about 820 ft to 500 ft. Test results are presented in Tables 2.5.4-215 and 2.5.4-216, with and without pore water pressure measurements, respectively. The test results are summarized below.

Limestone: CU triaxial strength tests performed on selected limestone samples indicate compressive strength ranging from 127 tsf to 587 tsf, with an average value of about 361 tsf.

Shale: CU triaxial strength tests performed on selected shale samples resulted in compressive strengths ranging from about 10 tsf to 82 tsf, with an average value of about 30 tsf.

To interpret the data, a series of p-q plots (Reference 2.5-412) that display q ($[\sigma_1-\sigma_3]/2$) versus p ($[\sigma_1+\sigma_3]/2$) were prepared. Separate plots were developed for limestone and shale samples. Mohr-Coulomb shear strength envelopes were derived from the p-q plots by constructing a best-fit K_f-line through the p-q points on each plot (Figures 2.5.4-232, 2.5.4-233, and 2.5.4-234).

2.5.4.2.3.2.3 Shear Strength from Unconsolidated-Undrained Tests

UU triaxial strength tests were performed on 11 rock samples ranging in elevation from about 824 ft to 570 ft. Test results are presented in Table 2.5.4-218 and summarized below.

Limestone: UU triaxial tests performed on selected limestone samples resulted in compressive strengths ranging between 204 tsf and 498 tsf, with an average value of about 362 tsf.

Shale: UU triaxial tests performed on shale samples resulted in compressive strengths ranging from about 4 tsf to 41 tsf, with an average value of about 20 tsf.

Sandstone: Only one UU triaxial test was performed on a weakly cemented sandstone sample, and it resulted in a measured compressive strength value of 50 tsf.

Results of the UU triaxial tests are also presented as p-q plots similar to CU triaxial tests and on the same plots (Figures 2.5.4-232, 2.5.4-233, and 2.5.4-234).

2.5.4.2.3.2.4 Unconfined Compressive Strength from Point Load Index Tests

PLI tests were performed on selected limestone samples from elevations ranging from 830 ft to 750 ft. The point load strength index ($I_{S(50)}$) values obtained from axial and diametral tests were corrected for specimen size and correlated with data obtained from UC strength tests. The PLI test results are presented in Table 2.5.4-219, and are summarized below.

The corrected $I_{S(50)}$ measured from axial PLI tests performed on selected limestone samples ranged from about 1 tsf to 66 tsf, with an average value of about 24 tsf. The correlated PLI compressive strength values ranged from 10 tsf to 742 tsf, with an average value of about 285 tsf.

The $I_{S(50)}$ values obtained from diametral tests performed on the same limestone samples are generally lower, ranging from about 1 tsf to 44 tsf, with an average value of about 15 tsf. The correlated PLI compressive strength values for the diametral tests range from about 14 tsf to 520 tsf, with an average value of about 180 tsf.

2.5.4.2.3.2.5 Shear Strength from Direct Shear Tests

Direct shear tests were generally performed for bedding plane surfaces in selected shale samples in the elevation range from 815 ft to 780 ft. Test results are summarized in Table 2.5.4-221, and are presented on Figure 2.5.4-235. The plot shows the range of the ultimate (fully-softened) drained shear strength values. A shaded, curved envelope that includes most of the shear strength values is also presented on this plot.

For comparison purposes, the procedure of Stark et al. (References 2.5-413 and 2.5-414) was also used to estimate the range of saturated, fully-softened drained shear strength values for shale. In this method, estimates of secant friction angles are made for a selected number of effective normal stress values based on liquid limit and clay fraction index test results. The data points are then connected by a line that passes through the origin to create a curved ultimate (fully-softened) shear strength envelope.

The results of LL and clay fraction index tests obtained for shale samples were used in the Stark et al. method. The upper- and lower-bound ultimate shear strength envelopes estimated by the Stark et al. procedure are shown on Figure

2.5.4-235. The midpoint envelope developed from the Stark et al. correlation and the lower bound of the direct shear test data envelope appears to be comparable.

Figure 2.5.4-236 shows the same ultimate shear strength envelopes along with the fully-softened strength data from the CU triaxial compression tests for shale. Considering the failure mode, the results of the triaxial compression tests more closely resemble the across-bedding failure mode of the shale.

2.5.4.2.3.2.6 Compressibility from One-Dimensional Consolidation Tests

One-dimensional consolidation tests were performed on selected shale samples between elevations 820 ft and 780 ft. Although sample disturbance may have obscured the pre-consolidation pressure and the resulting over-consolidation ratios (OCRs), consolidation test results were used to approximately estimate the OCRs. The estimated OCR values range from about 2 to 4, with an average of about 3. Sample disturbance has apparently occurred to some degree, even for core specimens. Sample disturbance could significantly affect the estimated OCR and compression ratios. One-dimensional consolidation test results are summarized in Table 2.5.4-222.

2.5.4.2.3.2.7 Potential Heave from One-Dimensional Swell Tests

One-dimensional swell tests were performed on nine shale samples. The percent of wetting-induced heave measured from initially unsaturated shale samples ranged from about 0.5 to 2.2 percent for inundation pressure ranging between 2.5 kips per square foot (ksf) and 8.0 ksf, except for one sample that did not show any expansion potential. Estimated swell pressure values appear to range roughly between 11 ksf and 32 ksf. The estimated swell pressures are an order of magnitude higher than the existing overburden pressures and generally indicate a high degree of expansion potential. Based on available correlations between swell potential and the index properties (Reference 2.5-415), the tested samples were also classified as having a medium to high degree of expansion. The results of the one-dimensional swell tests are presented in Table 2.5.4-223.

2.5.4.2.3.2.8 Poisson's Ratio

For selected UC tests, both the axial and lateral strains were measured to allow for estimation of the Poisson’s ratio values. Estimated values for both secant and tangent Poisson’s ratios at 50 percent of the maximum unconfined compression strength values are presented in Table 2.5.4-218 and plotted versus elevation on Figures 2.5.4-230 and 2.5.4-231.

2.5.4.2.3.3 Dynamic Properties

Dynamic properties of soils and rocks include the low-strain S-wave velocity, shear modulus, and damping. Shear modulus can be calculated from S-wave velocity and mass density. The low-strain dynamic properties of the subsurface

materials were primarily determined by field geophysical surveys and down-hole in situ testing conducted during the geotechnical site investigation program.

Borehole geophysical testing was conducted in 15 boreholes across the site. Site representative single values for both Shear Modulus and Damping percentages were developed for each engineering layer considering that the profile will respond linearly from low to high strains based on the measured S-wave velocities from the Suspension P-S borings.

The in situ borehole geophysical surveys are described in Subsections 2.5.4.2.1.7.2 and 2.5.4.2.1.7.3.

S-wave velocity measurements were also performed in the laboratory on selected samples. However, as mentioned previously, the laboratory measurements were only used to evaluate sample competence and not for developing the site velocity profile. Results of these tests are summarized below.

2.5.4.2.3.3.1 Laboratory-Based Shear Wave Velocity

Laboratory-based S-wave velocity measurements were performed on limestone, shale, and sandstone samples between elevations 830 ft and 730 ft. Test results are summarized in Table 2.5.4-224, and are graphically shown versus depth on Figure 2.5.4-238. The results obtained from the laboratory tests are presented below.

Limestone: Measured S-wave velocities from selected limestone samples ranged from 4,100 fps to 14,400 fps, with an average value of about 7,260 fps.

Shale: Measured S-wave velocities from selected shale samples ranged from 1500 fps to 3,900 fps, with an average value of about 2,600 fps.

Sandstone: Measured S-wave velocities from selected sandstone samples ranged from 2,100 fps to 5,800 fps, with an average value of about 3,300 fps.

As discussed in Subsection 2.5.4.2.2.2.16, the results from laboratory S-wave measurements correlate only to intact, small laboratory specimens, and are not representative of the in situ rock mass, that instead is characterized by an extensive array of borehole geophysical surveys.

2.5.4.2.3.4 Mineralogy and Chemical Properties

Rock Core Composition was analyzed by petrographic, x-ray, and chemical analyses of cores of the Glen Rose and Twin Mountain formations. These tests indicate that both the Glen Rose and Twin Mountain formations are not susceptible to solutioning, which verifies the field observations and experience. Limited testing of residual soil indicates that organic content is limited. These results are summarized in Tables 2.5.4-212, 2.5.4-213, and 2.5.4-214.

2.5.4.2.3.4.1 Carbonate Content

Test results for calcium carbonate content of rock are presented in Table 2.5.4-212, and are also described below.

Limestone: The carbonate content measured from selected limestone samples ranged from 56 to 100 percent, with an average calcium carbonate content of about 85 percent.

Shale: The carbonate content measured from selected shale samples ranged from 3 to 77 percent, with an average value of about 37 percent.

Sandstone: The carbonate content measured from selected sandstone samples ranged from 0 to 7 percent, with an average value of about 3 percent.

2.5.4.2.3.4.2 Petrographic Analysis

Petrographic analysis was performed in two phases by Spectrum Petrographics Inc. of Vancouver, Washington, and is summarized in two reports dated May and August 2007. Petrographic and photomicrographic descriptions were interpreted from x-ray diffraction analysis on thin sections. The descriptions included mineralogical composition, texture, and the alteration conditions. A total of 39 rock samples were analyzed. A statistical summary of the mineralogical compositions interpreted from the petrographic and photomicrographic analysis performed on the rock samples is presented in Table 2.5.4-213.

2.5.4.2.3.4.3 X-Ray Diffraction Analysis

X-Ray Diffraction analysis was performed at Portland State University in Portland, Oregon. X-Ray diffraction analysis was performed on random powder from the entire rock samples and the clay size fraction from crushed samples. A total of 14 rock samples were analyzed. Table 2.5.4-214 presents a statistical summary of the mineral quantification of the clay-size fraction ($< 2\ \mu m$) and the mineral quantification of the random powder of the analyzed bulk sample.

2.5.4.2.3.4.4 Organic Content

Organic contents were measured in two samples of fine-grained residual soils. Results indicate an organic content of 1.9 percent for a sample of sandy clay and 2.6 percent for a sample of silty clay.

2.5.4.3 Foundation Interfaces

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.3 with the following.

The following subsections describe the subsurface conditions determined from the extensive investigation and resulting data. The boring data, including detailed core descriptions, geophysical logs and surveys and laboratory test results, are

used to divide the vertical section into layers that are distinguished by different physical characteristics. These engineering layers were applied to develop a representative static and dynamic profile for engineering analysis as well as development of the seismic ground motion for the site, as described in Subsection 2.5.2. Significant discussion is focused on a prominent and thick limestone layer (referred to as engineering Layer C), the top of which is present at about 40 ft below the yard grade (elevation 822 ft). This limestone layer is the foundation bearing layer for all seismic category I structures. There are no site-specific seismic category I structures resting on backfill. Layer C has a uniform thickness of about 60 ft and a consistent S-wave velocity of about 6300 fpc. Subsurface conditions to a depth of about 550 ft are described in the following subsections.

2.5.4.3.1 Engineering Stratigraphy

The subsurface conditions and engineering stratigraphy for the site area are based on the integrated data acquired from the geotechnical exploration program described in Subsection 2.5.4.2 and shown on Figure 2.5.4-202. Figures 2.5.4-206, 2.5.4-207, and 2.5.4-208 are examples of boring in situ test summary logs from key boreholes that integrate geologic and geophysical data to help define and correlate engineering layers through the site.

Site bedrock materials are divided into discrete engineering layers for evaluation of foundation and seismic site response characteristics. The bedrock formations extending from the ground surface to a depth of about 550 ft (approximately elevation 294 ft) are divided into 13 stratigraphic-engineering (engineering) rock layers (Figures 2.5.4-204 and 2.5.4-205), and a thin cover of surface residual soils and localized undocumented fill. Engineering rock layers are correlated with the regional geologic stratigraphy described in Subsection 2.5.1, and rock strata defined for the CPNPP Units 1 and 2 FSAR that include the Glen Rose Formation, Twin Mountain Formation, and Mineral Wells Formation. Figure 2.5.4-205 shows the correlation between the site engineering layers and those defined for CPNPP Units 1 and 2. Each engineering layer is a unique stratigraphic layer differentiated on the basis of lithology (e.g., shale or limestone), rock mass property (e.g., degree of fracturing or cementation), geotechnical index properties (e.g., plasticity, shear strength), and geophysical characteristics (e.g., seismic wave velocity, natural gamma signature). Assigned engineering layers are laterally continuous throughout the CPNPP Units 3 and 4 site (and extend to the Units 1 and 2 site), and exhibit relatively constant thickness and material properties. Little to no lateral variations or changes are observed in the individual engineering layers throughout the site, based on characteristics observed in numerous borings and geophysical surface and borehole surveys.

The engineering layering follows an alpha-numeric system starting with the shallowest Glen Rose Formation upper limestone strata (Layer A) that occurs at, or near, the ground surface (locally buried by residual soil and/or fill). The vertical segregation of the profile into generalized engineering layers is based primarily on lithologic layers that can be correlated from borehole to borehole, and by geophysical survey velocity layers. The Glen Rose Formation is divided into

engineering Layers A through E. Layers B and E were further subdivided into sub-layers (B1, B2, E1, E2 and E3) to segregate important beds of shale or shaley intervals in the otherwise massive limestone of the Glen Rose Formation. Twin Mountain Formation, which underlies the Glen Rose Formation, includes engineering Layers G through I that consist of interbedded weakly cemented sandstone, shale, claystone, and sandstone. The deepest stratum encountered during site exploration is the Minerals Wells Formation that is designated engineering Layer MW.

Figures 2.5.4-216 and 2.5.4-217 show the locations of seismic category I and II plant facilities and the general foundation excavation profile, with respect to the engineering layering defined for the CPNPP Units 3 and 4 site.

The lower Cretaceous Glen Rose Formation (Layers A through E) extends from near ground surface (elevations between about 787 ft and 857 ft) to an approximate elevation of about 620 ft. This correlates to a composite thickness of between about 167 ft and 227 ft, depending on ground surface elevation and thickness of surficial residual soil and fill. The upper portion of the Glen Rose Formation that comprises Layers A and B, is alternating thin to massive beds of limestone and shale. Excavation of a uniform plant grade of 822 ft throughout the CPNPP Units 3 and 4 plant area largely strips away surficial soil and fill, and exposes engineering Layer A limestone at the plant grade surface (Figure 2.5.4-215). Engineering Layers A and B are exposed in the plant foundation excavation walls, and represent the bedrock interface along the lower sidewalls of the plant structures. The middle and lower beds of the Glen Rose Formation are divided into engineering Layers C, D, and E that consist principally of massive limestone (packstone and wackestone) and intervening thin shale interbeds.

Layer C is an approximately 60-foot thick massive and hard limestone layer that is the foundation support for major plant structures, including all seismic category I and II structures. The limestone composition, rock mass properties, and geotechnical properties of Layer C are substantially uniform throughout the CPNPP Units 3 and 4 site area. CPNPP Units 1 and 2 construction photographs show extensive exposures of Layer C limestone in the excavation walls for plant foundations. The Layer C limestone in the photographs exhibits high lateral and thickness uniformity, and appears to stand in a very stable condition in unsupported vertical cuts and in an unsupported tunnel roof for the cooling water tunnel excavations for CPNPP Units 1 and 2.

A thin shale bed defines engineering Layer D and separates the massive Layer C limestone from underlying massive limestone layers in the lower Glen Rose Formation, which includes thin shale intervals defined as Layers E1 through E3.

The contact between the base of the Glen Rose Formation and the underlying Twin Mountain Formation is a transitional, conformable contact identified by a gradational lithologic change from predominantly limestone to predominantly sandstone. This contact is noted to be gradational in many of the borings. Engineering Layer F is an approximately 40- to 50-foot thick transitional layer at

the Glen Rose-Twin Mountain formations interface. The first occurrence of sandstone with shale interbeds identifies the top of Layer F.

The lower Cretaceous Twin Mountain Formation extends from the base of the Glen Rose Formation (approximate elevation 620 ft) to the top of the Mineral Wells Formation at approximate elevation 390 ft. This geologic layer is approximately 217 ft to 242 ft thick. The Twin Mountain Formation underlying the CPNPP Units 3 and 4 site is subdivided into three engineering Layers, G, H, and I, composed of alternating beds of limestone, shale, and sandstone. Layers G and I are composed of relatively massive, weak to moderately cemented sandstone, and Layer H is primarily consolidated shale with sandstone interbeds.

Only borehole B-1012 was drilled deep enough (550 ft) to reach the Mineral Wells Formation below the Twin Mountain Formation. Boring B-1012 is located directly between Units 3 and 4 and was correlated with borings drilled for the existing Units 1 and 2 (Figure 2.5.4-202 and 2.5.4-205). The top of the Mineral Wells Formation was encountered at a depth of 455 ft (elevation 389 ft), and is composed of massive shale with interbeds of sandstone.

As discussed in Subsections 2.5.1 and 2.5.4.1, discrete engineering layers in the Glen Rose Formation can be traced in the subsurface throughout the site, and correlated approximately 2000 ft away in the CPNPP Units 1 and 2 borings and historical construction excavation photographs. Historic construction photographs from the Units 1 and 2 excavation show distinct interbeds of limestone and shale similar in appearance to rock encountered in the upper portion of the Glen Rose Formation at the CPNPP Units 3 and 4 site. These interbeds exhibit flat lying (no apparent dip) limestone and shale strata of varying thicknesses. Based on the known excavation limits, the stratigraphy exposed in the photos is correlated with Glen Rose Formation Layers A, B1, B2, and C (Figure 2.5.4-205). The geologic strata in the Glen Rose and Twin Mountain formations exhibit a gentle southeastward dip that is consistent with the regional dip of geologic layers mapped in the area.

2.5.4.3.2 Engineering Cross Sections

Three engineering cross sections show compiled borehole and geophysics data and interpreted engineering layering under the CPNPP Units 3 and 4 site. Figure 2.5.4-203 shows the locations of the cross sections, and Figures 2.5.4-209 to 2.5.4-211 present the cross sections. The cross sections do not have vertical exaggeration, and show a true 1(H):1(V) scaling.

Borings that fall along the cross section lines and borings within approximately 100 ft from section lines, are projected orthogonally into the sections. Boring data include information on percent recovery, RQD, stratigraphic contacts, and borehole P-S Suspension velocity profiles. The site engineering layers are defined on the cross sections, and shown with respect to CPNPP Units 3 and 4 plant grade, major plant structures, and foundation subgrade elevations. These cross

sections illustrate the relationships between plant foundations and subsurface materials. Each cross section is described in the following paragraphs.

Cross Section A–A' (Figure 2.5.4-209)**:** An approximate east-west oriented cross section that is projected through both Units 3 and 4. Cross Section A–A' is 1800 ft long and shows the interpreted stratigraphy to about 550 ft below ground surface. This cross section is composed of data from 11 boring logs and includes geophysical data and core data for select boreholes.

Cross Section B–B' (Figure 2.5.4-210)**:** An approximate north-south oriented cross section that shows the subsurface conditions below CPNPP Unit 4. Cross Section B–B' is 1140 ft long and displays stratigraphic data up to 398 ft below ground surface. This cross section is composed of data from 12 geotechnical boring logs and includes geophysical and core data for select boreholes.

Cross Section C–C' (Figure 2.5.4-211)**:** An approximate north-south oriented cross section that shows the subsurface conditions below CPNPP Unit 3. It is 1140 ft long and displays stratigraphic layers up to 398 ft below ground surface. This cross section is composed of data from 12 geotechnical boring logs including geophysical and core data for select boreholes.

The cross sections demonstrate that the Glen Rose Formation below plant foundations consists predominantly of massive limestone with relatively flat beds of uniform thickness, and uniform high rock quality without significant zones of structural weakness or deformation consistent with US-APWR Key Site Parameters (DCD Table 2.0-1) requiring no potential for surface deformation at the site, and no potential for soil liquefaction. Liquefaction evaluation is further discussed in Subsection 2.5.4.8.

2.5.4.3.3 Engineering Layer Contour Maps

Interpretive contour maps included in Figures 2.5.4-212, 2.5.4-213, and 2.5.4-214 show interpreted 3-dimensional distributions of the following key geologic layer contacts below the CPNPP Units 3 and 4 plant area:

- Surficial soil and fill thickness (Figure 2.5.4-212)
- Elevation of the top of Glen Rose Formation limestone engineering Layer A (top of rock; Figure 2.5.4-213)
- Elevation of the top of Glen Rose Formation limestone engineering Layer C (foundation subgrade material; Figure 2.5.4-214)

Control boreholes are shown on each figure. Extrapolation of contacts between boreholes is aided by evaluation of geophysical surface refraction layer velocity profiles and CPT soundings.

Contours are shown in 2-foot intervals where data are dense, and intervals of 5 ft where data are less dense.

Each of the contours was drawn using solid lines where data were available and dashed lines where data were inferred. Contours are shown as solid lines where subsurface control is good, and as dashed lines where extrapolated between widely spaced control points (e.g., borings).

The soil isopach (thickness) contour map (Figure 2.5.4-212) shows the distribution and thickness of surficial residual soil and undocumented swale fill. The thicknesses of soil and fill are variable throughout the plant site area, typically ranging between about 5 ft and 15 ft thick for residual soil, and between about 10 ft and 75 ft for swale fill. The swale fill thickness exhibits a steep gradient of increasing thickness near the margin of SCR, and typically ranges between about 5 ft and 15 ft within the CPNPP Units 3 and 4 power block and footprints, and between about 10 ft and 45 ft in the UHS areas. As discussed in Subsection 2.5.4.2, extensive subsurface explorations by borings, test pits, CPT soundings, and geophysical surface refraction surveys provide a high degree of control to define the margins of the swale fill areas with respect to the plant power block footprints. Based on information provided in Figure 2.5.4-212, residual soil and swale fill is largely stripped from the power block areas by mass excavation to form the plant grade at elevation 822 ft (Figure 2.5.4-215). Only thin residual soil or fill, about 2 ft to 5 ft, remains along localized areas of the power block perimeters.

Deeper excavation for power block and seismic category I and II foundations extends into Glen Rose Formation rock far below the remaining residual soil or fill. The thin mantle of residual soil and fill locally exposed in the upper parts of the perimeter foundation excavations is readily removed or laid back to mitigate any potential adverse impacts (e.g., shallow slumping or erosion into excavations). Essentially, the entire height of the foundation excavations along the power block perimeter is made in the Glen Rose Formation engineering Layers A and B.

Foundation excavations for the UHS structures encounter relatively thick deposits of swale fill that locally form a large percentage of the height of the excavation walls. These excavation slopes are laid back or supported to provide temporary construction support, as described in Subsection 2.5.4.5, and backfilled after construction. The foundation subgrade for the UHS is extended into competent Glen Rose Formation engineering Layer C, removing any fill from under the structure footprint. Geotechnical inspection of the exposed subgrade during site grading verifies that competent bedrock formation is exposed.

Figure 2.5.4-213 shows contours defining the elevation of the top of sound rock, correlative with Glen Rose Formation engineering Layer A throughout the CPNPP Units 3 and 4 plant site. An irregular bedrock surface, developed by past erosion, exhibits an overall slope to the north and east towards SCR. Former topographic swales northeast of Unit 4 and east of Unit 3 were eroded approximately 10 ft to 25 ft into bedrock prior to later in-filling by undocumented fill and residual soil.

Variations in the elevation of the top of rock, about 15 ft to 25 ft, occur within the power block footprints. The top of rock typically occurs above plant grade elevation of 822 ft, resulting in exposure of a flat rock surface at yard grade over most of the power block area (Figure 2.5.4-215). The top of rock elevation is more variable in the UHS areas, with differential elevations of about 30 ft to 40 ft (Figure 2.5.4-213). Massive excavation only partly exposes Glen Rose Formation engineering Layer A rock within the UHS footprint areas. The top of rock remains below the elevation of plant yard grade under the northeast portions of the Units 3 and 4 UHS footprint areas, but is reached by deeper foundation excavations that extend into competent engineering Layer C limestone (Figures 2.5.4-210 and 2.5.4-211).

Elevation contours of the top of Glen Rose Formation engineering Layer C, supporting seismic category I and II structures, are shown on Figure 2.5.4-214. The contoured contact is a conformable bedding contact in the Glen Rose Formation that exhibits an overall gentle east to northeast dip of less than about 1 degree, consistent with the regional bedrock structure discussed in Subsection 2.5.1. This contact represents an essentially horizontal buried surface within the restricted power block footprint area. The average elevation of the top of engineering Layer C is approximately 782 ft below the Unit 3 and Unit 4 power block (Figure 2.5.4-214). The Layer C contour map demonstrates the geometry of the foundation interface for plant structures, and shows that the foundation layer satisfies the US-APWR Key Site Parameters (DCD Table 2.0-1) criteria for maximum slopes of foundation bearing stratum of less than 20 degrees from horizontal.

2.5.4.4 Geophysical Surveys

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.4 with the following.

Geophysical surveys included both down-hole and surface surveys using methods described in Subsection 2.5.4.2.1.7. The following subsections describe how each of the techniques were integrated and applied to characterization of the subsurface conditions.

2.5.4.4.1 Integration of Geophysical Data

Subsection 2.5.4.2.1 describes the locations and general methodology for geophysical surveys at the CPNPP Units 3 and 4 plant site. Detailed results from the surface and borehole geophysical surveys are presented in project data reports. Integrated summary results from these surveys are described herein.

Locations, methodologies, and applications for borehole geophysical measurements are discussed in Subsection 2.5.4.2.1.7. The resulting geophysical measurements provide important independent correlation of bedrock stratigraphy and structure, as well as measurements of in situ engineering and seismic wave transmission properties. This information is integrated to develop a 3-dimensional geologic model of the volume of rock under, and within the foundation influence

zone, of CPNPP Units 3 and 4 plant power block and seismic category I and II structures. These properties are summarized in tables, text, and figures presented in Subsection 2.5.4.2. Multiple measurements using the same borehole methods, as well as independent measurements of the same properties using multiple geophysical methods, provide robust characterization of the site properties and confirm the high degree of lateral uniformity within individual engineering layer stratigraphy described in Subsection 2.5.4.3.

2.5.4.4.2 Integrated Seismic Velocity Profile

The integrated seismic velocity profile is divided into the shallow profile (surface to a depth of about 500 ft) and the deep profile (from a depth of about 500 ft to “basement”). The shallow profile represents the depth to which extensive characterization has been performed. The lateral and vertical control on the subsurface strata (engineering layers) was defined primarily on lithology and material properties obtained from the 141 geotechnical and 20 monitoring well profile borings. The velocity measurements in the shallow profile have been developed from 15 suspension logs that demonstrated a consistent correlation with the site stratigraphy as well as repeatable measurements between borings. These measurements were cross-checked with down-hole methods as well as compared to available cross-hole measurements conducted for the Units 1 and 2.

The deep profile was characterized from regional wells and maps. Strata that define the deep profile are based primarily on lithology and stratigraphic surfaces extrapolated to the CPNPP Unit 3 and 4 site. Velocity data for the deep profile were limited to only a few wells and consisted primarily of compressional wave velocities; however, limited S-wave velocities were acquired and used to develop a representative profile for the site. Basement was defined as the depth at which S-wave velocity of 9,200 fps and greater was achieved. Basement was therefore defined as the top of the Ellenburger limestone, which is a regionally extensive layer with an estimated S-wave velocity of nearly 11,000 fps.

A discussion of the aleatory and epistemic uncertainty for development of the integrated seismic velocity profile is provided in Subsection 2.5.2.

2.5.4.4.2.1 Shallow Seismic Profile

The shallow velocity profile (extending to the maximum depth of site exploration) has been extensively characterized by 141 geotechnical borings, 20 monitoring well stratigraphic profile borings, and site geologic mapping. These data were used to develop the site engineering stratigraphy that was demonstrated to closely match the velocity layering in the 15 P-S Suspension borings.

The foundation basemats of all seismic category I and II structures will be founded on a limestone layer (engineering Layer C). Excavation to Layer C will remove the shallower materials (Layers A, B1 and B2). Where the top of Layer C is below the bottom of the foundation elevation, fill concrete will be placed to achieve the bottom of basemat elevation. The average thickness of Layer C is greater than 60

ft and dips less than 1 degree. The average S-wave velocity of Layer C is greater than 6000 fps. Profiles for development of the GMRS and FIRS are detailed in Subsection 2.5.2.

Borehole-to-borehole correlation of engineering layers and respective P-S Suspension-log velocity measurements were used to develop velocity profiles for each of the fifteen suspension-log borings as well as a representative integrated site profile to a depth of about 500 ft. The mean engineering layer surface elevations and thicknesses were used to produce the integrated shallow seismic velocity profile. The mean layer boundaries determined from the entire set of geotechnical core borings and monitoring well profile holes described in Subsection 2.5.4.2 were used to evaluate potential lithologic variability and geologic influence on seismic wave transmission properties.

A summary plot of measured P- and S-wave velocity versus elevation is included on Figure 2.5.4-239. Spatial distribution of velocities for each engineering layer was evaluated to determine if lateral variability could be delineated and used to group borings for development of specific FIRS in conformance with the US-APWR Standard Design. As indicated by the statistical variability (defined as the standard deviation about the mean), the velocity measurements across the site for each individual engineering layer are very consistent. Therefore, a single seismic velocity profile is warranted for the shallow subsurface.

The elevation ranges for velocity layer boundaries match closely to stratigraphic engineering layers described in Subsection 2.5.4.3. Throughout the Glen Rose Formation, shale layers exhibit markedly lower velocities than limestone layers, and the interfaces between these strata are typically sharp and accompanied by strong velocity changes (Figure 2.5.4-239). Typical shale S-wave average velocities are about 2,500 to 3,000 fps. In contrast, typical limestone S-wave average velocities are about 3,500 to 7,000 fps. Seismic velocities exhibit a uniform decrease in Twin Mountain Formation engineering layers F through I (weakly cemented sandstone, shale, and claystone), irrespective of lithology. These layers exhibit a relatively small range in S-wave average velocity, about 3,000 to 3,500 fps. Seismic velocity increases substantially within the better-indurated shale and sandstone of the Mineral Wells Formation, with a typical average of about 5500 fps.

The deepest velocity measurements, and the only measurements within the lower parts of engineering layer I of the Twin Mountain Formation and Mineral Wells Formation, were acquired in boring B-1012, which was located halfway between the Unit 3 and 4 power blocks, as shown on Figure 2.5.4-202. Velocity measurements in this boring were acquired to a depth of about 550 ft. Drilling problems encountered in this boring resulted in an interval from 415 ft to 465 ft of depth (a 50-foot interval) where no velocity measurements were acquired. This missing interval corresponds to the lower portion of Layer I, which is predominately sandstone, as indicated from detailed petrographic analysis for samples HS-15, HS-16, and HS-17 (Table 2.5.4-213). The bottom 10 ft of the missing interval corresponds to the Mineral Wells Formation, which is

predominantly sandstone and silty claystone, as indicated from the core boring, corresponding geological signature, and detailed petrographic analysis (samples HS-18 and HS-19). Velocities were calculated for this interval by evaluating the condition of the core and other geophysical test results in the strata above and below the missing velocity interval. Based on these comparisons, velocities calculated for Layer I were extrapolated through the missing section to the top of the Mineral Wells Formation.

The average velocities for each engineering layer in the composite velocity profile were calculated by combining the mean layer elevations and thicknesses for each discrete borehole velocity profile, and applying a geometrical mean to develop a single average value representing the entire velocity data set. These layer average velocities were then used to calculate Poisson's ratios that are consistent with the average properties of each layer.

Velocity data provided in the available Cross-Hole Data Report for CPNPP Units 1 and 2 were also used to compare the site stratigraphy and representative velocity measurements between Units1 and 2 and Units 3 and 4 to a depth of about 500 ft beneath the site. Figure 2.5.4-205 compares the engineering stratigraphy layers of Units 1 and 2 and Units 3 and 4, plotted at their respective elevations. This is consistent with extrapolation of contact elevations from Units 3 and 4 to Units 1 and 2, based on regional strike and dip of bedrock (Reference 2.5-417). The agreement between the Units 3 and 4 P-S Suspension and the cross-hole survey for Units 1 and 2 are within about 10 percent.

The velocity range for the shallow profile was assigned a variability of +/-25 percent of the mean velocity of each engineering layer. This range envelopes the variability in S-wave velocity measured in the shallow profile rock sequence in the multiple borehole P-S Suspension borings, down-hole survey results, and cross-hole measurements from the Units 1 and 2, providing a conservative means to capture both epistemic and aleatory uncertainty.

2.5.4.4.2.2 Deep Velocity Profile

The deep profile defined as extending from about 500 ft depth (the bottom of shallow profile discussed above) and hard basement was developed from existing published data from regional wells and geologic maps. Strata that define the deep profile are based primarily on regional geologic formations projected to the Units 3 and 4 site. Therefore, elevations of primary stratigraphic layers and velocity boundaries have a higher uncertainty than the shallow profile. This uncertainty is incorporated in the ground motion calculations presented in Subsection 2.5.2.5 by varying the elevations of the geologic formation interfaces in the ground motion model.

A variety of regional information was used to determine the deep stratigraphy for CPNPP Units 3 and 4. Stratigraphic and velocity data were acquired from published literature and regional oil and gas wells.

The resulting deep stratigraphic profile begins in the lower Pennsylvanian Strawn group, which contains the Mineral Wells formation, the deepest unit defined as part of the shallow profile in discussed in Subsection 2.5.4.4.2.1. The remainder of the Strawn Series is lithologically similar to the Mineral Wells and consists of shales and intebedded sandstones and limestones. Included within the Strawn Series are the Garner and Millsap Lake formations. Below the Strawn is the Atoka Group which includes the Atoka Sand, the Smithwick Shale, and the Big Saline Conglomerate. The top of the Atoka Group, the Atoka sand, is shale interbedded with sands and limestones. The sandstone layers have an average thickness of about 30 feet (Reference 2.5-440). To the north and west of the study area, the upper portion of the Atoka Group includes the Caddo Reef, a massive limestone. In Sommervell County, however, located closer to the Ouachita thrust belt, deposition was more likely terrigenous (Reference 2.5-440). Beneath the Atoka sand, the Smithwick is primarily a black shale, with a thickness that varies from 300 to 600 feet (Reference 2.5-417). Below the Smithwick shale, the Big Saline Conglomerate has a variable thickness and pinches out just southeast of the site, so that at CPNPP Units 3 and 4 it has a projected thickness of about 40 feet. Underlying the Atoka Group is the Marble Falls limestone. The upper portion of this unit is a dark-colored fossiliferous limestone (Reference 2.5-417). The lower portion of the Marble Falls is interbedded dark limestone and gray-black shale, sometimes referred to as the Comyn Formation (Reference 2.5-343), and sometimes considered part of the Barnett Shale, which is stratigraphically below the Marble Falls. The Mississippian Barnett Shale (250 to 1000 ft thick, regionally) represents a gas source and reservoir in the region. The Barnett Shale unconformably overlies the top of the Ellenburger Group throughout most of the Fort Worth Basin, though in the northeastern portion of the basin the Upper Ordovician Viola and Simpson limestones intervene (Reference 2.5-343). The Cambrian to Ordovician Ellenburger limestone and a thin underlying clastic sequence rests unconformably on metamorphic basement in the Fort Worth Basin and was deposited in a passive continental margin setting (Reference 2.5-343).

The methods for determining stratigraphic elevations of units are listed in order of confidence:

- The top of the Strawn (Mineral Wells formation) was measured in wells at the CPNPP Units 3 and 4 site location.
- Using GEOMAP-stated elevations of horizons in the three nearest wells, the attitude of each horizon was determined and the elevation projected to the site location.
- The CPNPP Units 3 and 4 site was projected onto the line of section of GEOMAPS cross section through two nearby wells (Squaw Creek and 1-Davis).
- Horizon elevations determined from GEOMAPS structure contour maps.

For all stratigraphic units, more than one method was available for determining the elevation of a given horizon, and the standard deviation (σtop) of the elevations was used as an estimate of the error. Only a single elevation pick was determined for the top of the Big Saline thus, the average standard deviation in feet for the other stratigraphic units was applied as an estimate of the error.

S-wave velocity measurements were available from a single well located about 6 miles from the site, and were limited to a 4,000 ft thick section extending from the lower portion of the Marble Falls limestone, through the Barnett shale, and into the Ellenburger limestone, the top of which occurs at an estimated depth of about 5000 ft below the site, as described in Subsection 2.5.1. Basement is defined as the top the Ellenburger Formation for the Units 3 and 4 site, based on the layer thickness, regional extent, and measured S-wave velocities in excess of 9200 fps.

At an elevation of about -3973 ft, the Marble Falls limestone records an S-wave velocity of about 10,520 fps. Though this layer is sufficiently fast to be considered seismic basement (S-wave velocity > 9,200 fps), it is underlain by the seismically slow Barnett shale. The top of the underlying Ellenburger limestone is a thicker geologic formation mapped at an elevation of about -4443 ± 73 ft, which has an S-wave velocity of about 10,906 fps. The Ellenburger limestone encountered in the Officers Club well indicates greater than 3,000 ft of material with S-wave velocities greater than 9,200 fps.

A linear extrapolation was used to estimate the S-wave velocity between the top of the Ellenburger limestone and bottom of the shallow velocity profile to complete the velocity profile for the site. The variability about the harmonic mean S-wave velocity for the deep profile was assigned a coefficient of variability of 31 percent based on the largest variability calculated for all geologic formations where more than one velocity measurement was available.

The shallow and deep profiles, as described above, were combined by coupling the Strawn Group using the Mineral Wells Formation, which is the deepest stratigraphic layer encountered in the geotechnical exploration for Units 3 and 4 and the shallowest layer characterized for the deep profile.

2.5.4.5 Excavations and Backfill

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.5 with the following.

This subsection discusses site preparation, excavation, backfill, and earthwork requirements for CPNPP Units 3 and 4 site. The following items are addressed in this section:

- Horizontal and vertical limits of excavation, exposed subgrade preparation, fills, and slopes
- Construction excavation, temporary cut slopes, and dewatering

- Backfill material types, sources, specifications, and quality control observation and testing
- Foundation excavation, subgrade, and slope geologic monitoring during construction

Figures 2.5.4-209, 2.5.4-210, 2.5.4-211, and 2.5.4-217 illustrate the general layout and general excavation requirements for the main plant structures. Figures 2.5.4-246 and 2.5.4-247 provide preliminary excavation plans for CPNPP Units 3 and 4, respectively. Preliminary excavation section profiles along three north-south and four east-west directions are shown on Figures 2.5.4-248 through 2.5.4-254 for Unit 3, and on Figures 2.5.4-255 through 2.5.4-261 for Unit 4. For general grading and site preparation to plant yard grade elevation of 822 ft (Figure 2.5.5-204), excavation cuts of up to about 45 ft are required within the CPNPP Units 3 and 4 site. The general excavation cuts completely strip all surficial soils and the upper weathered zones of the Glen Rose Formation engineering Layer A. For foundation installations of the structures within the power block and UHS areas, additional temporary excavations are required to depths of approximately 40 ft to 45 ft below the yard grade elevation of 822 ft. As shown on Figure 2.5.4-217, Glen Rose Formation Layer B, which consists of shale beds, daylights into the temporary excavation sidecuts near the bottom of the excavation, creating. potential low strength beds and interfaces. The shale strata are generally horizontal, a geometry that is favorable for stability. However, shale strata are considerably weaker materials than limestone strata, and may undergo significant softening and pose potential sliding surfaces that undermine the rock masses within the excavation banks. Although the construction experience from CPNPP Units 1 and 2 suggests that vertical cuts are viable, construction precautionary and preventing methods (e.g. rock anchors or angle cut) that are typical procedures in bedded rock formations with potential weak zones provide an acceptable level of construction stability and ensure the safety of personnel and workers during construction. Since all temporary excavations are backfilled with engineered compacted fill, the potentially weak shale beds above the elevation of about 782 ft do not cause any hazard or instability issues to any of the CPNPP Units 3 and 4 seismic category I and II structures.

2.5.4.5.1 Cut, Fill and Excavation Limits

The limits of general site grading, excavation, and backfill for the power plant are shown on the preliminary grading and drainage plans. Site grading does not produce cut or fill slopes that directly support the seismic category I and II structures, or that are in sufficient proximity to be a potential hazard to seismic category I and II structures. Subsection 2.5.5 discusses slope stability analyses of permanent cut and fill slopes, and relationships to seismic category I and II plant structures. All seismic category I and II structures are supported on deeply embedded foundation mats that bear directly on prepared and cleaned sound rock of Glen Rose Formation limestone of engineering Layer C (Subsection 2.5.4.3).

The pre-construction ground surface grades within the CPNPP Unit 3 vary between approximate elevation 830 ft and 855 ft in the power block area, and between elevation 790 ft and 847 ft in the UHS area. For Unit 4, ground surface grades vary between approximate elevation 842 ft and 869 ft in the power block area, and between elevation 820 ft and 842 ft in the UHS area. Based on the site grading drawings (Figure 2.5.5-204), the post-construction main plant area for both Units 3 and 4 power blocks and UHS areas encompasses a rectangular pad roughly 1,700 ft long (east to west) and 1,100 ft wide (north to south) to form a relatively level plant grade ranging between elevations 820 ft and 822 ft. This requires overall area cuts ranging between 8 ft and 47 ft in the power block areas, and cuts of up to 27 ft in the UHS areas. The only area requiring fill is the northeast corner of the pad within the eastern two UHS structures of Unit 3, where fill of up to 30 ft is needed.

As discussed in Subsection 2.5.4.3, mass excavation removes all surficial residual soil and undocumented fill from the power block footprints, and exposes a flat surface comprised primarily of Glen Rose Formation engineering Layer A limestone. Figure 2.5.4-215 illustrates geologic layers exposed at plant yard grade elevation of 822 ft. Some residual soil and undocumented fill remains at plant grade in the areas of the UHS, but further excavation for the foundations of these structures strips these materials from the structure footprints. Additional excavations approximately 40 ft to 45 ft below plant yard grade elevation of 822 ft are required under the power block and UHS footprints to reach foundation basemat elevation of approximately 782 ft. Within east and northeast portions of Unit 3, and possibly in isolated areas of Unit 4, some additional “overexcavation,” possibly to elevations of low as about 778 ft (Figure 2.5.4-214), is required to reach the target Glen Rose Formation engineering Layer C limestone for foundation support.

A stretch of 15- to 50-foot-high cut slopes is formed along the west and south margins of the power plant main pad. These cut slopes have inclinations ranging between about 2(H):1(V) and 3.5(H):1(V). The closest approach between the toe of the cut slopes and seismic category I or II structures is approximately 150 ft, providing a substantial safe distance back from the cut slopes. Along the northern margin of the general plant area in the vicinity of the UHS structures, fill is placed on the reservoir slopes to form the outbound edge of the power plant yard. The fill slopes are approximately 25 ft to 30 ft high, and are inclined at approximately 2(H):1(V). Where the toe of the fill would otherwise project into the reservoir north of Unit 3, a 15-foot-high vertical retaining wall is constructed to constrain the fill. Stability analysis in Subsection 2.5.5 includes an evaluation of the slope and retaining wall. As discussed previously, the UHS structures bear on sound Glen Rose Formation limestone Layer C reached by deep excavation under the structure footprints. The fill slopes north of the UHS structures are used to re-establish ground surface grades on the reservoir side, and do not provide support for the UHS foundations or structural walls that are designed to be self-standing.

2.5.4.5.2 Excavation and Excavation Support

Figures 2.5.4-209, 2.5.4-210, and 2.5.4-211 illustrate the general excavation requirements below plant yard grade to reach Glen Rose Formation limestone Layer C that forms the foundation mat subgrade for all seismic category I and II structures and plant power block at elevation 782 ft. Steep to vertical cuts will be made around the perimeters of the power block and UHS areas, and a level, cleaned excavation floor in limestone will be developed for foundation inspection and preparation, as is illustrated on Figure 2.5.4-217. Some localized overexcavation may be required below elevation 782 ft to remove weathered, dilated, or shaley rock zones. Any overexcavation areas are backfilled to foundation subgrade elevation with fill concrete.

Excavation of residual soil, undocumented fill, and the upper several feet of the weathered zone at the top of the Glen Rose Formation can be accomplished with conventional heavy earth moving equipment, possibly with some areas of ripping.

Photograph records of CPNPP Units 1 and 2 construction show near vertical, 80 ft high sidecuts in the Glen Rose Formation bedrock. The photographs show that excavations were made without the use of rock anchors or any other excavation support, and appeared to be stable.

The sequence of Glen Rose Formation rock exposed in the CPNPP Units 1 and 2 excavations are the same layers that occur within the excavation ranges at Units 3 and 4. The extensive network of exploratory core borings and geophysical surveys performed throughout the Units 3 and 4 plant power blocks and UHS areas show that the rock is sub-horizontal, relatively uniform, and generally free of major steeply dipping discontinuities, shears, or dissolution zones. The horizontal to sub-horizontal bedding planes between discrete shale and limestone strata that typically are several inches to several feet in thickness are the primary rock mass feature. These conditions are favorable for excavation stability. As discussed in Subsection 2.5.4.1, rock stresses at the site are low, and significant stress-relief effects (e.g., excavation floor heave, sidewall bulging) are not anticipated. Geologic conditions are favorable for stability, and past construction experience for CPNPP Units 1 and 2 using vertical and unsupported deep rock excavations was positive. However, it is conservatively assumed that vertical tension cracks could develop in the rock mass behind excavation faces. Such tension cracks, combined with low strength shale bedding surfaces that daylight near the base of the excavation cuts, potentially form shallow rock blocks that could topple into the excavation in an unsupported condition.

Analyses of stability of temporary cut slopes indicate that if deep tension cracks were to develop, the computed short-term static factor of safety (construction period) would be less than the conventionally accepted minimum value of 1.3. Slope stability analyses indicate that adequate factors of safety (equal or greater than 1.3) could be achieved with a 0.25(H):1(V) or flatter rock cut slopes.

Two options are considered to permit safe excavation conditions:

- Vertical cuts with rows of rock anchors placed in a top-down sequence during excavation to prevent development of tension cracks; or
- Reduced slopes excavated at a maximum inclination of 0.25(H):1(V) without rock anchors.

Temporary cut slope inclinations in rock no steeper than 0.25(H):1(V) are expected to minimize the adverse effect of tension cracks, although flatter slopes might be locally recommended during construction quality control evaluation, depending on the actual rock conditions encountered in the field.

Cut slopes 40 ft or greater in height are provided with 10-foot wide flat benches at the mid height of the slope to control drainage from runoff during storm events, to provide a catchment to protect workers from loose rocks or materials dropped into the excavation, and to provide a potential access road if additional scaling of the rock surface or any other slope repairs are necessary.

Soil Excavation: Residual soil and/or undocumented fill overlie bedrock in some areas of the site. Interpreted contours of thickness of residual soil and fill materials are shown on Figure 2.5.4-212. Available data suggest that the maximum thickness of fill of nearly 45 ft occurs in the vicinity of the northeast corner of Unit 4, as well as at the southeast corner of Unit 3. Mass excavation to form plant yard grade largely removes these materials, and only localized and relatively thin remaining residual soil and undocumented fill remain north and east of the power block areas, as shown in Figure 2.5.4-215. The exceptions are the UHS areas, where some areas of relatively thick residual soil and undocumented fill remain below plant grade.

Temporary cut slopes in residual soil or undocumented fill are no steeper than 2(H):1(V). These cut soil slopes may require periodic maintenance and need protection against erosion, and include a minimum 6-foot wide bench at the mid height for cases where slopes exceed 25 ft in height.

Rock Excavation: Foundation excavations below plant yard grade in the power block and parts of the UHS areas are mainly within the relatively hard limestone of Glen Rose Formation engineering Layers A and B (Subsection 2.5.4.3), as illustrated in Figures 2.5.4-209, 2.5.4-210, and 2.5.4-211. Some shale beds and shaley zones occur in this rock sequence, primarily within the engineering Layer B in the lower parts of foundation excavations.

The upper several feet of rock is typically moderately weathered and dilated, but below this zone the rock mass is generally only slightly weathered to fresh and tight. Exploratory borings in the rock mass indicated RQD values average over 90 percent (Figure 2.5.4-240) and P-wave velocities with averages between about 7000 to 9000 fps in shale and limestone of engineering Layers A and B,

respectively, and over 11,000 fps in limestone of engineering Layer C (Figure 2.5.4-239).

Caterpillar Equipment Rippability guidelines (Reference 2.5-424) indicate that sedimentary rock (shale, sandstone, siltstone, claystone, and limestone) with P-wave velocities below about 5000 to 6000 fps can be excavated without blasting with a medium weight tractor ripper (such as Caterpillar D8R/D8T). Materials with P-wave velocities of 7500 to nearly 10,000 fps can be excavated using a heavier weight tractor ripper (such as Caterpillar D11T) without blasting.

Based on P-wave velocity data, the Glen Rose Formation engineering Layers A and B that form the bulk of the foundation excavation are marginally rippable to possibly non-rippable with a heavy weight tractor ripper. Because rippability is marginal with such heavy equipment, blasting and/or pre-splitting are most likely required for efficient excavation, and to reduce extreme wear on equipment.

Rock excavation for the CPNPP Units 1 and 2 was performed using a combination of blasting, pre-splitting methods, and heavy demolition equipment (hoe-rams), thereby providing a precedent for that these methods to be potentially effective and efficient for the Units 3 and 4 excavations. Development of foundation subgrade in hard Glen Rose Formation engineering Layer C requires carefully controlled blasting to reduce blast penetration below the subgrade elevation, prevent excessive fracturing of the foundation rock, and form a relatively level bearing surface. Carefully controlled blasting around excavation perimeters facilitates development of smooth and stable cut walls, and reduces fracturing and loosening of rock sidewalls. The following general blasting procedures are considered.

- Line drilling - This method consists of isolating the excavation area, where primary blasting is done within two to three blast drill hole rows from the final excavation line.
- Pre-splitting - This method uses a line of closely spaced holes drilled and blasted prior to the main blast to dissipate energy from the main blast and protect rock beyond excavation limits.
- Smooth blasting - This method is used when the main excavation is completed within a few feet of the excavation perimeter. A line of perimeter holes is drilled and loaded with light charges to achieve the final grade. Similar to “smooth” blasting, “cushion” blasting (with hole diameter significantly greater than the charge diameter) may provide protection to foundation rock.

Rock Foundation Protection Requirements: Protection of the foundation subgrade is essential to maintain good bearing properties and provide sound limestone rock directly below mat concrete for bearing and sliding capacity requirements. Protection of shale beds in excavation sidewall cuts mitigates potential slaking, raveling, and softening. The following measures provide foundation protection.

- Halting floor excavation about 2 ft short of final elevation if an extended exposure time (e.g. over winter) is anticipated prior to placement of concrete. Final excavation to expose and prepare foundation subgrade on the Glen Rose Formation engineering Layer C commences when equipment and materials are ready for concrete placement.

- Applying shotcrete to the excavated faces in shale beds or shaley intervals of limestone. Shotcrete should be provided with weep holes to prevent water pressure buildup.

- Localized overexcavation and dental concrete in possible zones of blast-damaged rock, closely fractured zones, and unusual deep weathering.

Geologic Mapping of Excavation, Documentation and Monitoring: Geologic mapping is required on a continuous basis during foundation excavation, with mapping consistent with the rock and engineering layer classifications described in Subsection 2.5.4.3. Detailed engineering geologic mapping should be supplemented with photographs, video tapes, and topographic survey of the excavated surfaces and pertinent geologic features exposed. All final excavation cuts and foundation subgrade exposures require final inspection and mapping in order to ensure that all shale and unsuitable materials are removed and competent rock materials are exposed.

2.5.4.5.3 Dewatering

As discussed in Subsections 2.4.12 and 2.5.4.1, permanent groundwater occurs deep in the rock mass below plant grade and foundation subgrade elevations. Groundwater inflows into excavations are not considered to be a significant issue, and no significant dewatering or control measures are required during excavation, or for permanent groundwater control. The groundwater elevation at the site meets US-APWR Key Site Parameter (DCD Table 2.0-1) requirements for maximum groundwater level of 1 ft below plant grade.

Possible temporary (e.g., storm-induced) perched water tables that could develop in thin residual soils or undocumented fill that remain in restricted areas of the Units 3 and 4 power blocks and around the UHS should drain quickly and not produce significant volumes or rates of inflow into excavations. The perched water table condition can be controlled by having sumps and pumps installed at key locations in the excavations.

Other than “perched” water, localized water bearing layers or lenses, no groundwater was encountered in the primary Glen Rose Limestone. Therefore only normal pumping equipment and procedures are required to remove storm runoff and concrete curing water that could enter the open excavations.

During construction of CPNPP Units 1 and 2, only small and localized seeps were reportedly observed in foundation excavations that extended to deeper levels (and lower elevations) than at CPNPP Units 3 and 4.

2.5.4.5.4 Backfill Material

Backfill is required between the foundation excavation sidewalls and lower structural walls of seismic category I and II facilities, the main power block structures, and the UHS. The volume of backfill is minimized by using steep or vertical excavation cuts.

No exclusions are placed on the use of limestone or sandstone derived from the mass grading to develop plant grade or foundation excavations. The total volume of excavation in the Units 3 and 4 power block and UHS areas greatly exceeds the volume of required backfill. Shale materials are not acceptable for backfill material in structural areas because of their fine-grained nature, high plasticity, and expansion potential. Testing of limestone and shale samples is discussed in Subsection 2.5.4.2. Dynamic properties assigned to engineered backfill are discussed in Subsection 2.5.4.7.4. The source of backfill to be used adjacent to category I structures will be the limestone and sandstone removed from the excavation and that there will be sufficient quantity of material from the excavation for that purpose. The acceptance criteria, test method, and frequency of verification for fill placement are provided for each fill application in Subsection 2.5.4.5.4.8. Continuous geotechnical engineering observation and inspection of all fill is required to certify and ensure that the fill is properly placed and compacted as discussed in Subsection 2.5.4.5.4.2.

Clean sand may be used as a select granular backfill material around the buried structure walls. A discussion of the materials for engineered fill is provided in Subsection 2.5.4.5.4.1.1. All major seismic category I and II buildings and structure are founded directly on solid limestone or fill concrete (Subsection 3.7.1.3). Recommendations for concrete fill under power block structure foundations are provided in Subsection 2.5.4.5.4.1.2.

Concrete fill may be used as backfill to replace unsuitable rock removed below elevation 782 ft as part of foundation preparations. The concrete fill foundation details are shown on Figure 2.5.4-217.

2.5.4.5.4.1 Material Properties and Sources

2.5.4.5.4.1.1 Fill

All engineered fill materials need to contain no rocks or hard lumps greater than three inches in size, and require to have at least 80 percent of material smaller than 1/2 inch in size. No organic, perishable, spongy, or other improper material such as debris, bricks, cinders, metal, wood, etc. shall be present in the fill. Three types of engineered fill materials are used at the site.

Structural Fill: Structural fill is used in the majority of excavated areas around Units 3 and 4 and north-facing fill slope areas adjacent to SCR, except where select free-draining materials are required (filter and drain curtain) immediately behind the retaining walls. The structural fill requirements include the following.

- Consist of durable materials free from organic matters or any other deleterious or perishable substances, and of such nature that it can be compacted readily by watering and rolling to a firm and non-yielding state,
- Be granular in nature, with a well-graded grain size distribution and less than 25 percent by weight passing standard US Sieve No. 200 (ASTM D422 and D1140),
- Contain particles no larger than 3 inches in maximum dimension, with less than 15 percent by weight larger than 2.5 in,
- Have an expansion index (ASTM D4829) less than 20; material otherwise deemed to be expansive and is not acceptable,
- Have a liquid limit less than 40 percent, and a plasticity index not exceeding 12 (ASTM D4318), and
- Be placed in lifts no thicker than 8 in (measured in loose state), moisture-conditioned to at least within 2 percent of the optimum moisture content and compacted to a minimum relative compaction of 95 percent (ASTM D1557).

Random Fill: Random fill is used in non-structural areas where no structures or slopes are located within the immediate vicinity. The random fill requirements include the following.

- Consist of durable materials with no appreciable amount of organic matters or any other deleterious or perishable substances, and of such nature that it can be compacted readily by watering and rolling to a relatively firm and stable state,
- Contain particles no larger than 4 inches, and
- Be placed in lifts no thicker than 8 in (measured in loose state), moisture-conditioned to at least within 2 percent of the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D1557).

Pipe Bedding: To be used as pipe bedding material and backfill around the pipe, up to about 12 in from the top of the pipe. The pipe bedding material requirements include the following.

- Consist of granular materials, well graded, with all material passing ½-inch sieve, and at least 95 percent retained on standard US Sieve No. 200 as determined in accordance with ASTM D422.
- Be placed under and equally along both sides of the pipe in uniform layers not exceeding 6 in (measured in loose state) to a height of at least the centerline of the pipe, or preferably to 12 in above the top of the pipe and compacted by hand, pneumatic tamper, or other approved means without damaging the pipe or the coatings.
- Be compacted to a relative compaction of 90 percent, except in the structural areas or within 12 in below the roadways and slabs, where 95 percent relative compaction governs (ASTM D1557).
- Above the pipe zone, general structural fill may be used with a similar degree of compaction as specified for the bedding materials.

Fill is derived from either off-site borrow areas or on-site cut areas and foundation excavations. The excavated materials from on-site areas require appropriate segregation, handling, and processing. Geotechnical testing is required for all fill materials to verify that their characteristics and properties meet the minimum requirements.

Representative samples from fill material are control tested for grain size, Atterberg Limits, Expansion Index, Modified Proctor, pH, sulfates, and chlorides. Where the type or the source of fill materials changes or is suspect, a new set of control tests like the ones indicated above is performed for the new or changed material.

2.5.4.5.4.1.2 Fill Concrete

Fill concrete and flowable fill mix designs are required to be approved in advance to ensure that they meet the minimum strength requirements. Continuous field observation is needed to verify that the appropriate mixes are used. A systematic quality control sampling and testing program is required to assure that the fill concrete and flowable fill material properties are in compliance with the design specifications.

The fill concrete has a design compressive strength of 3,000 psi that corresponds to a shear wave velocity of 6,400 ft/sec. The fill concrete mix design is required to be approved in advance to ensure it meets minimum strength requirements. The fill concrete conforms to pertinent requirements of ACI 349 (Reference 2.5-440) and generally conforms to ASTM C94/C94M-07, "Standard Specification for Ready-Mixed Concrete." Other ACI and ASTM standards applicable to the fill concrete are discussed in US-APWR DCD Subsection 3.8.4.6.1.1.

Safety-related fill concrete conforms to durability requirements given in Chapter 4 of ACI 349 (Reference 2.5-440). Durability of the fill concrete is assured by the

site-specific mix design and by the particular site conditions at CPNPP. The site is located away from the ocean and salt water bodies such that the fill concrete is not exposed to seawater. As stated in Subsection 2.5.1.2.5.9, there are no expansive soils or reactive minerals of appreciable amounts at the site. Therefore, issues related to chemical attack by sulfate, salt attack, or acid attack do not pose concerns for the fill concrete. In addition, CPNPP is located in a relatively warm climate where concerns due to exposure to freeze-thaw action under moist conditions and detrimental effects due to the presence of ice removal agents are insignificant.

The foundation and fill concrete design at CPNPP are such that the issues contained in NRC Information Notice (IN) 97-11 (Reference 2.5-441) are not applicable to fill concrete. No mortar or concrete containing high amounts of calcium aluminate cement is used in foundation or fill concrete. The fill concrete mix design uses Type II Portland cement, consistent with US-APWR DCD Subsection 3.8.4.6.1.1, which is limited to a tricalcium aluminate content of 8% by ASTM C150 and is classified by ASTM C150 as moderately resistant to sulfate attack. The maximum anticipated groundwater elevation is at elevation 760 ft, as stated in FSAR Subsection 2.4.1.2.5 and 2.5.4.1.7. This is well below the anticipated bottom of fill concrete. The fill concrete mix design uses fine aggregates, unlike porous concrete consisting only of coarse aggregates and cement. The plant structures are equipped with dampproofing coatings on the sides of below-grade walls and underground drains to collect underground water and channel it away from the structures. Perched water and precipitation run-off do have the potential to come in contact with the fill concrete. However, because of the low groundwater elevation, the use of non-porous fill concrete, and the low amounts of calcium aluminate present in the mix, erosion and leaching concerns and subsequent related effects discussed in IN 97-11 (Reference 2.5-441) are not an issue at CPNPP. Further, FSAR Subsection 3.8.4.7 requires that ground water chemistry be periodically monitored to assure that it remains nonaggressive with respect to concrete structures.

A systematic quality control sampling and testing program ensures that material properties are in compliance with design specifications. Field inspections verify that the required mix is used and that test specimens are collected for testing.

Testing of fill concrete is performed by a qualified testing laboratory that has an established quality assurance program that conforms to NQA-1 requirements. The testing laboratory implements a concrete fill quality control program that includes all aspects of the fill concrete program from the qualification of materials to confirmatory strength testing. Field testing utilizes preapproved procedures that conform to ASTM C31/C31-08a, “Standard Practice for Making and Curing Concrete Test Specimens in the Field.”

Strength verification laboratory tests are performed to confirm that the compressive strength of the fill concrete is satisfactory. The tests are conducted using cylindrical test specimens molded during construction and conforms to ASTM C39/C39M-05e2, “Standard Test Method for Compressive Strength of

Cylindrical Concrete Specimens.” The specimens are taken from different batches of fill concrete. The strength of the fill concrete is considered satisfactory if the average compressive strength from three cylinders molded at a location equals or exceeds the required strength and no individual strength test falls below the required value by more than 500 psi. If these acceptance criteria are not met, an evaluation of the acceptability of the fill concrete for its intended function is performed before acceptance.

The fill concrete testing results, non-conformance related to fill concrete, and QA audits of fill concrete activities will be reviewed and dispositioned to ensure that the fill concrete meets the specified strength requirement.

These measures will ensure that the design properties of fill concrete are achieved during construction activities.

2.5.4.5.4.2 Compaction Requirement

All engineered fill materials need to be compacted at a moisture content of ±2 percent of the optimum, and to a minimum relative compaction of 95 percent in the structural areas and 90 percent in non-structural areas. The maximum dry density and optimum moisture content is determined in accordance with ASTM D1557.

2.5.4.5.4.3 Clearing and Preparing Fill Areas

Prior to placing engineered fill or concrete fill, the excavation bottoms or the ground surfaces to receive fill need to be observed, probed, tested, and approved by qualified personnel as part of the quality control measures.

2.5.4.5.4.4 Placing, Spreading, and Compacting Fill Material

All fill materials need to be placed in horizontal layers not greater than eight inches in loose thickness. Each layer is required to be spread evenly and mixed thoroughly to obtain uniformity of material and moisture in each layer.

When the moisture content of the fill material is below that specified, water needs to be added until the moisture content is as specified. When the moisture content of the fill material is too high, the fill material needs to be aerated through blading, mixing, or other satisfactory methods until the moisture content is as specified.

After each fill layer has been placed, mixed, and spread evenly, it needs to be thoroughly compacted to the specified degree of compaction. Compaction needs to be accomplished by sheepsfoot rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compacting equipment. Equipment is required to be of such design and nature that it is able to compact the fill to the specified degree of compaction. Compaction should be continuous over the entire area and the equipment should make sufficient passes to obtain the desired uniform compaction.

The surface of fill slopes requires compaction until the slopes are stable and there is no loose soil on the slopes. Compaction of the slopes needs to be performed by over-building and cutting back.

Keying and benching into competent rock or material is required at all locations where the natural slope or excavated ground is steeper than 5(H):1(V), or wherever judged necessary during geotechnical quality control evaluation. All new fills are required to have proper keying and benching through all unsuitable top soil, materials susceptible to creep deformation, existing undocumented or questionable fills, and into competent soil, sound bedrock, or firm natural materials.

2.5.4.5.4.5 Observation and Testing of Fill Placement

Continuous geotechnical engineering observation and inspection of all fill placement and compaction operations is required to certify and ensure that the fill is properly placed and compacted in accordance with the project plans and specifications.

Field density tests based on sand cone method (ASTM D1556) are required to be performed for each layer of fill. Field density tests may also be performed using a nuclear density gage (ASTM D2922) provided that 5 percent of all tests are by ASTM D1556. Moisture content may be determined in the laboratory (ASTM D2216) or in the field using nuclear methods (ASTM D3017). If the surface is disturbed, the density tests are to be made in the compacted materials below the disturbed zone. When these tests indicate that the degree of compaction of any layer of fill or portion thereof does not meet the specified minimum requirement, the particular layer or portions requires reworking until the specified relative compaction is obtained.

The geotechnical quality control for soil backfill placement includes the following minimum in-place field density and moisture content testing.

- All structural fill areas - one test every 1,000 sq ft of each lift
- Under paved areas - one test every 2,000 sq ft of each lift
- Road base and sub-base - one test every 2,000 sq ft of base or sub-base
- Pipe bedding and trench backfill areas - one test for every 50 linear ft of each lift
- Random fill - one test every 5,000 sq ft of each lift
- At least one test for every full shift of compaction operations on mass grading and earthwork

At least one in-place moisture content and one density test are required on every lift of fill, and further placement is not allowed until the required relative compaction has been achieved.

The number of tests is increased if a visual inspection determines that the moisture content is not uniform or if the compacting effort is variable and not considered sufficient to meet the project specification.

Light hand-guided compaction equipment is required for compaction of soils within 5 ft of the below-grade walls or other earth-retaining concrete walls. Heavier compaction equipment can be used at distances greater than 5 ft from the walls. The use of light, hand-guided compaction equipment near the walls avoids applying excessive compaction-induced soil pressure against the wall.

2.5.4.5.4.6 Field Monitoring and Quality Control

This subsection describes methods and procedures used for verification and quality control of the foundation subgrades and materials. Properties of the foundation materials are discussed in Subsection 2.5.4.2.

2.5.4.5.4.6.1 Exposed Subgrades

Quality control is required to verify that competent subgrade and quality foundation materials are exposed prior to placement of fill materials. This applies to foundations supported directly on rock, as well as fill or structural concrete. The quality of rock or fill concrete provides very high safety margins against bearing capacity failure under both static and seismic loading conditions, and allows only nominal settlements to occur.

The quality control testing requirements for rock and concrete foundation material are discussed below.

The procedure for verification of foundation conditions consists of geologic mapping of the final exposed excavation surface prior to placement of foundation concrete or fill concrete.

Geologic mapping of final exposed rock surfaces beneath Units 3 and 4 and any required extension to reach suitable rock material is periodically carried out at a scale of 1 in equals 5 ft. Areas where further detail is needed for documentation of significant features are also documented on the geologic map in order to ensure that all shale and unsuitable materials are removed and competent rock materials are exposed.

The geologic mapping program includes photographic documentation of exposed surfaces and laboratory testing and documentation of significant features.

2.5.4.5.4.6.2 Concrete Fill

Quality control of fill concrete placement below Units 3 and 4 foundation areas is required. Field observation is required to verify that appropriate mixes are used and that test specimens are collected for testing to verify that the required compressive strengths are met.

2.5.4.5.4.6.3 Excavation Monitoring, Observations, Testing, Geologic Mapping and Instrumentation

Geotechnical quality control includes continuous observations and monitoring of excavations during construction as well as geologic mapping by qualified and trained geotechnical personnel and geologists to verify that foundation quality materials are reached.

Observations are required to be performed during 1) general excavation, to achieve mat foundation bearing elevations, 2) additional excavations below the design mat bearing elevations, and 3) cleanout of any defects in the rock foundation. The exposed excavation bottoms also need to be mapped by the project engineering geologist to record the conditions of the foundation prior to placement of reinforcing steel or fill concrete.

Similar to Units 1 and 2 foundation excavations, extensometers are also needed during foundation excavation for Units 3 and 4 to monitor foundation deformation.

2.5.4.6 Groundwater Conditions

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.6 with the following.

2.5.4.6.1 In Situ Packer Testing

Field hydraulic conductivity (Packer) tests were performed within rock in six boreholes at depths ranging between 20 ft and 175 ft, as summarized in Table 2.5.4-206 and illustrated on Figure 2.5.4-224. For tests conducted at depths where borehole logs indicated limestone, hydraulic conductivity values ranged from 4 x 10^{-7} cm/sec to 1 x 10^{-9} cm/sec, with an average of about 3 x 10^{-8} cm/sec. For tests conducted at depths where borehole logs indicated shale, values ranged from 9 x 10^{-8} cm/sec to 1 x 10^{-11} cm/sec, with an average value of about 2 x 10^{-8} cm/sec.

Results of the Packer tests performed during the site investigation suggest that the Glen Rose Formation is fairly tight and practically impervious, with little potential for significant seepage and groundwater. The hydraulic conductivity values are in the lower range for similar rock types (shale, limestone, and dolomite).

2.5.4.6.2 Groundwater Occurrence

According to the preliminary results from monitoring of field piezometers within the Units 3 and 4 area, the piezometric levels range between about elevation 775 ft to 858 ft. However, there are also a number of wells that remain dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation, and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site.

As discussed in Subsections 2.4.12 and 2.5.4.1, permanent groundwater occurs deep in the rock mass below plant grade and foundation subgrade elevations. Groundwater inflows into excavations are therefore not considered to be a significant issue. No significant dewatering or control measures are required during construction excavations. The groundwater elevation at the site meets US-APWR Key Site Parameter (DCD Table 2.0-1) requirements for maximum groundwater level of 1 ft below plant grade.

2.5.4.6.3 Construction Dewatering

Groundwater, seepage, or runoff, if encountered in open excavations during construction, is anticipated to be of a relatively low volume and may be handled by sumping and pumping. Sumps may be placed within either Glen Rose limestone or sub-foundation concrete that replaces excavated shale materials.

2.5.4.6.4 Groundwater Impacts on Foundation Stability

Because foundations bear directly on limestone with no indication of active karst conditions, as described in Subsection 2.5.1.2.4, or on sub-foundation concrete (that replaces excavated shale materials), the presence of groundwater is not anticipated to significantly impact foundation stability, bearing capacity, or settlement characteristics.

Groundwater or seepage may impact construction activities if water infiltrates shale and claystone materials on excavated side slopes. Shale is likely to deteriorate in the presence of water as a result of excavation and construction traffic that exposes shale surfaces to slaking. Shale materials require removal from trafficked surfaces.

Shale is present at the base of slopes excavated for construction of Units 3 and 4. The surface of shale exposed within the excavated slope is required to be immediately covered by shotcrete or other suitable materials upon completion of excavation to prevent deterioration of shale through exposure to air and/or water.

To minimize the buildup of hydrostatic pressures, adequate drainage for below-grade and retaining walls and at the base of all fill slopes is required. Impacts to below-grade wall or retaining wall design and performance of the fill slopes are not significant as long as drainage systems perform satisfactorily.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.7 with the following.

2.5.4.7.1 Overview

This subsection discusses the response of soil and rock to dynamic loading and collection and evaluation of field and laboratory dynamic measurements in order to develop the dynamic site characteristics for seismic design and earthquake engineering purposes. Information presented in Subsections 2.5.1, 2.5.4.1, 2.5.4.2, and 2.5.4.4 form the basis for the dynamic evaluation described herein. The site dynamic properties are used as input for classification of the site in conformance with US-APWR Key Site Parameters (DCD Table 2.0-1), development of the site GMRS presented in Subsection 2.5.2.6.1, and development of FIRS presented in Subsection 2.5.2.6.2. Site dynamic properties also are used for any required SSI analysis as described in Subsection 2.5.2.6.2.

Requirements in 10 CFR Parts 50 and 100 pertaining to site dynamic characterization include:

- An investigation of the effects of prior earthquakes in site soils and rocks including evidence of paleoearthquake liquefaction;
- Field seismic surveys and presentation of interpreted data to develop bounding seismic S-wave and P-wave velocity profiles; and,
- Dynamic laboratory tests on undisturbed samples of foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysteretic damping properties.

All seismic category I and II structures are founded at elevation 782 ft directly on competent and massive Glen Rose Formation Layer C limestone, or thin fill concrete placed over the Layer C limestone. The GMRS and primary FIRS 1 profiles applicable for these conditions are equivalent, and developed at elevation 782 ft at the top of Layer C limestone, as described in Subsection 2.5.2.6. An additional four FIRS profiles (FIRS 2, FIRS 3, FIRS 4_CoV30, and FIRS 4_CoV50) are for specific conditions that are different than the GMRS/FIRS 1 condition. The remaining FIRS are established at plant grade elevation 822 ft and factor combinations of in-place Glen Rose Formation Layers A and B and granular engineered backfill to facilitate evaluation of shallow-embedded plant facilities. The following subsections describe development of the site characteristics used as input for the GMRS and FIRS calculations.

2.5.4.7.2 Site Earthquake Effects

As discussed in Subsections 2.5.2, 2.5.3, and 2.5.4.1, the CPNPP Units 3 and 4 site is located within a stable continent area with relatively low stress conditions and low historic seismicity. No active structural deformation occurs within the site

vicinity (25 mi radius), site area (5 mi radius), and site (0.6 mi radius). No Quaternary faults, liquefaction features, or possible tectonic features have been identified within the site vicinity (25 mi radius) by the U.S. Geological Survey or compilation of local mapping, or were identified by aerial photograph analysis and field reconnaissance within the site area (5 mi radius) for the CPNPP Units 3 and 4 investigations. Site subsurface explorations demonstrate that competent Glen Rose Formation bedrock occurs at shallow depths throughout the plant area. This rock is stable and not subject to earthquake-induced ground failure from liquefaction, lateral spreading, or lurching.

As discussed in Subsection 2.5.2, horizontal peak ground accelerations (PGA) range between 0.04g and 0.07g, although 0.10g is used for seismic design per minimum requirement of Appendix S to 10 CFR Part 50 and US-APWR DCD Subsection 3.7.1.1. No significant adverse ground shaking hazard or seismic slope instability is anticipated at the project site based on the low seismicity and estimated PGA values.

2.5.4.7.3 Field Seismic Velocity Profile Input

Subsection 2.5.4.4.2 discusses the integrated seismic velocity profile for the site, which consists of a shallow profile extending to the maximum depth of site explorations and geophysical surveys, and a deep profile extending from the base of the shallow profile to hard basement.

Figure 2.5.4-239 shows the shallow integrated profile that extends from the ground surface to the maximum depth of site geophysical surveys of approximately 550 ft (elevation 300 ft). On the basis of field measurements, the CPNPP Units 3 and 4 site is classified as a Firm Rock site, according to the US-APWR Key Site Parameters table (DCD Table 2.0-1).

Extension of the site seismic velocity profile between the bottom of the shallow profile to hard basement that exhibits an S-wave velocity of >9200 fps is described in Subsection 2.5.4.4.2.2. Hard basement is defined at the top of the Ellenburger limestone at a depth of about 5273 ft below plant grade. Table 2.5.2-227 presents a stepped deep velocity profile used as input for the GMRS.

2.5.4.7.4 Dynamic Soil and Rock Input Parameters for GMRS and FIRS

Table 2.5.2-227 presents dynamic properties of site soil and rock materials for development of the GMRS and FIRS. These values are based on field and laboratory measurements described in Subsection 2.5.4.2 and the information provided below.

Plant grade is directly underlain by Glen Rose Formation limestone of engineering Layer A around the CPNPP Units 3 and 4 power block, and seismic category I and II structures, with the exception of the UHS, as shown in Figure 2.5.4-215. Foundation support for all seismic category I and II structures and power block is provided by a level, cleaned excavated surface in Glen Rose Formation limestone

of engineering Layer C, as described in Subsection 2.5.4.3 and shown diagrammatically in Figure 2.5.4-217. Layer C is massive, competent limestone with an average thickness of 60 ft. Layer C and underlying Glen Rose Formation Layers D through F (primarily massive limestone with thin shale intervals), are indurated rock materials of Late Cretaceous age that are not susceptible to significant seismically induced strength degradation, particularly at the low level of seismic strain associated with the GMRS ground motions. As a result, static properties measured for Glen Rose Formation rock are reflective of anticipated seismic response (Subsection 2.5.2). Any required overexcavation below seismic category I and II foundation basemat elevations to reach the Layer C limestone are backfilled with fill concrete that is equal to, or stiffer than, the Glen Rose Formation rock layers (Table 2.5.2-227).

Dynamic shear modulus reduction (G/G_{max}) and damping properties for rock strata are developed based on field seismic velocity measurements summarized in Subsection 2.5.4.4.2 and laboratory-determined static properties described in Subsection 2.5.4.2. Best estimate values for both shear modulus and damping are provided for each layer in Table 2.5.2-227, and consider essentially linear response within the seismic strain ranges. As discussed in Subsection 2.5.4.3, the rock strata are horizontal to near-horizontal, and lateral variability in rock properties within each stratum is very low. Therefore, a single set of G/G_{max} and damping curves is justified and can be applied for the site seismic evaluation. Lower bound shear modulus for site rock strata ranges between 110.1 (shale) ksi and 879.1 (limestone) ksi. Upper bound shear modulus for rock strata ranges between 317.1 (shale) ksi and 2,531.7 (limestone) ksi. Low strain damping values range between 1.8 and 2.0 percent, and are based on in situ geophysical borehole seismic velocity measurements for the shallow velocity profile discussed in Subsection 2.5.4.4.2. Low strain damping values for the deep velocity profile below the maximum depth of borehole testing are based on linear extrapolation of velocity and lithologic matching from the shallow profile.

The GMRS and FIRS analysis profiles consider fill concrete between the base of the seismic category I and II structural foundation mats and the top of Glen Rose Formation engineering Layer C. Dynamic modulus values ranging between 748.0 ksi (lower bound) and 2,991.8 ksi (upper bound) for fill concrete are based on an assumed mean S-wave velocity of about 6,400 fps and an approximate wet unit weight of 150 pcf for typical concrete, meeting the specification discussed in Subsection 2.5.4.5.4.1.2.

Although no seismic category I or II structures are supported by engineered fill, limited compacted backfill is placed against the lower structural walls between subgrade and plant yard elevations. Dynamic properties for compacted backfill listed in Table 2.5.2-227 are derived based on standard EPRI (1993) (Reference 2.5-387) shear modulus reduction and damping curves for granular fill. Subsection 2.5.4.5.4.1.1 discusses compacted backfill requirements, including the use of granular material. Fill specifications are generally consistent with the specifications and the fill placed at CPNPP Units 1 and 2, and are derived either from processing of on-site excavation materials, or commercial quarries in the site

vicinity. Compacted backfill is divided into three basic layers: a surface layer from plant grade to a depth of 3 ft; a shallow layer from 3 ft to 20 ft depth below plant grade; and, a deeper layer between the depths of 20 ft and 40 ft below plant grade. Different EPRI curves are used for the fill less than 20 ft deep and greater than 20 ft deep. Shear modulus and damping values are based on assumed mean S-wave velocities of 650 fps for surface fill, 800 fps for shallow fill, and 1000 fps for deeper fill, Poisson's ratio of 0.35, and wet unit weight of 125 pcf. Based on a minimum shear modulus variation factor (C_v) of 1.0, the Upper and Lower bound ranges for shear moduli for compacted fill are between 5.7 ksi and 22.8 ksi for surface fill, between 8.7 ksi and 34.6 ksi for fill between 3 ft and 20 ft deep, and between 13.5 ksi and 54.0 ksi for fill greater than 20 ft deep. The broad range between Lower and Upper Bound values accommodates significant variation in fill properties that are larger than typically achieved by controlled fill materials and placement specified in Subsection 2.5.4.5.4.1.1. This approach conservatively captures reasonable ranges for fill properties. Low-strain damping ratios are assigned as 1.5 percent for fill less than and equal to 20 ft deep, and 1.1 percent for fill deeper than 20 ft. EPRI-based (Reference 2.5-387) shear modulus reduction and damping curves for the compacted fill are shown on Figure 2.5.2-232.

2.5.4.8 Liquefaction Potential

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.8 with the following.

In accordance with the requirements of 10 CFR Parts 50 and 100, an analysis of soil liquefaction potential was performed for soils adjacent to and under the seismic category I and II structures according to guidelines provided in RG 1.198. US-APWR Key Site Parameters (DCD Table 2.0-1) allows no liquefaction potential for seismic category I structures.

Soil materials that are considered to be susceptible to liquefaction include loose saturated sands and non-plastic silts. Liquefaction is typically restricted to Holocene and late-Pleistocene age alluvial soils and hydraulically-placed sand fill in areas of moderate to high seismicity. The site is an area of very low seismicity. The results of the ground motion and site response analysis indicate that the peak ground acceleration (PGA) ranges between 0.045g and 0.07g.

All seismic category I and power block structures associated with Units 3 and 4 are founded on stable Glen Rose Formation limestone Layer C, as discussed in Subsection 2.5.4.3. The Glen Rose Formation rock is late Cretaceous in age, indurated, and not susceptible to liquefaction. As discussed in Subsection 2.5.4.1, no paleoseismic evidence of past liquefaction was observed at the site, or is documented within the 25 mi radius region surrounding the site.

The foundation base mats of all seismic category I and II structures are founded on a limestone layer (engineering Layer C).

The fill materials placed within the excavated areas around Units 3 and 4 and in the north-facing fill slopes are not considered prone to liquefaction for the following reasons:

- All fill material consists of engineered compacted fill with a minimum relative compaction of 95 percent (ASTM D1557). The corrected/ normalized standard penetration test N-Values are expected to be higher than 30 blows per foot, which is outside the range considered susceptible to soil liquefaction.
- The engineered compacted fill materials are not in a saturated state. The permanent groundwater table is well below the engineered compacted fill malerials.
- To minimize any potential for buildup of hydrostatic pressures within the engineered compacted fill, adequate drainage is provided for all below-grade structures and retaining walls, and at the base of all fill slopes.

Thus, the engineered compacted fill does not meet the conditions stated in RG 1.206 or RG 1.198 that would cause suspicion of a potential for liquefaction, and no liquefaction analysis is necessary. Even in the unlikely event that the engineered compacted fill became completely saturated, the soil density is too high and the site PGA range is too low to suspect a potential for liquefaction.Liquefaction is therefore not a hazard to CPNPP Units 3 and 4 seismic category I or major plant structures, and the site characteristics meet the US-APWR Standard Design criteria.

Soil liquefaction is also not anticipated within the engineered compacted fill surrounding Units 3 and 4 structures because 1) the permanent groundwater is below the lowest elevation of fill and 2) fill is placed with a high degree of material control and compaction, and 3) the CPNPP site is an area of low seismicity with low GMRS design motions, as described in Subsection 2.5.2.

2.5.4.9 Earthquake Site Characteristics

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.9 with the following.

This subsection briefly summarizes the derivation of the site GMRS and Safe Shutdown Earthquake (SSE) that are detailed in Subsection 2.5.2.6.

The CPNPP Units 3 and 4 site is in a stable continent area with relatively low regional stress and low regional seismicity, as described in Subsections 2.5.1 and 2.5.2, and summarized in Subsection 2.5.4.1. Design ground motions are also relatively low.

A performance-based, site-specific GMRS was developed in accordance with the methodology provided in RG 1.208. This methodology and the GMRS are provided in Subsection 2.5.2.6. The GMRS satisfies the requirements of

10 CFR 100.23 for development of a site-specific SSE ground motion. The SSE is the envelope of the GMRS and the minimum earthquake requirements of 10 CFR Part 50 Appendix S, based on the shape of the Certified Seismic Design Response Spectra (CSDRS) scaled down to a PGA of 0.1g. The CSDRS for the US-APWR is a modified RG 1.60 shape formed by shifting the control points at 9 Hz and 33 Hz to 12 Hz and 50 Hz, respectively.

As recommended in RG 1.208, the following general steps were undertaken:

- Review and update the EPRI (1986) (Reference 2.5-369) seismic source model for the site region (200 mi radius), including updated characterization of the Meers fault, which represents the nearest active seismic source to the site
- Update the EPRI (1989) (Reference 2.5-370) ground motion attenuation model using the EPRI (2004) (Reference 2.5-401) ground motion attenuation model
- Perform sensitivity studies and an updated Probabilistic Seismic Hazard Analysis (PSHA) to develop rock hazard spectra and define the controlling earthquakes
- Derive performance-based GMRS from the updated PSHA at a free field hypothetical outcrop at the top of competent material beneath the site (defined as top of Glen Rose Formation Layer C)

The resulting GMRS and derivative FIRS are presented in Subsection 2.5.2.6.

2.5.4.10 Static Stability

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.10 with the following.

2.5.4.10.1 Bearing Capacity

Seismic category I and II structures for Units 3 and 4 are founded on mat foundations bearing directly on sound Glen Rose Formation limestone Layer C (Subsection 2.5.4.3), or concrete fill placed over limestone. Strength and compressibility properties for the Glen Rose Formation materials are discussed in Subsection 2.5.4.2. Extensive core borings and geophysical surveys performed throughout the CPNPP Units 3 and 4 seismic category I and II structure footprints demonstrate that the targeted Glen Rose Formation engineering Layer C limestone is approximately 60 ft thick below foundation subgrade elevation, massive, and highly uniform in characteristics. Average RQD of the limestone below the foundation subgrade is greater than 95 percent (Figure 2.5.4-240), and S-wave and P-wave velocities average over 5500 fps and 11,000 fps, respectively (Figure 2.5.4-239). The rock is horizontally to subhorizontally layered, and no significant voids, shears, or weak zones occur in the Layer C limestone that could form potential bearing sliding surfaces or differential settlement. The foundation

subgrade elevation of 782 ft provides deep confinement of the limestone of about 40 ft below plant grade, and no slopes or sloping rock surfaces exist around the Units 3 and 4 power blocks that could result in lateral confinement reduction.

Ultimate bearing capacity for both Units 3 and 4 seismic category I and II structures was estimated for three potential failure mechanisms of general shear failure, local shear failure, and compressive failure, as presented in the Rock Foundations Manual by the U.S. Army Corps of Engineers (COE, Reference 2.5-420).

The traditional Buisman-Terzaghi bearing capacity expression is used to calculate ultimate bearing capacity for the general shear failure condition, as shown below:

$$q_{ult} = cC_cN_c + 0.5\gamma BC_\gamma N_\gamma + \gamma DN_q$$

$$N_c = 2N_\phi^{1/2}\left(N_\phi + 1\right)$$

$$N_\gamma = N_\phi^{1/2}\left(N_\phi^2 - 1\right)$$

$$N_q = N_\phi^2$$

$$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Where:

q_{ult}	=	Ultimate bearing capacity
γ	=	Effective unit weight (i.e. submerged unit weight if below groundwater table) of rock mass
B	=	Width of foundation
D	=	Depth of foundation
c	=	The cohesion intercept for rock mass
ϕ	=	Angle of internal friction angle for rock mass
C_c	=	Foundation shape correction factor for N_c (see Table 6-1, Reference 2.5-420)
C_γ	=	Foundation shape correction factor for N_γ (see Table 6-1, Reference 2.5-420)
N_c, N_γ, N_q	=	Bearing capacity factors

Local shear failure is a case where a failure surface starts to develop but does not propagate to the surface. For this mode of failure, depth of embedment contributes little to the total bearing capacity. The expression for the ultimate bearing capacity applicable to localized shear failure is as follows:

$$q_{ult} = cC_c N_c + 0.5\gamma B C_\gamma N_\gamma$$

The parameters are the same as those defined for the general shear failure condition.

Compressive failure is a case characterized by a foundation that is supported on poorly constrained columns of rock, and the failure mode is similar to unconfined compression failure. The expression for the ultimate bearing capacity applicable to compressive failure is as follows:

$$q_{ult} = 2c\tan(45 + \frac{\phi}{2})$$

The parameters are the same as those defined for the general shear failure condition. Assuming ϕ=0, the ultimate bearing capacity for compressive failure is approximated by the unconfined compressive strength of rock mass (q_{ult}=2c).

COE recommends that the initial strength parameters selected for analysis should be based on lower bound estimates because rock masses generally provide generous margins of safety against bearing capacity failure. For a conservative estimation of the bearing capacity using the above procedures, the angle of internal friction is assumed to be zero and the cohesion is assumed to be one-half of the lower bound of the unconfined compression strength values.

Results of the bearing capacity analysis performed for main seismic category I and II structures (Table 2.5.4-228) indicate that the ultimate bearing capacity for foundations bearing in Glen Rose Formation engineering Layer C limestone is governed by the compressive failure mode and is at least 146 ksf. The estimated bearing capacity is compared to minimum bearing capacity values referenced in the US-APWR Key Site Parameters (DCD Table 2.0-1) that are 15 ksf static and 95 ksf dynamic. The estimated ultimate bearing capacity for engineering Layer C limestone provide factors of safety against bearing capacity failure of about 10 for static loading and at least 1.5 for seismic loading. The actual available factors of safety for specific structures (Table 3.8-202) are much higher than these levels and clearly indicate that the Glen Rose Formation engineering Layer C limestone provides adequate bearing capacity for support of the proposed structures.

Additional information and details regarding the procedure and results of the bearing capacity calculations are provided in the Settlement and Bearing Capacity report.

2.5.4.10.2 Settlement

As discussed in Subsection 2.5.4.3, massive and sound Glen Rose Formation engineering Layer C extends about 60 ft below foundation subgrade for seismic category I and II structures. Layer C is underlain by competent Glen Rose Formation engineering Layers D through F that consist principally of limestone with similar characteristics to Layer C, and interbedded indurated shale. As shown in Figure 2.5.4-240, the rock mass for a minimum distance of about 150 ft below foundation level is massive, and exhibits an average RQD greater than 95 percent. Settlement estimates are based on interpreted compressibility characteristics and elastic modulus properties of Glen Rose Formation limestone and shale materials, as discussed in Subsection 2.5.4.2. Elastic modulus values that were interpreted based on field and laboratory tests, were used to develop a "Best Estimate (BE)" as well as a "Lower Bound (LB)" modulus profile.

For the BE profile, the subsurface rock deformation characteristics were estimated using in situ S-wave velocities measured during the borehole suspension P-S logging. Because the borehole velocity measurements reflect the local influence of rock discontinuities and material variations, the resulting calculated modulus values are considered to be more indicative of the rock mass conditions. However, due to the low strain nature of the S-wave velocity, the calculated modulus is an upper bound case when used for settlement calculations. The low strain modulus values were then adjusted to reflect the relative higher strain levels anticipated for the fully loaded foundations. The modulus values developed based on this procedure are considered to represent the best estimated model for use in settlement analysis.

In situ rock modulus is estimated from the shear wave velocities using the following relationships:

$$G_{\max} = \frac{\gamma}{g} \cdot V_s^2$$

Where:

$G_{\max}$ = Low Strain Shear Modulus (psf)

V_s = Shear Wave Velocity (fps)

γ = Total Unit Weight (pcf)

g = Gravitational Acceleration Constant (32.2 ft/s^2).

Poisson's ratio (ν) is determined as follows:

$$\nu = \frac{V_p^2 - 2V_s^2}{2 \cdot \left(V_p^2 - V_s^2\right)}$$

Where:

ν = Poisson's ratio

V_p = Compression Wave Velocity (fps).

From the above information, the Modulus of Elasticity or Young's Modulus (E) is determined from:

$$E_{\max} = 2G \cdot (1 + \nu)$$

$$E = E_{\max}(RF)$$

Where:

$E_{\max}$ = Low Strain Modulus of Elasticity or Young's Modulus

E = Strain Adjusted Modulus of Elasticity or Young's Modulus

RF = Reduction Factor for Modulus Strain Adjustment

The low strain modulus (E_{max}) values were empirically reduced in order to develop a modulus model that is more compatible with the level of anticipated settlement. An iterative process was used between strain, calculated modulus, and settlement in order to select the appropriate reduction factor for each layer. A summary of the velocity data, Poisson's ratio values, calculated Modulus values, and the calculated BE modulus profile versus depth and engineering layers is presented in Table 2.5.4-226.

For the LB profile, the subsurface rock deformation characteristics were estimated using the results of stress-strain measurements in the laboratory on intact core samples, and in situ tests in boreholes using the pressuremeter. Because the individual core samples and pressuremeter tests do not consider the discontinuities or material variations, the Rock Mass Rating (RMR) System (Reference 2.5-409), and GSI System (References 2.5-421 and 2.5-422) were used with empirical approaches to incorporate the effects of discontinuities and material variations and assess the overall rock mass deformation characteristics. The modulus model developed based on this procedure is expected to produce a conservative lower bound modulus model for use in settlement analysis.

Laboratory test results from individual rock samples and the RMR and GSI values were used to estimate the deformation modulus of the rock mass by using empirical equations summarized by Hoek and Diederichs (Reference 2.5-422). Four empirical approaches recommended by Nicholson & Bieniawski (1990), Mitri et al. (1994), Sonmez et al. (2004), and Hoek & Diederichs (2006) were selected to define the Rock Mass Modulus range (Reference 2.5-422) for the CPNPP Units 3 and 4 site. The estimated range of the Rock Mass Modulus (E_{rm}) values for each of the stratigraphic layers, based on the above four correlations and their

average value, is presented on Figure 2.5.4-245. Modulus values from the field pressuremeter tests and the laboratory unconfined compression tests are also shown on Figure 2.5.4-245 for comparison. The average estimated rock mass modulus compare well with the lower bound of the intact modulus values from the laboratory or field measurements and is considered to be a reasonable representation of LB modulus profile for deformation characteristics of the site rock mass profile. Table 2.5.4-227 presents a summary of the calculated LB modulus profile and other pertinent data versus depth and engineering layers.

A summary of both BE and LB models used for the settlement calculations (i.e., the variation of elastic modulus versus elevation), the modulus values calculated directly based on in situ S-wave velocities, and pressuremeter and UC tests, are shown on Figure 2.5.4-241.

Due to the elastic nature of the subsurface rock materials, settlements from foundation loading are anticipated to be elastic in nature. Settlements are estimated by elastic theory using two methods of non-layered and layered systems. For the non-layered system, the subsurface rock layers supporting the foundations are considered to be a homogeneous elastic half-space medium with a uniformly loaded rectangular area.

The formulas by Schleicher (1926) are used to calculate the settlement of any location beneath a loaded rectangle foundation (Reference 2.5-437).

$$\delta_d(x,y) = C_s q B\left(\frac{1-\nu^2}{E}\right)$$

The parameter C_s is a geometric factor that accounts for the shape of the rectangle and the position of the point for which the settlement is being calculated. The formula for calculating C_s is as follows (Reference 2.5-437):

$$C_s = \frac{1}{2\pi}(C_1 + C_2 + C_3 + C_4)$$

$$C_1 = B_1 \ln\frac{\sqrt{A_1^2 + B_1^2} + A_1}{\sqrt{A_2^2 + B_1^2} - A_2}$$

$$C_2 = B_2 \ln\frac{\sqrt{A_1^2 + B_2^2} + A_1}{\sqrt{A_2^2 + B_2^2} - A_2}$$

$$C_3 = A_1 \ln\frac{\sqrt{A_1^2 + B_1^2} + B_1}{\sqrt{A_1^2 + B_2^2} - B_2}$$

$$C_4 = A_2 \ln \frac{\sqrt{A_2^2 + B_1^2} + B_1}{\sqrt{A_2^2 + B_2^2} - B_2}$$

$$A_1 = 1 - \frac{2x}{B}$$

$$A_2 = 1 + \frac{2x}{B}$$

$$B_1 = \frac{L}{B} - \frac{2y}{B}$$

$$B_2 = \frac{L}{B} + \frac{2y}{B}$$

Where:

$\delta_d(x, y)$	=	Settlement of the point with coordinates x and y
q	=	Uniform load intensity
C_s	=	Geometric factor
B	=	Width of the loaded area
L	=	Length of the loaded area
ν	=	Poisson's ratio
E	=	Average Elastic or Young's modulus
A_1, A_2	=	Factors to be calculated based on the above formulas and then inserted into the formulas for C_1 through C_4
B_1, B_2	=	Factors to be calculated based on the above formulas and then inserted into the formulas for C_1 through C_4
$C_1 - C_4$	=	Factors to be calculated based on the above formulas and then inserted into the main formula for C_s
x, y	=	Coordinates of the point

The average elastic modulus for the half-space was calculated using a weighted average modulus approach, as indicated by the following relationships (Reference 2.5-420):

$$E_{avg} = \frac{\sum_{i=1}^{n}\left(E_i \Big/ \sum_{j=1}^{i} h_j\right)}{\sum_{i=1}^{n}\left(1 \Big/ \sum_{j=1}^{i} h_j\right)}$$

Where:

E_{avg} = Weighted average modulus

E_i = Elastic modulus of each layer

h_j = Thickness of each layer

n = Number of layers

The layered method is similar to the non-layered method, but considers the subsurface rock materials supporting the foundations to be a layered system. The stress increase with depth caused by a rectangular uniform surface load is computed using a stress distribution theory. Superposition of rectangular areas covering the loaded surfaces is used in the cases where the stress calculation point is not located directly under the corner of a given loaded area or when there is more than one loaded area. The strain of each layer is calculated by dividing the stress increment by the layer modulus, and then the strain is multiplied by the layer thickness to provide the layer compression or settlement. The computed settlement values of all layers are summed to provide the total settlement values shown below:

$$\delta = \sum_{i=1}^{n} \delta_i = \sum_{i=1}^{n} \varepsilon_i h_i = \sum_{i=1}^{n} \frac{\Delta\sigma_i}{E_i^e} h_i$$

Where:

δ = Total Settlement

δ_i = Settlement of each layer

ε_i = Strain in each layer

h_i = Thickness of each layer

$\Delta\sigma_i$ = Stress increment in each layer due to loading

E_i^e = Equivalent elastic modulus of each layer

In the above formula for the equivalent elastic modulus (E_i^e), values of Young's modulus (E_i), plane strain modulus ($E_i^{'}$), or constrained modulus (M_i) may be used as defined below, depending on the boundary conditions or location of the settlement point (Reference 2.5-439).

$$E_i^{'} = \frac{E_i}{1 - \nu^2}$$

$$M_i = E_i\left[\frac{1 - \nu_i}{(1 + \nu_i)(1 - 2\nu_i)}\right]$$

Where:

E_i	=	Young's modulus of each layer
$E_i^{'}$	=	Plane strain modulus of each layer
M_i	=	Constrained modulus of each layer
ν_i	=	Poisson's ratio of each layer

For the cases where the foundation dimensions are relatively large, the lateral deformation at points below the center of the foundation is considered fully constrained and use of the constrained modulus is more appropriate. For the cases of small foundations or areas near corners or edges of large foundations, the lateral deformations are not constrained and the Young's modulus is more appropriate for settlement computations. For the settlement calculations provided herein, the plane strain modulus, which consider the strain to be constrained in only one direction, was adopted. The plane strain modulus, which is lower than the constrained modulus and slightly higher than the Young's modulus, is judged to be a reasonable selection and appropriate for representing all points below loaded areas for both large and small size foundations.

There are several elastic solutions that can be used to calculate stress distribution, such as Boussinesq, Mindlin, and Westergaard. There is no definitive proof that either of these solutions is more accurate than the other for soil or rock applications. Among the available solutions, the Boussinesq solution has been most widely used for geotechnical applications. It has also been found that settlements obtained through use of the Boussinesq equation are larger than the observed settlements in the great majority of cases. The Boussinesq solution was conservatively selected for computing the stresses distribution under the loaded areas for the settlement calculations. The Boussinesq equation for calculating vertical stress increment under a corner of a rectangular uniformly distributed flexible loaded area is expressed as follows (Reference 2.5-438):

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\frac{m^2+n^2+2}{m^2+n^2+1}+\sin^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right)\right]$$

$$m = \frac{L}{Z}$$

$$n = \frac{B}{Z}$$

Where:

σ_z	=	Stress increment at a depth z
q	=	Uniform load intensity as surface
B	=	Width of the loaded area
L	=	Length of the loaded area
Z	=	Distance below the loaded area
m,n	=	Ratio of loaded area width or length to depth

The vertical stress induced at other locations than the corner or by more than one foundation can be obtained through the superposition approach.

A summary of the results of the settlement and deformation analyses conducted by the non-layered and layered methods described above for the two BE and LB deformation modulus models are presented in Tables 2.5.4-229 and 2.5.4-230, respectively.

Estimated total settlements for seismic category I and II structures founded on Glen Rose limestone Layer C are estimated to be less than 1/2 in. Estimated differential settlement is not anticipated to exceed about 1/4 in across the foundation widths or around the perimeters of the structures. Settlement estimates assume excavation procedures do not affect integrity or compromise the load bearing capacity of limestone to any appreciable degree.

These estimated settlements are consistent with estimated settlements for foundations of CPNPP Units 1 and 2 supported in similar Glen Rose Formation limestone, as discussed in the FSAR (Reference 2.5-201). They conform to total and differential settlement criteria for the US-APWR Standard Design.

Additional information and details regarding the procedure and results of the settlement calculations are provided in the Settlement and Bearing Capacity report.

During construction and after the completion of Units 3 and 4, a number of settlement points or plates will be established on selected parts of the structures for settlement monitoring purposes during the life of the plant. The existing Maintenance Effectiveness Monitoring Program for Units 1 and 2 will also be adopted to carry on the monitoring program for Units 3 and 4.

2.5.4.10.3 Excavation Rebound Potential

As discussed in Subsection 2.5.4.1, regional stresses in the geologic formations at the CPNPP site are low, and significant stress relief during excavation is not expected. Rebound deformation estimates are carried out using a similar procedure as described in Subsection 2.5.4.10.2. The BE modulus profile was considered more applicable and therefore was used for the rebound estimates. Rebound deformation due to removal of about 40 ft of soil and rock material to the top of Layer C limestone rock is not anticipated to exceed about 1/8 in. A summary of the rebound estimates for the center points of the main structures is shown in Table 2.5.4-231. Based on these results, the potential for any significant heave or rebound of the foundation rock due to foundation excavation during the construction is considered very low.

The CPNPP Units 1 and 2 FSAR (Reference 2.5-201) discusses rock stress relief measurements associated with general plant site excavation recorded in two extensometers. A maximum rebound of 0.02 in was measured by the extensometers during deep excavation (approximately 30 ft to 60 ft) into upper Glen Rose Formation strata that are laterally contiguous with the rock strata that will be excavated for the CPNPP Units 3 and 4 plant site and seismic category I and II foundations. No occurrences of high stress or stress-induced instability are described.

Additional information and details regarding the procedure and results of the excavation rebound calculations are provided in the Settlement and Bearing Capacity report.

2.5.4.10.4 Lateral Earth Pressures

Lateral earth pressures acting on below-grade structures and walls are due to the self weight of backfill soils, backfill compaction, hydrostatic, surface (temporary or permanent) loads, and transient (seismic) loads.

Lateral active and at-rest earth pressures are calculated for select granular backfill, and are summarized on Figures 2.5.4-242 and 2.5.4-243, respectively. Lateral earth pressures acting on non-yielding walls (rigid and restrained from displacement and rotation), such as the seismic category I and II structures, are to be calculated for an at-rest condition. Other walls that are capable of yielding (including flexible or walls free to displace or to rotate at the top) are calculated for active conditions. Intermediate cases of lateral earth pressure may exist depending on the degree of rigidity, stiffness, and restraining characteristics of the wall system.

2.5.4.10.5 Resistance to Lateral Loads

Lateral loads can be resisted by an allowable passive soil pressure acting on the sides of the foundations. In addition, lateral loads may be resisted by friction acting along the side walls and the base of the foundation.

Ultimate passive pressures are calculated for select granular backfill and are summarized on Figure 2.5.4-244. The upper 2 ft of passive resistance should be neglected unless the soil is confined by pavement or slab.

For concrete tightly poured against firm foundation limestone bedrock (at approximate elevation 782 ft), base coefficient of friction of 0.6 is applicable for use between the base of concrete foundation and the limestone bedrock interface, or concrete foundation and concrete fill interface.The coefficient of friction is applied to net buoyant (dead, normal) loads for the portion of the structure that extends below the groundwater table.

All seismic category I and II structures are designed based on friction acting along the base of the foundations and by shear keys (if and where needed) for lateral sliding. No passive pressure or frictional resistance along the sides of the foundations or the below-grade structures are used for resisting lateral loads. Additional details are provided in Subsection 3.8.4.

2.5.4.11 Design Criteria

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.11 with the following.

Methods used to evaluate bearing capacity, settlement and lateral earth pressures are discussed in Subsection 2.5.4.10. Soil and rock properties used in the analyses are provided in Subsections 2.5.4.2 and 2.5.4.3.

The estimated ultimate bearing capacity suggests that minimum factors of safety against bearing capacity failure are approximately 10 for static loading and 2 for seismic loading condition. For all seismic category I and II structures, the foundations are founded in Layer C limestone. The estimated total settlements are generally less than 1/2 in, with differential settlements of up to about 1/4 in. Seismic category I and II structures are expected to experience settlements that are within the acceptable criterion.

Fill concrete material is required to meet the requirements as defined in Subsection 2.5.4.5.4.1.2.

The design criteria used for static stability analyses and settlement are identified in Subsection 2.5.4.10. Factors of safety estimates are applicable to the calculation of bearing capacity and sliding, and are discussed in Subsections 2.5.4.10.1 and 2.5.4.10.5, respectively. Discussion of assumptions and conservatism in static stability analyses are included in Subsection 2.5.4.10.

2.5.4.12 Techniques to Improve Subsurface Conditions

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.12 with the following.

No special techniques are required for improvement of foundation conditions, except where blasting, if used during construction, has shattered or fractured the rock adjacent to or within the excavated foundation area. To improve these areas of exception, dental concrete can be used to replace fractured rock and to fill overexcavated areas. In areas where fractures are present, but do not require removal, pipes can be installed to inject grout.

The geotechnical and geological quality control observation of excavation bottoms and faces determines if ground improvement is necessary prior to foundation installation.

2.5.4.12.1 Mechanical Cleanup

When suitable continuous rock is reached at or below the specified foundation elevation, the exposed rock surfaces require cleaning and preparation to receive fill concrete or foundation concrete. Cleaning and preparation of foundation materials consists of the following:

- Remove loose soil, rock, or other materials from the foundation surface.
- Remove protrusions and overhangs within the rock or concrete.
- Wash the exposed rock or concrete surface with air and/or water.
- Treat isolated depressions or cracks in the rock or concrete surface with fill concrete.
- Roughen exposed concrete surfaces.

2.5.4.12.2 Grouting and Concrete Dental Repair

As discussed in Subsection 2.5.4.3, the rock mass at and below foundation subgrade elevations is massive, slightly weathered to fresh, Glen Rose Formation engineering Layer C that does not contain significant voids, close fracture zones, or shears. Possible isolated and small zones of unusual deep weathering, close fracturing, or excavation blast damage are excavated to sound rock and treated with fill concrete. This treatment applies to localized zones greater than 3 ft in maximum dimension.

2.5.4.12.3 Rock Bolting/Anchoring

Although no rock bolting or anchoring was necessary during excavation of about 80 ft vertical cuts that were made for construction of Units 1 and 2, rock bolting or anchoring is beneficial to stabilize steep side slope rock excavations.

The site geologic investigation did not reveal any persistent jointing that is likely to provide planes of weakness for excavation sidewall failures. However, should stabilization be required before foundation excavations begin, rock bolts may be installed to improve the shear resistance along potential failure planes.

2.5.4.12.4 Foundation Improvement Verification Program

No foundation verification improvement testing program is required, as rock subgrade preparation is accomplished by means of removal of any and all unsuitable material.

Mapping of the all excavation surfaces is required as described in Subsections 2.5.4.5.4.2 and 2.5.4.5.4.4 prior to the foundation treatment, foundation installation, and placement of concrete fill material.

Geotechnical quality control includes foundation bottoms observation and evaluation in order to verify that 1) shale has been removed from the bottom of the seismic category I structures, 2) non-structural fill, if present, is removed down to bedrock or competent native material, and 3) any potentially adverse geologic feature (such as pervasive joints, cracks, or deeply weathered rock) is properly mitigated prior to and during foundation installation.

2.5.5 Stability of Slopes

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.5 with the following.

In conformance with Regulatory Guide (RG) 1.206, this subsection provides an evaluation of the static and dynamic stability of all natural and man-made earth and rock slopes that could adversely affect the safety of seismic category I and II structures for CPNPP Units 3 and 4. The slope evaluation incorporates site characterization information described in Subsection 2.5.4, and applies geologic- and geotechnical-based slope stability methodology in current practice for nuclear power projects. In general, all seismic category I and II structures within the nuclear islands are founded on stable and competent Glen Rose Formation limestone Layer C at about elevation 782 ft. The design of the Ultimate Heat Sinks (UHS) consists of reinforced concrete structures that are also founded on the Glen Rose Formation limestone Layer C, and does not include any earth embankments for side wall support. Geologic conditions, past slope performance, and slope stability analyses presented in this subsection indicate that a postulated failure of soil, fill, or rock materials above Layer C in any slopes in the vicinity of the plant would not adversely affect the safety or performance of seismic category I and II structures.

Temporary cuts below plant yard grade are required for construction of safety-related structures. However, all temporary cuts and excavations are backfilled with engineered fill up to plant yard grade level, and do not pose any post-construction or operational slope stability hazard. Temporary construction cut slopes are discussed in Subsection 2.5.4.5.

A map showing the locations of the proposed CPNPP Units 3 and 4 plant facilities, with respect to site setting, is shown on Figure 2.5.4-201. Safety-related seismic category I and II facilities are shown on Figure 2.5.4-216.

As specified in the DCD and RG 1.206 (pages C.I.2-35 to C.I.2-37), this subsection is organized into the following subsections:

- Slope Characteristics (2.5.5.1)
- Design Criteria and Analyses (2.5.5.2)
- Logs of Borings (2.5.5.3)
- Compacted Fill (2.5.5.4)

Slope stability analyses considered temporary and permanent loading conditions, pre- and post-construction topography (Figure 2.5.5-204), groundwater conditions described in Subsections 2.4.12 and 2.5.4.6, and seismic ground motions described in Subsection 2.5.2.

2.5.5.1 Slope Characteristics

2.5.5.1.1 Locations and Descriptions of Slopes

The CPNPP Units 3 and 4 plant yard area is a large (approximately 1700 ft by 1000 ft) level pad at elevation 822 ft (Figure 2.5.5-204). The pre-construction ground surface grade within the power block area of CPNPP Unit 3 varies in elevation between approximately 830 ft and 855 ft, and the grade within the power block area of the CPNPP Unit 4 varies in elevation from approximately 842 ft to 868 ft. Site grading to prepare the level yard grade involves general cut and excavation ranging between approximately 8 ft and 33 ft for CPNPP Unit 3 and from about 20 ft to 46 ft for CPNPP Unit 4.

The plant grade transitions into gently sloping natural and artificial ground along the west, south, and eastern margins of the pad. No slopes of significant gradient and/or height exist in these areas to present a potential slope stability issue. As shown on Figure 2.5.5-204, a combination of natural and graded slopes descends from the northern margin of the plant yard to SCR along the north margin of the plant site and in the area of the UHS. Reservoir pool elevation is 775 ft, and the side slopes rising above reservoir level to plant grade are between 40 ft and 45 ft high. The closest approach of these slopes to the plant power blocks are northeast of CPNPP Unit 3, and north to northwest of CPNPP Unit 4. The pre-construction slopes northeast of CPNPP Unit 3 have an overall maximum inclination of approximately 5(H):1(V), and those north and northwest of CPNPP Unit 4 have an overall maximum inclination of approximately 3(H):1(V). Some localized areas may have slightly steeper inclinations. Portions of the slopes also continue for some distance below the reservoir water level.

Table 2.5.5-201 provides a summary of the post-construction slopes and their pertinent data such as conditions, types, locations, heights, maximum inclinations, and their distances to seismic category I structures.

2.5.5.1.2 Past Slope Performance

There is no evidence of past significant landsliding within a 0.6 mi radius of the CPNPP Units 3 and 4 site, based on aerial photograph evaluation and field reconnaissance mapping. Intact outcropping strata of Glen Rose Formation bedrock are visible tracing along the topographic contour in the area of the reservoir slope on pre-reservoir and modern aerial photographs. Discrete bedrock strata of the Glen Rose Formation can be correlated with borings along the north margin of the plant site at expected elevations based on projections considering bedding dip (nearly flat). This correlation provides geologic evidence that the bedrock has not been displaced by past landsliding.

Localized surficial erosion and raveling has occurred in undocumented fill and/or native colluvial soils on the reservoir slopes. This is considered a routine/normal maintenance issue involving surficial conditions and does not present a significant slope stability hazard to the CPNPP Units 3 and 4 plant site.

Shale layers in the upper parts of the Glen Rose Formation (within engineering Layers A and B; Subsection 2.5.4.3) daylight in the reservoir slopes, as shown on pre-construction Cross Sections D-D' and E-E' (Figures 2.5.5-202 and 2.5.5-203), above reservoir pool level. The dip of the shale beds is near-horizontal, a geometry that is favorable for slope stability and helps limit the size of potential slope failures. Although significant sliding has not occurred to date or during the geologic history of the slopes, and the bedding dip is generally favorable for stability, the beds represent weaker zones in the rock mass that could act as a potential sliding surface, especially if softened by perched groundwater conditions. Stability analysis in Subsection 2.5.5.2 evaluates the long-term slope stability safety factors for this potential failure mode with respect to the UHS structures in proximity to the reservoir slopes. Massive, stable limestone of Glen Rose Formation engineering Layer C daylights in the reservoir slope slightly above the pool elevation. This limestone is resistant to sliding, and constrains the depth and toe locations of possible slope failure. Slope failure in limestone at or below the reservoir pool elevation is not likely to occur.

Potential sliding along shallow bedrock shale beds in Glen Rose Formation Layers A and B would not affect the stability of power block facilities for CPNPP Units 3 and 4 because these structures are founded on Glen Rose Formation Layer C limestone below the shale layers, and are set back a considerable distance from the reservoir slopes.

Thick, undocumented fill in former topographic swale areas north and east of the CPNPP Units 3 and 4 power block footprints (Figures 2.5.4-212 and 2.5.4-215) extends to the margin of SCR, and forms localized portions of the reservoir slopes. The fill bodies appear to be in hydraulic communication with the reservoir. As a result, groundwater occurs as a perched condition in the swale fill, at higher elevations than encountered in the bedrock surrounding the filled swale areas. Fill in the eastern swale area has undergone differential settlement, indicated by ground surface cracks and depressed areas. Sliding failure of undocumented fill over native soils, bedrock, or along failure planes in the fill are modeled by slope stability analysis in Subsection 2.5.5.2. Stability analyses in Subsection 2.5.5.2 evaluate the stability safety factors for this potential failure mode with respect to the UHS structures in proximity to the reservoir slopes and fill areas.

Potential fill sliding would not affect the power block facilities for CPNPP Units 3 and 4 because these structures are founded on competent limestone below the elevation of fill, and are set back from the fill and reservoir slope areas.

Existing permanent slopes associated with the CPNPP Units 1 and 2 include artificial cuts at the intake and discharge structures on the shore of SCR and road cuts. These slopes are made largely in Glen Rose Formation limestone and shale bedrock, but also include engineered fill slopes. Slope heights are typically on the order of about 5 ft to 25 ft, and are inclined between about 2(H):1(V) to near-vertical. Field observations indicate that the existing slopes are generally stable and have performed well since construction that typically occurred 20 to 30 years ago.

The generalized stratigraphy and assignment of engineering layers adopted for use in the slope stability analyses are based on the site geologic-engineering model/profile presented in Subsection 2.5.4.3.

2.5.5.2 Design Criteria and Analyses

2.5.5.2.1 Analysis Cross Sections

Slope stability analyses were performed for the four most critical post-construction slope sections along the reservoir margin identified on the basis of grading plan inspection. The selected analysis locations include the maximum slope inclinations and permanent slopes at, or in the vicinity of, the UHS structures. The four analyzed cross sections are labeled D-D', E-E', E1-E1', and F-F', and their locations are shown on Figure 2.5.5-201. The cross sections are described below:

- Cross Section D-D' (Figure 2.5.5-205) - This post-construction cross section is oriented roughly north-south and is located northwest of Unit 4, passing through the western UHS unit from plant yard grade into the SCR. Cross Section D-D' ranges in elevation from 815 ft to 758 ft with a resulting total height difference of approximately 57 ft (40 ft above reservoir pool elevation of 775 ft). This cross section contains two breaks in slope at approximately elevation 795 ft and elevation 780 ft. Maximum gradients above, between, and below the slope breaks are approximately 2(H):1(V) (compacted fill over shale slope), 3.5(H):1(V) (compacted fill over limestone slope), and 0.75(H):1(V) (limestone slope within SCR), respectively.

- Cross Section E-E' (Figure 2.5.5-206) - This post-construction cross section is oriented in a southwest-northeast direction and is located northeast of CPNPP Unit 3, passing through the eastern UHS unit into SCR and through an intervening retaining wall structure. Cross Section E-E' ranges in elevation from 820 ft to 758 ft, with a resulting total height difference of approximately 62 ft (45 ft above reservoir pool elevation of 775 ft). This section contains two breaks in slope at approximately elevation 795 ft and elevation 775 ft. Maximum gradients above, between, and below these slope breaks are approximately 2(H):1(V) (compacted fill over shale and limestone slope), vertical (15-foot-high retaining wall), and 4(H):1(V) (limestone slope within SCR), respectively.

- Cross Section E1-E1' (Figure 2.5.5-207) - This post-construction cross section is oriented similarly to Cross Section E-E', but is located slightly to the east in order to capture areas adjacent to the end of the UHS retaining wall. Cross Section E1-E1' ranges in elevation from 820 ft to 758 ft, with a resulting total height difference of approximately 62 ft (45 ft above reservoir pool elevation of 775 ft). This cross section contains one break in slope at approximately elevation 790 ft. Maximum gradients above, and below the slope break are approximately 2(H):1(V) (compacted fill over

shale and limestone slope), and 4(H):1(V) (limestone slope within SCR), respectively.

- Cross Section F-F' (Figure 2.5.5-208) - This cross section is oriented in a southeast-northwest direction, and passes through undocumented fill between CPNPP Units 3 and 4 and into SCR. Cross Section F-F' ranges in elevation from 829 ft to 744 ft, with a resulting total height difference of approximately 75 ft (54 ft above reservoir pool elevation of 775 ft). This section contains one break in slope at approximately elevation 782 ft. Maximum gradients above and below that slope break are approximately 3(H):1(V) (existing undocumented fill slope) and 2(H):1(V) (limestone within SCR), respectively.

The cross sections show the post-construction site grading as interpreted from the site grading plans, the interpreted vertical and lateral extent of the surficial soils, and the depth to various bedrock layers. Based on the site grading plans (Figure 2.5.5-204), engineered compacted fill is placed on the reservoir side of the UHS units, as shown on post-construction Cross Sections D-D', E-E', and E1-E1' (Figures 2.5.5-205, 2.5.5-206, and 2.5.5-207). Where the toe of the compacted fill slopes projects into SCR, a retaining wall is used to keep the toe of the slope a safe distance away from the reservoir shoreline, as shown on post-construction Cross Section E-E' (Figure 2.5.5-206).

2.5.5.2.2 Subsurface Materials and Properties

2.5.5.2.2.1 Subsurface Stratigraphy

Subsurface stratigraphy for the analysis sections is based on the site exploration and test results presented in Subsection 2.5.4.2 and geologic-engineering layers described in Subsection 2.5.4.3. The inferred subsurface stratigraphy for the steepest slope section northwest of CPNPP Unit 4 is shown on Figure 2.5.5-202, and that for the steepest slope northeast of CPNPP Unit 3 is shown on Figure 2.5.5-203.

The locations of the field exploration, in situ tests, and geophysical surveys performed in the area of CPNPP Units 3 and 4 are shown on Figure 2.5.5-201. Engineering strength properties for the cross sections are based on the interpretation of field and laboratory test data, and are presented in Table 2.5.5-202 and shown on Figures 2.5.4-235 and 2.5.4-237 for shale and limestone materials, respectively. The basis for selection of these properties is discussed in Subsection 2.5.4.2.3.2.

The subsurface materials encountered within the project site that are relevant to the slope stability analysis include fill soils, residual soil, and the Glen Rose Formation bedrock. The Glen Rose Formation within the depth of the stability analysis models consists of interbedded limestone and shale. The bedrock is generally overlain by fill or residual soil that varies in thickness from a few feet to a few tens of feet.

2.5.5.2.2.2 Residual Soil

Residual soil materials range from sand and gravel with varying amounts of fines, to silt and lean sandy clay. Shear strength for the residual soil is based on soil descriptions, Standard Penetration Test (SPT) blow counts from exploratory borings, Cone Penetration Test (CPT) soundings, and empirical correlations.

Results of Atterberg Limits tests performed on silty and clayey residual soil samples were used in conjunction with an ultimate strength correlation published by Stark et al. (References 2.5-413 and 2.5-414) for fully-softened materials. The fully-softened (ultimate) strength was selected to perform the slope stability analysis. The undrained shear strength of fine-grained residual soils was empirically estimated from CPT sounding results, and ranged from approximately 0.5 to 3.5 kips per square foot (ksf). The effective friction angles for granular residual soils were estimated based on the CPT sounding test results, and ranged from about 35 to 45 degrees.

2.5.5.2.2.3 Undocumented Fill

Undocumented fill in swale areas is quite heterogeneous and variable in composition, including layers and zones of granular soil intermixed with fine-grained soil.

The consistency of the granular materials ranges from loose to medium dense, and that of fine-grained materials ranges from soft to stiff. The effective friction angles for granular materials are estimated to range from approximately 30 to 45 degrees, based on the CPT sounding results. The undrained shear strength of the fine-grained materials was empirically estimated from the CPT sounding results, and ranged from approximately 1.5 to 2.5 ksf.

2.5.5.2.2.4 Compacted Fill

The selection, placement, and compaction of the new compacted (engineered) fill are in accordance with the project plans and specifications. The drained Mohr-Coulomb shear strength parameters for the compacted fill require a minimum effective friction angle of 32 degrees and a minimum cohesion value of 200 psf, as indicated in Subsection 2.5.4.5.4.

2.5.5.2.2.5 Glen Rose Formation Shale

Shale bedding is essentially horizontal at the site, as described in Subsection 2.5.4.1. Along-bedding, fully-softened drained shear strength parameters for shale were estimated based primarily on laboratory direct shear test results, as shown on Figure 2.5.4-235. The plot shows the range of the ultimate (fully-softened) shear strength values, with a shaded zone identifying the most likely lower- and upper-bound values. Based on borehole core samples and field outcrop exposures, the shale appears to be consolidated and does not show extensive or persistent shear fabric or slickensides. As discussed in Subsection

2.5.5.1.2, no evidence of past landsliding was observed in the CPNPP Units 3 and 4 site area. Therefore, the fully-softened (ultimate) strength criterion is considered more appropriate for stability analysis than residual strength values that are warranted only if pre-existing sheared sliding planes are present.

For comparison purposes, the procedure developed by Stark et al. (References 2.5-413 and 2.5-414) was also used to estimate the range of fully-softened shear strength of shale based on correlations with liquid limit and clay fraction index test results. The Stark et al. correlations were developed based on comparison of the results of torsional ring shear tests with the rock index property tests for soft rock materials that have not previously undergone substantial shearing. The upper- and lower-bound fully-softened shear strength envelopes estimated by the Stark et al. procedure are also shown on Figure 2.5.4-235. The shear strength envelopes estimated from Stark et al. are generally comparable with the lower-bound fully-softened envelope from direct shear tests presented on Figure 2.5.4-235.

The curved lower-bound shear strength envelope from the direct shear test results, as shown on Figure 2.5.4-235, was conservatively selected for stability analyses of the permanent slopes. This non-linear lower-bound failure envelope was then used directly in the slope stability analysis.

2.5.5.2.2.6 Glen Rose Formation Limestone

Glen Rose Formation limestone typically is massive and well-cemented, and it exhibits brittle hard rock strength properties. The shear strength parameters for limestone were derived from laboratory unconfined compression test results that were modified to account for rock mass properties using published strength correlations initially developed by Hoek and Brown (References 2.5-409 and 2.5-410), and subsequently refined to include rock mass disturbance factors (from blasting and stress relief) by Hoek et al. (Reference 2.5-411). The Hoek-Brown criteria consider the scale effect of potential rock mass failure and the weakening influence of joints and other discontinuities in the rock mass. To develop a range of strength values, each unconfined compression test value was used to develop a Hoek-Brown shear-strength vs. normal-stress curve. The range of rock strength envelopes was used to estimate the limestone shear strength, as shown on Figure 2.5.4-237. The lower-bound Hoek-Brown shear strength envelope curve was selected as a conservative strength model for the in situ limestone rock mass. The lower-bound envelope was then used in the slope stability program to estimate the shear strength as a function of effective normal stress.

2.5.5.2.3 Groundwater

Groundwater within the existing fill is controlled by the water level in the adjacent SCR. According to the USGS, the pool elevation of the SCR is normally about elevation 775 ft, and has historically fluctuated between elevations 773 ft and 778 ft. Filled swale areas northeast of CPNPP Unit 4 and east of Unit 3 extend to the reservoir shoreline. The fill appears to be in hydraulic communication with the

reservoir, and a perched groundwater table at, or near, the elevation of the reservoir pool exists in the fill. According to the preliminary results from monitoring of field piezometers within the Units 3 and 4 area, the piezometric levels range between about elevation 775 ft and 858 ft, although some wells remain dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site. Groundwater and hydrogeologic conditions of the site are discussed in detail in Subsection 2.4.12.

For the purposes of modeling the slope stability, the groundwater table was conservatively assumed to be at elevation 780 ft.

2.5.5.2.4 Slope Stability Analysis Methodology

The slope stability analyses were performed for static and dynamic (pseudo-static) loading conditions. The latter analysis was performed using both horizontal and vertical seismic coefficients.

Conventional two-dimensional limit-equilibrium analyses were performed considering permanent (long-term) slope stability conditions.

Various methods of analysis, including Janbu and Bishop's (References 2.5-428, 2.5-429, and 2.5-430), were used for initial screening of possible failure surface geometries. Various failure surface shapes were considered, including Rankine-type, random block, and circular surfaces. Refined analyses were performed using Spencer's method (Reference 2.5-431) on targeted failure surfaces identified by the screening analysis. Spencer's method is considered more appropriate as it satisfies both force and moment equilibrium.

Soil and rock materials that exhibit anisotropic shear strength properties are more appropriate to be modeled by assigning Mohr-Coulomb strength parameters with two sets of shear strength parameters: "along" and "across" bedding. For conservatism, only along-bedding shear strength parameters of the shale were used in the stability analysis of permanent slopes (Subsection 2.5.5.2.2.5). This approach was used to model the Glen Rose Formation shale beds. Hoek-Brown criteria for rock-mass shear strength parameters were used to model the massive Glen Rose Formation limestone.

The computer program Slope/W 2007 (Geo-Slope International) was used to perform the slope stability analyses. This program models heterogeneous soil types, soil and rock anisotropy, complex stratigraphic and slip surface geometry, and variable pore water pressure conditions. The program was validated and verified for these analyses.

2.5.5.2.5 Dynamic Slope Stability

A pseudo-static method of analysis was used for stability evaluation of the slopes at the project site. In this method, the effects of seismic loading conditions on the slopes are accounted for through the application of constant horizontal and vertical seismic coefficients to the slope and computation of a pseudo-static factor of safety. With the conservative assumption of vertical-to-horizontal ratio of 1.0 the magnitude of the vertical coefficient is taken equal to the horizontal PGA. Both positive (downward) and negative (upward) vertical coefficients were considered. The orientation resulting in the lower factor of safety is considered the critical condition. If pseudo-static slope stability analyses, in which the horizontal and vertical seismic coefficients are taken equal to the PGA, result in factors of safety greater than 1.1, seismic slope performance is considered acceptable.

Ground motion and site response analyses discussed in Subsection 2.5.2 indicate that the horizontal PGA corresponding to the GMRS and FIRS1 at the CPNPP Units 3 and 4 site is about 0.045g. Horizontal PGA corresponding to the other FIRS are all below 0.07g, as shown on Figures 2.5.2-234 and 2.5.2-239. Therefore, the US-APWR DCD minimum PGA of 0.10g is used as the design PGA for both the horizontal and vertical seismic coefficients used in the slope stability modeling.

2.5.5.2.6 Analyses

Each subsection was analyzed for the following conditions using Spencer's method. Permanent slopes at the site were considered, and analyses were performed for the cases of circular (rotational), block/wedge (translational), or random potential failure modes as follows:

- Global (deep-seated) stability conditions
- Surficial stability conditions
- Pseudo-static (seismic or transient) loading conditions

Surficial stability of the 2(H):1(V) compacted fill slopes was also analyzed using the procedure developed by the U.S. Army Corps of Engineers (Reference 2.5-426) for both the static and pseudo-static loading conditions.

External loading conditions modeled in the slope stability analyses consisted of structural loads, traffic loads, and earthquake loads. Traffic and construction loads were modeled on top of the fill slopes, assuming a uniform surcharge pressure of 250 psf.

The following minimum factors of safety were established for this analyses based on the U.S. Army Corps of Engineers' Slope Stability Manual (Reference 2.5-426):

- Static Long-Term Factor of Safety: 1.5

- Static Temporary Factor of Safety: 1.3
- Pseudo-static Factor of Safety: 1.1

2.5.5.2.7 Results

The results of slope stability analyses of the permanent slopes indicate acceptable static long-term and pseudo-static factors of safety with values greater than 1.5 and 1.1, respectively, as summarized in Table 2.5.5-203. Example slope stability sections showing final critical circles and factors of safety are included as follows:

- Figures 2.5.5-209, 2.5.5-210, 2.5.5-211, and 2.5.5-212 for static global stability of permanent slopes, including Cross Sections D-D', E-E', E1-E1', and F-F' through Units 3 and 4, and the area between them, respectively.
- Figures 2.5.5-213, 2.5.5-214, 2.5.5-215, and 2.5.5-216 for seismic global stability of permanent slopes, including Cross Sections D-D', E-E', E1-E1', and F-F', respectively.

The results of the surficial stability for 2(H):1(V) compacted fill slopes also indicate that the engineered compacted fill slopes do have adequate surficial slope stability factors of safety, provided that the compacted fill materials exhibit the specified effective cohesion value of at least 200 psf, and an effective friction angle of at least 32 degrees, in accordance with the engineered fill specification.

Factors of safety are summarized in Table 2.5.5-203. The estimated factors of safety for permanent slopes satisfy the minimum required value.

Pseudo-static factors of safety were estimated using horizontal and vertical acceleration coefficients equal to 0.1g. The resulting factors of safety range between 1.47 and 1.96 (Table 2.5.5-203) and are considerably greater than the required minimum value of 1.1. These results demonstrate that the seismic performance of analyzed slopes is acceptable and that no seismically induced permanent slope displacement is expected at CPNPP Units 3 and 4 site.

A liquefaction potential evaluation, as discussed in Subsection 2.5.4.8, indicates that the native rock material supporting all seismic category I and II structures and the engineered compacted fill surrounding the structures are not susceptible to soil liquefaction and there is no impact on any safety related structures.

The post-construction cut slopes around the west and south periphery of the CPNPP Units 3 and 4 site presented in Table 2.5.5-201 and shown on Figure 2.5.5-204, are not considered to pose any slope stability issues or hazards to seismic category I and II structures. The closest approach between the toe of the cut slopes and seismic category I or II structures is approximately 150 ft, with a minimum ratio of at least three times the height of slope, providing a substantial safety setback from the cut slopes. Additionally, the inclination of cut slopes is

generally 2(H):1(V) or flatter. Considering the strength properties of the materials comprising the cut slopes (residual soil over Glen Rose Formation rock) and the maximum inclination of 2(H):1(V), all these cut slopes are considered to be inherently stable.

All safety-related plant structures are supported by foundations bearing into the competent Glen Rose Formation Layer C limestone below the plant grade at elevation of about 782 ft, and do not use any of the adjacent slopes or embankments for support. As a result, embankments or fill slopes around the perimeter of the plant do not affect the stability or performance of the safety-related structures.

2.5.5.3 Logs of Borings

The slope stability analyses incorporated relevant exploratory boring information, and derivative laboratory test data from these borehole samples, as described in Subsection 2.5.4.2. Figure 2.5.5-201 shows the distribution of exploratory borings with respect to the slope stability analysis cross sections.

Subsection 2.5.4.3 presents the CPNPP Units 3 and 4 geologic-geotechnical model integrated from all site explorations. This model was adopted for the slope stability analysis. All boring logs for CPNPP Units 3 and 4 explorations, including the subset of borings near the analysis sections considered for the slope stability evaluation, are summarized and included in the project boring log data report.

2.5.5.4 Compacted Fill

The slope stability analyses described in Subsection 2.5.5.2 included engineered, compacted permanent fill slopes. Based on comparison of pre- and post-construction ground surface elevation contours, as shown on Figure 2.5.5-204, portions of the slopes along the north boundary of the Units 3 and 4 consist of engineered compacted fill slopes. Figures 2.5.5-205, 2.5.5-206, and 2.5.5-207 show geometric conditions for the compacted fill slopes along the north boundary of the site. Specific sources of borrow material for the construction of the permanent fill slopes were not identified during the exploration program. However, portions of the on-site cut materials are expected to be acceptable for fill slope construction, provided that the excavated materials are segregated and/or processed properly to meet the minimum property requirements. Subsection 2.5.4.5 describes the specific property requirements, site preparation and fill placement, compaction requirements, and Quality Control and Quality Assurance requirements, as well as the testing and monitoring that is required to ensure proper verification and installation of engineered compacted fill.

2.5.6 Combined License Information

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.6 with the following.

2.5(1) Seismic and Geological Characteristics of the Site and Region

This COL item is addressed in Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4 and 2.5.5 along with the associated tables and figures.

2.5.7 References

CP SUP2.5(1) Add the following references after the last DCD reference.

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Table 2.5.1-201 (Sheet 1 of 2)
Radiocarbon Ages Used to Constrain the Timing of Events on the Meers Fault

Site	Sample	Radiocarbon Age (yrs B.P.)	Calibrated Age (cal yrs B.P.)
		Madole (1988)	
Browns Creek	DIC-3165	70 +/- 150	NA
Browns Creek	W-5533	310 +/- 150	NA
Browns Creek	W-5540	470 +/- 150	NA
Browns Creek	DIC-3166	13670 +/- 120	NA
Browns Creek	DIC-3169	1360 +/- 100	NA
Browns Creek	W-5543	1740 +/- 200	NA
Canyon Creek	DIC-3179	9880 +/- 160	NA
Canyon Creek	DIC-3170	12240 +/- 240	NA
Canyon Creek	DIC-3161	600 +/- 50	NA
Canyon Creek	DIC-3167	1280 +/- 140	NA
		Crone and Luza (1990)	
Canyon Creek	DIC-3183	1360 +/- 50	1290 +80/-110
Canyon Creek	DIC-3180	1660 +/- 50	1570 +/- 120
Canyon Creek	DIC-3266	1730 +/- 55	1646 +144/-126
Ponded Alluvium	PITT-0339	1480 +/- 35	1354 +64/-49
Ponded Alluvium	PITT-0340	1640 +/- 50	1539 +155/-129
Ponded Alluvium	PITT-0114	1865 +/- 25	1816 +60/-72
		Swan et al. (1993)	
NW Ponded Alluvium	PITT-0380	1270 +/- 25	1238 +44/-89
NW Ponded Alluvium	PITT-0381	1295 +/- 50	1265 +45/-180
NW Ponded Alluvium	PITT-0379	1565 +/- 45	1484 +76/-134
NW Ponded Alluvium	PITT-0378	1950 +/- 40	1912 +127/-92
SE Ponded Alluvium	PITT-0480	1120 +/- 60	1053 +117/-123
SE Ponded Alluvium	PITT-0489	1225 +/- 30	1167 +97/-102

CP COL 2.5(1)

**Table 2.5.1-201 (Sheet 2 of 2)
Radiocarbon Ages Used to Constrain the Timing of Events on the Meers Fault**

Site	Sample	Radiocarbon Age (yrs B.P.)	Calibrated Age (cal yrs B.P.)
SE Ponded Alluvium	PITT-0481	1445 +/- 45	1336 +74/-49
SE Ponded Alluvium	PITT-0479	1720 +/- 60	1669 +141/-149
SE Ponded Alluvium	PITT-0478	2105 +/- 55	2093 +216/-153
SE Ponded Alluvium	PITT-0477	3185 +/- 50	3397 +148/-48
SE Ponded Alluvium	PITT-0475	5180 +/- 60	5943 +228/-117
SE Ponded Alluvium	PITT-0476	5940 +/- 40	6836 +53/-113
SE Ponded Alluvium	PITT-0482	755 +/- 60	684 +96/-106
Valley Site	PITT-0375	1105 +/- 80	1296 +236/-84
Valley Site	PITT-0372	1380 +/- 60	1296 +94/-114
Valley Site	PITT-0369	1705 +/- 30	1610 +96/-110
Valley Site	PITT-0370	1990 +/- 45	1942 +113/-80
Valley Site	PITT-0373	2795 +/- 40	2918 +74/-125
Valley Site	AA-4093	865 +/- 95	777 +183/-13
Valley Site	PITT-0368	865 +/- 60	777 +183/-97

(Reference 2.5-283), (Reference 2.5-284), (Reference 2.5-285)

CP COL 2.5(1)

Table 2.5.1-202
Summary of Meers Fault Characterizations From Existing Literature

	Ramelli and others	Madole	Crone and Luza	Swan and others
Age of events				
Young Holocene	Within several thousand years	1280 years B.P. (uncalibrated C-14 age)	1200 to 1300 cal. years B.P.	1300 to 1400 cal. years B.P.
Old Holocene	NA	NA	NA	2100 to 2900 cal. years B.P.
Pre-Holocene	NA	NA	Greater than 100,000 years B.P.	Greater than 200,000 to 500,000 years B.P.
Style of faulting	Left oblique slip with lateral to vertical ratio of 2:1 to 4:1	NA	Left oblique slip with lateral to vertical ratio of 1.6:1 to 3.3:1	Left oblique slip with lateral to vertical ratio of 1.3:1
Length of surface rupture	37 km	NA	26 to 37 km	26 to 37 km
Event displacement	NA	NA	3.1 to 5.9 m	Average 1.75 to 3 m; maximum 3.5 to 5.25 m
Slip rate				
Holocene	NA	NA	NA	1 to 5 mm/yr
Quaternary	NA	NA	NA	10^{-4} to 10^{-5} mm/yr
Clustered behavior	NA	NA	NA	Yes, cannot assume out of cluster
Event magnitude	Ms 6.75 to 7.25	NA	Approximately Ms 7	Ms 6.75 to 7.25

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Table 2.5.1-203
Summary of Experts' Responses to Questions Regarding the Meers Fault

Question	Keith Kelson	Kathryn Hansen & Dr. Frank Swan	Dr. Anthony Crone	Alan Ramelli	Dr. Ken Luza
Active	Yes	Yes	Yes	Yes	Yes
Line source	Yes	Yes	Yes	Yes	Yes
Mmax	Take from Kelson and Swan (1990) and Swan et al. (1993)	Take from Swan et al. (1993)	Take from Crone and Luza (1990)	Take from Ramelli and Slemmons (1990) and Ramelli et al. (1987)	Recalled a magnitude 6, but wasn't sure.
Mmax methodology	Fault magnitude regressions	Fault magnitude regressions	Take from Crone and Luza (1990)	Fault magnitude regressions, but thinks these will underestimate magnitude	Fault magnitude regressions
Recurrence model	Characteristic with clustered behavior	Characteristic model	NA	Characteristic with clustered behavior	NA
In cluster	Likely	Likely	Difficult to judge	Difficult to judge	No. Only thinks fault has one Holocene event and long (>100,000 year) return period
Recurrence methodology	Paleoseismic data from trenching studies	Paleoseismic data from trenching studies	NA	NA	Use data from Madole (1988)

NOTES:

1. Preferred values identified by the study authors are given when available; otherwise the range of possible values from the study is presented.
2. NA - indicates that the study did not address a topic.

(Reference 2.5-271), (Reference 2.5-283), (Reference 2.5-284), (Reference 2.5-285), (Reference 2.5-286), (Reference 2.5-287), (Reference 2.5-277)

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Table 2.5.1-204 (Sheet 1 of 15)
Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
03530108	Shut-In Well (Type Gas)	32.1537217, -97.7179818	5266	GAS, on schedule	9	215734	XTO ENERGY INC. [945936]	SETH ELEVEN	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 07/2005	ongoing as of 06/2007	NA	0	0
14330398	Plugged Gas Well	32.3455909, -98.0272618	5138	GAS, not on schedule	7B	107503	DERNICK RESOURCES, INC. [216477]	THREE CIRCLE RANCH	MORGAN MILL (MARBLE FALLS)	4535-4836	L 4	completion date 02/20/1979	plugged / abandoned 12/30/1984	5138	0	0
14330412	Gas Well	32.3045384, -97.9944132	5353	GAS, not on schedule	7B	107502	DERNICK RESOURCES, INC. [216477]	THREE CIRCLE RANCH	MORGAN MILL (MARBLE FALLS)	4447-4767	P 10	completion date 02/19/1979	plugged / abandoned 12/28/1983	5353	0	0
14330477	Gas Well	32.3551503, -98.0102421	4850	GAS, on schedule	7B	084595	MARSHALL, SAM [527835]	DEAVER, A. UNIT	MORGAN MILL (MARBLE FALLS)	4625-4698	1	completion date 06/04/1979	ongoing as of 06/2007	NA	13293	46
22130056	Plugged Gas Well	32.4193948, -97.8351171	4972	GAS, not on schedule	7B	111288	POPE, W. E. [671410]	SCOTTISH RITE CRIPPLED CHILDREN	LIPAN (MARBLE FALLS)	3436-3886	1	completion date 06/20/1975	plugged / abandoned 07/20/1987	4975	0	0
22130057	Plugged Gas Well	32.4219411, -97.9478268	4135	GAS, not on schedule	7B	058956	BEACON ENERGY CORPORATION (1) [59101]	MARSHALL CLYDE JR	LIPAN (STRAWN)	2448-3757	2	completion date 06/28/1974	plugged / abandoned 03/28/2002	4135	12169	0
22130065	Gas Well	32.2995439, -97.872856	5010	GAS, on schedule	7B	059670	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4181-4182	1 A	completion date 08/07/1974	ongoing as of 06/2007	NA	173502	0
22130090	Plugged Gas Well	32.4142997, -97.9546314	4815	GAS, not on schedule	7B	066862	GRYNBERG, JACK J. [337114]	BRAZELL	LIPAN (STRAWN)	3950-3966	1	completion date 09/24/1975	plugged / abandoned 10/18/1984	4815	10184	39
22130091	Plugged Gas Well	32.4048361, -97.9046741	4880	GAS, not on schedule	7B	066297	SMITH PIPE OF ABILENE [794745]	JACOBS	CHAMPION (STRAWN SAND)	4417-4453	1	completion date 10/24/1975	plugged / abandoned 05/06/1994	4880	180	0
22130097	Plugged Gas Well	32.4173917, -97.9425021	4199	GAS, not on schedule	7B	067570	NICHOLS OIL & GAS CORPORATION [610380]	MARSHALL CLYDE JR.	LIPAN (STRAWN)	3788-3952	2 A	completion date 06/08/1976	plugged / abandoned 10/07/1995	4199	35615	0
22130098	Plugged Gas Well	32.4163952, -97.9514113	4199	GAS, not on schedule	7B	067569	BEACON ENERGY CORPORATION (1) [59101]	MARSHALL CLYDE JR.	LIPAN (STRAWN)	3833-4059	1 A	completion date 06/08/1976	plugged / abandoned 05/21/2002	4199	0	0
22130101	Plugged Gas Well	32.4104657, -97.9025219	5000	GAS, not on schedule	7B	068389	SMITH PIPE OF ABILENE [794745]	GILLIAM J. W.	CHAMPION (STRAWN SAND)	2951-3373	1	completion date 04/15/1976	plugged / abandoned 05/05/1994	5000	0	0
22130102	Plugged Gas Well	32.4181358, -97.960141	4200	GAS, not on schedule	7B	069005	GUARDIAN LEASING CO. [337103]	CAMPBELL O.H.	LIPAN (STRAWN)	3883-3993	1	completion date 09/07/1976	plugged / abandoned 09/09/1987	4200	0	0
22130103	Plugged Gas Well	32.4077394, -97.9599368	no record found	GAS, not on schedule	7B	068999	E.G. OPERATING [238558]	BROOKSHIRE D. W.	LIPAN (STRAWN)	3843-4037	1	completion date 09/07/1976	plugged / abandoned 05/27/1993	4207	0	0
22130104	Plugged Gas Well	32.4156721, -97.9389069	4194	GAS, not on schedule	7B	069006	GUARDIAN LEASING CO. [337103]	LUCAS HERMAN W.	LIPAN (STRAWN)	3715-4117	1	completion date 09/08/1976	plugged / abandoned 09/22/1987	4194	0	0
22130105	Gas Well	32.4159937, -97.9535568	4850	GAS, on schedule	7B	070509	SUN-KEY OIL CO., INC. [829320]	ANDERSON	LIPAN (STRAWN)	3818-3836	1	completion date 10/28/1976	ongoing as of 06/2007	NA	19996	45
22130106	Plugged Gas Well	32.4082027, -97.9293085	4695	GAS, not on schedule	7B	070917	GRYNBERG, JACK J. [337114]	HOLMES	LIPAN (STRAWN)	no record found	1	no record found	no record found	no record found	0	0
22130107	Plugged Gas Well	32.4153306, -97.9136133	5000	GAS, not on schedule	7B	070818	GRYNBERG, JACK J. [337114]	NEELY KAY	LIPAN (STRAWN)	no record found	1	no record found	plugged / abandoned 02/29/1980	5000	0	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130109	Gas Well	32.3787263, -97.8739389	6000	GAS, not on schedule	7B	069342	XTO ENERGY INC. [945936]	MCINTOSH D.	MCINTOSH (STRAWN)	4356-4360	1	completion date 08/21/1976	ongoing as of 06/2007	NA	133339	0
22130117	Gas Well	32.4174854, -97.954352	4300	GAS, not on schedule	7B	069251	GULF OIL CORP. [338130]	HOBBS H T	LIPAN (STRAWN)	no record found	1	no record found	plugged / abandoned 07/06/1979	4300	0	0
22130118	Plugged Gas Well	32.4137311, -97.9687244	5075	GAS, not on schedule	7B	071135	E.G. OPERATING [238558]	MARSHAL LOFTIN TRACT 1	LIPAN (STRAWN)	3968-3998	1	completion date 04/08/1977	plugged / abandoned 04/10/1997	5075	26899	0
22130120	Plugged Gas Well	32.4178077, -97.9653518	4972	GAS, not on schedule	7B	069180	E.G. OPERATING [238558]	WILLIAMS C.	LIPAN (STRAWN)	3930-3934	1	completion date 11/16/1976	plugged / abandoned 05/25/1993	4972	116132	0
22130121	Plugged Gas Well	32.4212208, -97.9661996	5269	GAS, not on schedule	7B	068939	GULF OIL CORP. [338130]	GILLIAM, J. H.	LIPAN (STRAWN)	3366-4036	1	completion date 10/15/1976	plugged / abandoned 05/23/1984	5269	0	0
22130122	Gas Well	32.4171897, -97.979245	5189	GAS, on schedule	7B	071136	XTO ENERGY INC. [945936]	MARSHALL LOFTIN TRACT 4	LIPAN (STRAWN)	no record found	2	completion date 04/09/1977	ongoing as of 06/2007	NA	212215	0
22130127	Gas Well	32.4181524, -97.9350694	5062	GAS, on schedule	7B	070826	SUN-KEY OIL CO., INC. [829320]	HOOPER	LIPAN (STRAWN)	3737-3751	3 E	completion date 02/13/1977	ongoing as of 06/2007	NA	152940	7
22130129	Gas Well	32.3459873, -97.8563837	5200	GAS, on schedule	7B	071128	XTO ENERGY INC. [945936]	CRAVENS	LIPAN (MARBLE FALLS)	4786-5152	1	completion date 04/11/1977	ongoing as of 06/2007	NA	105219	0
22130131	Gas Well	32.4034516, -97.9085989	4207	GAS, on schedule	7B	071330	XTO ENERGY INC. [945936]	FAULKNER, K. H.	LIPAN (STRAWN)	3210-3220	1	completion date 11/21/1976	ongoing as of 06/2007	NA	57663	0
22130132	Plugged Gas Well	32.4039365, -97.9596945	5000	GAS, not on schedule	7B	088963	E.G. OPERATING [238558]	BROOKSHIRE, D.W.	LIPAN (STRAWN)	3834-3957	2	completion date 03/24/1977	plugged / abandoned 08/01/1997	5000	1771	0
22130133	Gas Well	32.4053484, -97.98306	4198	GAS, on schedule	7B	072569	XTO ENERGY INC. [945936]	BROOKSHIRE, BOBBY G.	LIPAN (STRAWN)	3854-3968	1	completion date 12/24/1976	ongoing as of 06/2007	NA	35915	0
22130134	Plugged Gas Well	32.4220751, -97.9549398	4200	GAS, not on schedule	7B	072572	GUARDIAN LEASING CO. [337103]	KNAUF, E. DANIEL, JR.	LIPAN (STRAWN)	3921-4021	1	completion date 02/02/1977	plugged / abandoned 10/14/1987	4200	0	0
22130135	Plugged Gas Well	32.3979146, -97.9829988	4200	GAS, not on schedule	7B	072571	E.G. OPERATING [238558]	HENDERSON, JACK C.	LIPAN (STRAWN)	3858-4021	1	completion date 01/22/1977	plugged / abandoned 08/04/1997	4500	1791	0
22130137	Gas Well	32.3995368, -97.9695123	5077	GAS, on schedule	7B	070511	SUN-KEY OIL CO., INC. [829320]	HALBERT	LIPAN (STRAWN)	3808-3896	2 E	completion date 02/21/1977	ongoing as of 06/2007	NA	207802	39
22130141	Plugged Gas Well	32.4096759, -97.9696388	4400	GAS, not on schedule	7B	071133	VISTA RESOURCES, INC. [886249]	MARSHALL LOFTIN TRACT 1	LIPAN (STRAWN)	3922-3979	3	completion date 04/08/1977	plugged / abandoned 07/01/1982	4400	0	0
22130142	Gas Well	32.3758378, -97.8779173	5250	GAS, on schedule	7B	069574	XTO ENERGY INC. [945936]	MCINTOSH D.	MCINTOSH (STRAWN)	4386-4390	2	completion date 12/29/1976	ongoing as of 06/2007	NA	173716	0
22130143	Water Supply from Gas	32.4149807, -97.9813586	5195	GAS, on schedule	7B	071137	XTO ENERGY INC. [945936]	MARSHALL LOFTIN TRACT 2	LIPAN (STRAWN)	4725-4814	4	completion date 04/12/1977	ongoing as of 06/2007	NA	48459	0
22130146	Gas Well	32.3710509, -97.8738197	5200	GAS, on schedule	7B	070901	GARDNER, OLLIE [294515]	GARDNER, OLLIE	MCINTOSH (STRAWN)	4182-4190	1	completion date 3/16/1977	ongoing as of 06/2007	NA	7216	0
22130147	Gas Well	32.2749886, -97.8852833	5000	GAS, on schedule	7B	071329	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4054-4650	3	completion date 01/26/1977	ongoing as of 06/2007	NA	359272	80
22130148	Gas Well	32.2847198, -97.8729032	5000	GAS, not on schedule	7B	070166	GULF OIL CORP. [338130]	BRATTEN RANCH	BRATTEN (MARBLE FALLS)	no record found	2	start of reporting 01/1993	ongoing as of 06/2007	NA	0	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130148	Gas Well	32.2847198, -97.8729032	5000	GAS, on schedule	7B	079872	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4119-4273	2	completion date 12/07/1978	ongoing as of 06/2007	NA	137637	0
22130150	Plugged Gas Well	32.4209393, -97.9778277	5120	GAS, not on schedule	7B	073824	SUN-KEY OIL CO., INC. [829320]	MUSICK -A-	LIPAN (STRAWN)	2900-2918	2 E	completion date 10/02/1978	plugged / abandoned 04/03/1996	5120	31397	0
22130153	Gas Well	32.4096771, -97.9617768	4200	GAS, not on schedule	7B	071134	VISTA RESOURCES, INC. [886249]	MARSHALL LOFTIN TRACT 6	LIPAN (STRAWN)	3850-3950	5	completion date 04/12/1977	plugged / abandoned 07/01/1982	4200	0	0
22130158	Plugged Gas Well	32.3148523, -97.8389931	5170	GAS, not on schedule	7B	078628	E.G. OPERATING [238558]	WIGGINS	MCINTOSH (STRAWN)	3943-3957	1	completion date 09/01/1978	plugged / abandoned 05/26/1993	5170	11950	17
22130161	Plugged Gas Well	32.3320509, -97.9024101	5300	GAS, not on schedule	7B	070905	E.G. OPERATING [238558]	ROLLINS, OTIS GAS UNIT	OTIS ROLLINS (MARBLE FALLS)	5130-5140	1	completion date 02/06/1977	plugged / abandoned 09/28/1993	5300	18045	179
22130164	Plugged Gas Well	32.3992549, -97.9592251	4189	GAS, not on schedule	7B	072570	E.G. OPERATING [238558]	GILMORE, A. R. ESTATE	LIPAN (STRAWN)	3828-3988	1	completion date 02/25/1977	plugged / abandoned 11/07/1994	4189	30573	0
22130167	Plugged Gas Well	32.3992213, -97.9515039	4196	GAS, not on schedule	7B	072568	GUARDIAN LEASING CO. [337103]	BRIDIER, C. L.	LIPAN (STRAWN)	3770-3920	1	completion date 02/13/1977	plugged / abandoned 09/11/1987	4196	3520	0
22130176	Gas Well	32.3757238, -97.8648428	5200	GAS, on schedule	7B	071382	XTO ENERGY INC. [945936]	MCINTOSH D.	MCINTOSH (STRAWN)	2916-2926	3	completion date 04/04/1977	ongoing as of 06/2007	NA	159983	0
22130179	Plugged Gas Well	32.4104304, -97.9327589	4189	GAS, not on schedule	7B	073042	GUARDIAN LEASING CO. [337103]	DUMAS, BILLY	LIPAN (STRAWN)	3687-3945	1	completion date 04/05/1977	plugged / abandoned 09/19/1987	4189	1757	0
22130180	Plugged Gas Well	32.4126474, -97.9282503	4183	GAS, not on schedule	7B	073043	GUARDIAN LEASING CO. [337103]	DUMAS, BILLY	LIPAN (STRAWN)	3683-3845	2	completion date 04/16/1977	plugged / abandoned 09/14/1987	4183	4985	0
22130182	Plugged Gas Well	32.3114759, -97.8242117	5030	GAS, not on schedule	7B	071129	E.G. OPERATING [238558]	CRAVENS "B"	LIPAN (MARBLE FALLS)	4835-5030	1	completion date 04/26/1977	plugged / abandoned 09/30/1993	5030	23601	0
22130184	Plugged Gas Well	32.3653411, -97.8759239	5390	GAS, on schedule	9	222405	CARRIZO OIL & GAS, INC. [135401]	BRAWNER UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 01/1993	end of reporting 05/1994	no record found	1298	0
22130185	Plugged Gas Well	32.3827745, -97.8611416	5250	GAS, not on schedule	7B	071701	GULF OIL CORP. [338130]	WILKERSON, W. D.	MCINTOSH (BIG SALINE)	no record found	1	no record found	plugged / abandoned 08/16/1980	5250	0	0
22130187	Gas Well	32.3092633, -97.8637794	5100	GAS, on schedule	7B	072353	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (BIG SALINE)	4636-4642	4	completion date 05/16/1977	ongoing as of 06/2007	NA	314073	64
22130188	Plugged Gas Well	32.4096616, -97.9472231	5075	GAS, not on schedule	7B	072665	CHEVRON U.S.A. INC. [148113]	ANDERSON, R. W. GAS UNIT	LIPAN (STRAWN)	3820-3830	1	completion date 05/03/1977	plugged / abandoned 05/25/1984	5075	5438	0
22130189	Gas Well	32.3258023, -97.8638334	5233	GAS, on schedule	7B	073998	SWAIM, CHARLES [831570]	TROTTER	MCINTOSH (BIG SALINE)	4770-4780	1	completion date 05/25/1977	ongoing as of 06/2007	NA	49722	0
22130190	Gas Well	32.3019919, -97.8377159	no record found	GAS, on schedule	7B	073996	SWAIM, CHARLES [831570]	NELON	MCINTOSH (BIG SALINE)	4742-4750	1	completion date 05/15/1977	ongoing as of 06/2007	NA	74977	0
22130191	Gas Well	32.3111555, -97.8482466	5085	GAS, on schedule	7B	073997	HILL CITY INJECTION, LLC [386385]	SSG KEVIN W. BUTLER	MCINTOSH (BIG SALINE)	4466-4486	1	completion date 05/05/1977	ongoing as of 06/2007	NA	2921	0
22130192	Plugged Gas Well	32.3288811, -97.914409	5265	GAS, not on schedule	7B	086954	PROGAS PROPERTIES, INC. [681150]	LAWRENCE, CLIFTON	LIPAN (STRAWN)	4938-5008	1	completion date 07/23/1978	plugged / abandoned 11/02/1993	5265	5221	55
22130193	Plugged Gas Well	32.4138415, -97.9253346	4184	GAS, not on schedule	7B	073045	GUARDIAN LEASING CO. [337103]	SASSER, ET AL	LIPAN (STRAWN)	3700-4078	1	completion date 05/02/1977	plugged / abandoned 09/16/1987	4184	8270	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130194	Gas Well	32.2950819, -97.8786477	5000	GAS, on schedule	7B	072356	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4188-4198	7	completion date 06/08/1977	ongoing as of 06/2007	NA	61560	0
22130195	Gas Well	32.2913151, -97.8700683	4950	GAS, not on schedule	7B	072357	GULF OIL CORP. [338130]	BRATTEN RANCH	BRATTEN (MARBLE FALLS)	no record found	8	start of reporting 01/1993	ongoing as of 06/2007	NA	0	0
22130195	Gas Well	32.2913151, -97.8700683	4950	GAS, on schedule	7B	079846	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4126-4277	8	completion date 11/21/1978	ongoing as of 06/2007	NA	192839	0
22130196	Plugged Gas Well	32.3009241, -97.8581096	5100	GAS, not on schedule	7B	072354	CHEVRON U.S.A. INC. [148113]	BRATTEN RANCH	BRATTEN (STRAWN)	2359-2369	5	completion date 05/28/1977	plugged / abandoned 10/03/1984	5100	5535	0
22130197	Plugged Gas Well	32.2859218, -97.8609748	5000	GAS, not on schedule	7B	072355	CHEVRON U.S.A. INC. [148113]	BRATTEN RANCH	BRATTEN (MARBLE FALLS)	4686-4696	6	completion date 06/29/1977	plugged / abandoned 10/04/1984	5000	459	1
22130198	Plugged Gas Well	32.3547706, -97.8503674	5100	GAS, not on schedule	7B	072666	E.G. OPERATING [238558]	WEST, F.	MCINTOSH (BIG SALINE)	4752-4762	1	completion date 07/14/1977	plugged / abandoned 04/15/1997	5100	136035	350
22130199	Plugged Gas Well	32.3408749, -97.8636532	5200	GAS, not on schedule	7B	076831	BARON CO., THE [52965]	WILSON	MCINTOSH (BIG SALINE)	no record found	1	no record found	plugged / abandoned 12/27/1979	5200	0	0
22130201	Gas Well	32.3051779, -97.8240576	5625	GAS, on schedule	7B	079588	XTO ENERGY INC. [945936]	NELON "B"	MCINTOSH (BIG SALINE)	4946-5095	1	completion date 10/14/1978	ongoing as of 06/2007	NA	104321	0
22130202	Gas Well	32.4141129, -97.9357197	4182	GAS, on schedule	7B	073044	XTO ENERGY INC. [945936]	DUMAS, BILLY	LIPAN (STRAWN)	3767-4134	3	completion date 05/16/1977	ongoing as of 06/2007	NA	53750	0
22130205	Gas Well	32.3999874, -97.8465403	5200	GAS, on schedule	7B	075871	SWAIM, CHARLES [831570]	MAYES E. D.	MCINTOSH (STRAWN)	no record found	1	completion date 09/27/1977	ongoing as of 06/2007	NA	60979	0
22130206	Plugged Gas Well	32.3033475, -97.8805052	no record found	GAS, not on schedule	7B	074267	BUCHANAN, WAYMAN W. [105190]	HEATHINGTON	BRATTEN (MARBLE FALLS)	no record found	1	no record found	plugged / abandoned 07/21/1980	4879	0	0
22130386	Gas Well	32.387698, -97.862883	5256	GAS, on schedule	7B	072736	XTO ENERGY INC. [945936]	CAMPBELL, C. L.	MCINTOSH (STRAWN)	2994-3004	2	completion date 08/10/1977	ongoing as of 06/2007	NA	95322	0
22130387	Gas Well	32.3757466, -97.8580684	5193	GAS, on schedule	7B	072737	XTO ENERGY INC. [945936]	NELSON, D.	MCINTOSH (STRAWN)	2918-2928	1	completion date 07/28/1977	ongoing as of 06/2007	NA	51184	0
22130393	Gas Well	32.3460499, -97.8945121	6300	GAS, on schedule	7B	073194	XTO ENERGY INC. [945936]	SWAIM, V.	MCINTOSH (STRAWN)	2762-2768	1	completion date 08/22/1977	ongoing as of 06/2007	NA	98729	0
22130397	Gas Well	32.3535082, -97.8449929	5010	GAS, on schedule	7B	073995	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (BIG SALINE)	4736-4746	2	completion date 08/31/1977	ongoing as of 06/2007	NA	32566	0
22130402	Gas Well	32.3589551, -97.8550226	5150	GAS, on schedule	7B	073094	XTO ENERGY INC. [945936]	WEST, F.	MCINTOSH (BIG SALINE)	4770-4780	2	completion date 09/01/1977	ongoing as of 06/2007	NA	468090	153
22130403	Gas Well	32.34746, -97.8469177	5010	GAS, on schedule	7B	074055	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (BIG SALINE)	4740-4750	4	completion date 09/14/1977	ongoing as of 06/2007	NA	44973	120
22130405	Gas Well	32.3683953, -97.8690703	5100	GAS, on schedule	7B	074867	XTO ENERGY INC. [945936]	MCINTOSH, L.	MCINTOSH (STRAWN)	2868-2878	1	completion date 11/15/1977	ongoing as of 06/2007	NA	146599	0
22130412	Gas Well	32.3221754, -97.86152	5050	GAS, on schedule	7B	121055	MCREYNOLDS, DOYCE [556220]	COKER	MCINTOSH (STRAWN)	2588-2592	1	completion date 12/18/1984	ongoing as of 06/2007	NA	89793	16
22130413	Gas Well	32.3468931, -97.8407732	4784	GAS, on schedule	7B	074905	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (STRAWN)	2975-2985	5	completion date 10/16/1977	ongoing as of 06/2007	NA	30301	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130416	Gas Well	32.2809659, -97.8703234	4950	GAS, not on schedule	7B	073974	GULF OIL CORP. [338130]	BRATTEN RANCH	BRATTEN (MARBLE FALLS)	no record found	9	start of reporting 01/1993	ongoing as of 06/2007	NA	0	0
22130416	Gas Well	32.2809659, -97.8703234	4950	GAS, on schedule	7B	078928	XTO ENERGY INC. [945936]	BRATTEN RANCH	BRATTEN (STRAWN)	4470-4780	9	completion date 10/04/1978	ongoing as of 06/2007	NA	422370	0
22130417	Gas Well	32.3800118, -97.8536856	5230	GAS, not on schedule	7B	073999	E.G. OPERATING [238558]	SCHOMERS, W. L. ET UX GAS UNIT	MCINTOSH (BIG SALINE)	4816-4826	1	completion date 11/03/1977	ongoing as of 06/2007	NA	75469	89
22130417	Gas Well	32.3800118, -97.8536856	5230	GAS, on schedule	7B	139232	XTO ENERGY INC. [945936]	SCHOMERS	MCINTOSH (STRAWN)	no record found	1	start of reporting 01/1993	ongoing as of 06/2007	NA	22839	0
22130419	Gas Well	32.3860559, -97.8565877	5032	GAS, on schedule	7B	085025	SHAW, GARY P. [771620]	SHAW	MCINTOSH (STRAWN)	3290-3302	1	completion date 04/15/1978	ongoing as of 06/2007	NA	15677	21
22130422	Gas Well	32.3505871, -97.8919284	5338	GAS, not on schedule	7B	076176	SUN-KEY OIL CO., INC. [829320]	MORRISON, SID	OTIS ROLLINS (MARBLE FALLS)	2760-2770	1 U	completion date 01/21/1978	plugged / abandoned 03/27/1997	5338	20481	0
22130422	Gas Well	32.3505871, -97.8919284	5339	GAS, not on schedule	7B	076177	SUN-KEY OIL CO., INC. [829320]	MORRISON, SID	MCINTOSH (STRAWN)	4183-4233	1 L	completion date 01/21/1978	plugged / abandoned 03/27/1997	no record found	61948	0
22130424	Plugged Gas Well	32.3121381, -97.8666591	5169	GAS, not on schedule	7B	084143	BUCHANAN, WAYMAN W. [105190]	SWAIM	BRATTEN (MARBLE FALLS)	no record found	1	no record found	plugged / abandoned 08/09/1980	6165	0	0
22130425	Plugged Gas Well	32.394359, -97.9190907	5000	GAS, not on schedule	7B	079769	MORRIS, W. DALE INC. [588595]	CITY OF TOLAR LEANDIRS NO 1	MCINTOSH (STRAWN)	3972-3981	1	completion date 01/19/1978	plugged / abandoned 06/18/1985	5000	0	0
22130430	Gas Well	32.3032168, -97.8424865	5152	GAS, on schedule	7B	075717	XTO ENERGY INC. [945936]	PARKER, J. T.	MCINTOSH (STRAWN)	4730-4746	1	completion date 12/16/1977	ongoing as of 06/2007	NA	141578	1
22130433	Plugged Gas Well	32.422825, -97.7794507	5358	GAS, not on schedule	7B	102357	BROWN & HINDS ENTERPRISES, INC. [97513]	WHITEHEAD	GRANBURY, E. (CONGL.)	4995-5015	1	completion date 04/30/1978	plugged / abandoned 09/18/1985	5358	0	0
22130435	Gas Well	32.3419439, -97.8907264	5150	GAS, on schedule	7B	075617	XTO ENERGY INC. [945936]	NIX, T. H.	MCINTOSH (STRAWN)	2802-2812	2	completion date 01/16/1978	ongoing as of 06/2007	NA	123822	0
22130436	Gas Well	32.3555632, -97.8416354	5100	GAS, on schedule	7B	077155	XTO ENERGY INC. [945936]	WEST, F.	MCINTOSH (STRAWN)	2904-3709	6	completion date 05/17/1978	ongoing as of 06/2007	NA	171322	0
22130442	Plugged Gas Well	32.3232287, -97.8646915	5056	GAS, not on schedule	7B	077156	BALL OIL FIELD SERVICE [48163]	MAJ. HENRY P.	BRATTEN (BIG SALINE)	4719-4969	1 T	completion date 04/28/1978	plugged / abandoned 11/15/1988	5057	12860	0
22130442	Plugged Gas Well	32.3232287, -97.8646915	5057	GAS, not on schedule	7B	077157	BALL OIL FIELD SERVICE [48163]	MAJ. HENRY P.	BRATTEN (STRAWN)	3148-3160	1 C	completion date 04/28/1978	plugged / abandoned 11/15/1988	5057	20808	0
22130443	Plugged Gas Well	32.3694135, -97.8025201	5260	GAS, not on schedule	7B	092049	DALLAS PRODUCTION, INC. [197690]	WILLIAMS "C"	MCINTOSH (STRAWN)	4678-4720	1	completion date 02/01/1981	plugged / abandoned 10/18/1983	5260	17333	0
22130444	Gas Well	32.3731215, -97.8314613	5200	GAS, on schedule	7B	102359	SWAIM, CHARLES [831570]	GAUNTT	MCINTOSH (STRAWN)	3236-3282	1	completion date 10/11/1979	ongoing as of 06/2007	NA	2938	0
22130446	Gas Well	32.3502307, -97.7843189	5275	GAS, on schedule	7B	092050	COOK, DAVID W. [174327]	COOK "C"	MCINTOSH (STRAWN)	4248-4380	1	completion date 01/05/1981	ongoing as of 06/2007	NA	12440	0
22130449	Gas Well	32.3507907, -97.8533796	5057	GAS, on schedule	7B	075691	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (BIG SALINE)	4733-4737	6	completion date 02/15/1978	ongoing as of 06/2007	NA	68240	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130453	Gas Well	32.3155343, -97.8177859	5350	GAS, on schedule	7B	077364	XTO ENERGY INC. [945936]	PEVELER, M.	MCINTOSH (BIG SALINE)	3816-3842	1	completion date 06/26/1978	ongoing as of 06/2007	NA	258582	49
22130470	Plugged Oil Well	32.2449839, -97.9341403	4579	OIL, not on schedule	7B	014359	DALLAS PRODUCTION, INC. [197690]	SEALE	DEEP DOWN RANCH (MARBLE FALLS)	no record found	1	no record found	plugged / abandoned 02/17/1982	4579	0	0
22130476	Gas Well	32.3081308, -97.8451192	5285	GAS, on schedule	7B	077792	XTO ENERGY INC. [945936]	PARKER, J. T.	MCINTOSH (STRAWN)	4445-4763	2	completion date 07/15/1978	ongoing as of 06/2007	NA	128119	1
22130483	Gas Well	32.3276799, -97.8269775	5093	GAS, on schedule	7B	079132	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (BIG SALINE)	4663-4673	7	completion date 08/03/1978	ongoing as of 06/2007	NA	224251	60
22130484	Gas Well	32.3356124, -97.8369086	5150	GAS, on schedule	7B	079133	XTO ENERGY INC. [945936]	CRAVENS	MCINTOSH (BIG SALINE)	4748-4760	3	completion date 09/01/1978	ongoing as of 06/2007	NA	32613	0
22130485	Gas Well	32.3125859, -97.9042428	5300	GAS, on schedule	7B	080640	SWAIM, CHARLES [831570]	LAWRENCE, C.H.	HILL CITY, W. (MARBLE FALLS)	4792-5003	1	completion date 06/07/1978	ongoing as of 06/2007	NA	34649	0
22130486	Plugged Gas Well	32.3947035, -97.717265	no record found	GAS, not on schedule	7B	078793	TAYLOR OPER. CO. [837900]	SWEATT-LANGDON	LANGDON (CONGL.)	no record found	1	no record found	plugged / abandoned 10/16/1980	5300	0	0
22130487	Gas Well	32.3306758, -97.8361949	5350	GAS, on schedule	7B	080641	SWAIM, CHARLES [831570]	LOWDEN, R R	HILL CITY, (CONGL)	4445-4477	1	completion date 07/01/1978	ongoing as of 06/2007	NA	7894	0
22130494	Gas Well	32.3232578, -97.8152451	5375	GAS, on schedule	7B	093534	XTO ENERGY INC. [945936]	PEVELER, M.	MCINTOSH (BIG SALINE)	3780-4245	2	completion date 02/15/1980	ongoing as of 06/2007	NA	177353	240
22130497	Plugged Gas Well	32.2803691, -97.9070986	4992	GAS, not on schedule	7B	084712	D AND N NATURAL GAS OP. CO. INC. [195894]	CROWELL, J. W.	BRATTEN (MARBLE FALLS)	4363-4689	1	completion date 09/07/1978	plugged / abandoned 01/29/1999	4992	6678	70
22130506	Gas Well	32.2957998, -97.9075863	5050	GAS, not on schedule	7B	085164	LOUISIANA LAND & EXPL. CO., THE [509500]	LAWRENCE, C. H.	HILL CITY, W. (MARBLE FALLS)	no record found	3	no record found	plugged / abandoned 07/26/1980	5050	0	0
22130508	Plugged Gas Well	32.3570056, -97.8914741	5130	GAS, not on schedule	7B	079666	TAYLOR OPER. CO. [837900]	VEST	MCINTOSH (BIG SALINE)	4261-4484	1	completion date 10/18/1978	plugged / abandoned 02/22/1985	5130	8989	0
22130514	Gas Well	32.3119775, -97.831238	no record found	no record found	no record found	no record found	no record found	no record found	no record found	no record found	no record found	no record found	no record found	no record found	0	0
22130519	Gas Well	32.317714, -97.9006125	5308	GAS, on schedule	7B	085181	SWAIM, CHARLES [831570]	NANNY	HILL CITY, W. (MARBLE FALLS)	4801-4990	1	completion date 09/30/1978	ongoing as of 06/2007	NA	115797	71
22130520	Gas Well	32.3282182, -97.806649	5330	GAS, on schedule	7B	084836	XTO ENERGY INC. [945936]	PEVELER, M	MCINTOSH (BIG SALINE)	3797-4254	3	completion date 02/21/1979	ongoing as of 06/2007	NA	53094	0
22130521	Gas Well	32.3125734, -97.8153672	5136	GAS, on schedule	7B	080365	XTO ENERGY INC. [945936]	CRAVENS "B"	MCINTOSH (STRAWN)	3790-3822	2	completion date 01/02/1979	ongoing as of 06/2007	NA	314002	76
22130525	Plugged Gas Well	32.3527807, -97.8185805	5022	GAS, not on schedule	7B	079836	S & J OPERATING CO. [740320]	COMANCHE PEAK "B"	MCINTOSH (STRAWN)	4214-4438	1	completion date 12/07/1978	plugged / abandoned 07/28/1989	5022	101232	0
22130526	Plugged Gas Well	32.3570683, -97.9320786	5254	GAS, not on schedule	7B	117229	SMITH PIPE OF ABILENE [794745]	WAGNER	HILL CITY, W. (MARBLE FALLS)	no record found	1	start of reporting 01/1993	plugged / abandoned 02/07/2007	5254	1699	0
22130541	Plugged Gas Well	32.3549489, -97.8962223	5100	GAS, not on schedule	7B	080979	TAYLOR OPER. CO. [837900]	VEST	OTIS ROLLINS (MARBLE FALLS)	4220-4251	2	completion date 01/23/1979	plugged / abandoned 2/25/85	5100	2582	0

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130543	Gas Well	32.3515475, -97.8789347	5273	GAS, on schedule	7B	086273	HILL CITY INJECTION, LLC [386385]	LUKER, GARY	MCINTOSH (STRAWN)	3312-3324	1	completion date 11/12/1979	ongoing as of 06/2007	NA	60766	0
22130546	Plugged Gas Well	32.2771944, -97.9581841	5110	GAS, not on schedule	7B	086150	SUN-KEY OIL CO., INC. [829320]	E.L. RHOADES UNIT	BRATTEN (MARBLE FALLS)	4398-4632	1	completion date 02/27/1979	plugged / abandoned 04/04/1996	5110	0	0
22130548	Gas Well	32.3761781, -97.920053	5010	GAS, not on schedule	7B	081214	TEXACALI PETROLEUM CORP. [844149]	DIABLO	MCINTOSH (STRAWN)	3907-3913	1	completion date 02/22/1979	plugged / abandoned 10/01/1980	5010	0	0
22130550	Gas Well	32.4249899, -97.9897078	4070	GAS, on schedule	7B	117413	WILLIS OPERATING CO. [926594]	CHISOLM	LIPAN (STRAWN)	2575-2579	1	completion date 12/02/1981	no record found	no record found	0	0
22130552	Gas Well	32.3503137, -97.8813109	5100	GAS, on schedule	7B	082437	XTO ENERGY INC. [945936]	CATE, W.E.	MCINTOSH (STRAWN)	4194-4207	1	completion date 04/04/1979	ongoing as of 06/2007	NA	168889	0
22130554	Gas Well	32.313417, -97.8081558	5370	GAS, on schedule	7B	083538	XTO ENERGY INC. [945936]	PEVELER, M	BRATTEN (MARBLE FALLS)	4851-5142	4	completion date 02/14/1979	ongoing as of 06/2007	NA	109880	1004
22130559	Gas Well	32.3327054, -97.7966528	5010	OIL, not on schedule	7B	016305	TAYLOR OPER. CO. [837900]	RAWLS	RAWLS (MARBLE FALLS)	no record found	2	no record found	no record found	no record found	0	0
22130559	Gas Well	32.3327054, -97.7966528	5010	GAS, not on schedule	7B	082250	TAYLOR OPER. CO. [837900]	RAWLS	MCINTOSH (STRAWN)	4800-4820	2	completion date 04/08/1979	reclassified to oil 06/01/1981	no record found	0	0
22130559	Gas Well	32.3327054, -97.7966528	5010	GAS, on schedule	7B	099943	SUN-KEY OIL CO., INC. [829320]	RAWLS	RAWLS (MARBLE FALLS)	no record found	2	start of reporting 01/1993	ongoing as of 06/2007	NA	96444	24
22130560	Plugged Gas Well	32.3394778, -97.7981077	5100	GAS, not on schedule	7B	082249	REPUBLIC ENERGY INC. [702645]	RAWLS	MCINTOSH (STRAWN)	4303-4839	1	completion date 04/08/1979	plugged / abandoned 01/29/2007	5100	51509	42
22130562	Plugged Gas Well	32.338804, -97.808282	4850	OIL, not on schedule	7B	016238	TAYLOR OPER. CO. [837900]	SQUAW CREEK	WILDCAT	no record found	1	completion date 11/23/1981	plugged / abandoned 12/09/1982	no record found	0	0
22130562	Plugged Gas Well	32.338804, -97.808282	4850	GAS, not on schedule	7B	098173	TAYLOR OPER. CO. [837900]	SQUAW CREEK	MCINTOSH (STRAWN)	4254-4292	1	completion date 11/23/1981	plugged / abandoned 12/09/1983	4850	1589	0
22130565	Gas Well	32.3297325, -97.7989087	5375	GAS, on schedule	7B	084559	SUN-KEY OIL CO., INC. [829320]	PEVELER, M.	MCINTOSH (BIG SALINE)	4734-4763	5	completion date 07/17/1979	ongoing as of 06/2007	NA	140767	75
22130570	Plugged Gas Well	32.3512232, -97.8158409	5580	GAS, not on schedule	7B	084178	E.G. OPERATING [238558]	CRAVENS	MCINTOSH (BIG SALINE)	4690-4704	8	completion date 08/15/1979	plugged / abandoned 09/29/1993	5582	61786	34
22130574	Gas Well	32.3167343, -97.8273124	4950	GAS, on schedule	7B	083636	XTO ENERGY INC. [945936]	WIGGINS	MCINTOSH (BIG SALINE)	4719-4738	2	completion date 07/22/1979	ongoing as of 06/2007	NA	266164	74
22130585	Plugged Gas Well	32.3829695, -97.7814729	5175	GAS, not on schedule	7B	093662	TEXLAND-RECTOR & SCHUMACHER [849745]	KLIMIST, DAVE UNIT	MCINTOSH (STRAWN)	no record found	1	no record found	plugged / abandoned 04/30/1987	5175	0	0
22130594	Plugged Gas Well	32.3622783, -97.8733153	5003	GAS, not on schedule	7B	100887	PACK-MAN ENERGY COMPANY [632916]	GIFFORD	MCINTOSH (STRAWN)	3283-3490	1	completion date 02/02/1982	plugged / abandoned 05/19/1994	5003	52593	0
22130600	Gas Well	32.3838849, -97.8456462	4967	GAS, on schedule	7B	118038	XTO ENERGY INC. [945936]	OSBORNE, LORA	MCINTOSH (STRAWN)	2838-2860	1	completion date 01/17/1985	ongoing as of 06/2007	NA	34535	0
22130619	Gas Well	32.367018, -97.7699483	5300	GAS, on schedule	7B	092319	GIANT ENERGY CORP. [302597]	SANDERS "A"	MCINTOSH (STRAWN)	4910-4922	1	completion date 01/20/1981	ongoing as of 06/2007	NA	89033	6
22130654	Gas Well	32.3357926, -97.7917061	4885	GAS, on schedule	7B	102915	MARSHALL, SAM [527835]	CLEVELAND	MCINTOSH (STRAWN)	4736-4740	2	completion date 09/22/1981	ongoing as of 06/2007	NA	296272	397

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API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130671	Gas Well	32.3690017, -97.7784697	5150	GAS, on schedule	7B	094088	MARSHALL, SAM [527835]	CLEVELAND GAS UNIT	MCINTOSH (STRAWN)	4912-4922	1	completion date 03/12/1981	ongoing as of 06/2007	NA	380179	710
22130672	Gas Well	32.3495389, -97.8956655	5075	GAS, on schedule	7B	099959	SWAIM, CHARLES [831570]	COLEMAN-NIX	MCINTOSH (STRAWN)	4198-4260	1	completion date 04/09/1981	ongoing as of 06/2007	NA	6685	0
22130676	Gas Well	32.352599, -97.8576736	4850	GAS, on schedule	7B	098193	SWAIM, CHARLES [831570]	NIX, OTIS J.	MCINTOSH (STRAWN)	3064-3188	2	completion date 12/09/1981	ongoing as of 06/2007	NA	134463	2
22130677	Gas Well	32.3564736, -97.8574554	4852	GAS, on schedule	7B	098191	SWAIM, CHARLES [831570]	NIX, OTIS J.	MCINTOSH (STRAWN)	3128-3215	1	completion date 10/20/1981	ongoing as of 06/2007	NA	129299	2
22130693	Plugged Gas Well	32.2882838, -97.8779794	4800	GAS, not on schedule	7B	120628	NORMANDY OIL AND GAS COMPANY [613078]	SLACK	BRATTEN (STRAWN)	4149-4172	1	completion date 08/25/1982	plugged / abandoned 03/12/1986	4800	184	0
22130710	Plugged Gas Well	32.3528594, -97.8132298	5004	GAS, not on schedule	7B	111553	MARCON OPERATING COMPANY, INC. [525765]	HAMILTON	MCINTOSH (STRAWN)	4348-4354	1	completion date 10/26/1983	plugged / abandoned 09/01/1992	5004	7488	0
22130764	Plugged Gas Well	32.4044592, -97.7890047	5325	GAS, not on schedule	7B	123295	ENQUEST EXPLORATION INC. [253060]	BOYLES	GRANBURY, NE (CONGLOMERATE)	4880-4890	1	completion date 10/01/1985	plugged / abandoned 10/05/1988	5325	20068	0
22130765	Plugged Gas Well	32.3994528, -97.7891539	5955	GAS, not on schedule	7B	126491	ENQUEST EXPLORATION INC. [253060]	BOYLES	BOYLES-LUKER (CONG 4870)	4870-4888	2	completion date 07/01/1985	plugged / abandoned 10/04/1988	5955	202	0
22130768	Plugged Gas Well	32.3914388, -97.7856715	5102	GAS, not on schedule	7B	126492	ENQUEST EXPLORATION INC. [253060]	LUKER	BOYLES-LUKER (CONG 4870)	4870-4888	3	completion date 06/08/1987	plugged / abandoned 10/07/1989	5102	1639	0
22130883	Plugged Gas Well	32.3429073, -97.9195528	6000	GAS, not on schedule	7B	142232	MILES OF TEXAS OIL AND GAS, INC. [566999]	CARDWELL, J. R.	MCINTOSH (STRAWN)	3502-3518	1	completion date 03/15/1991	plugged / abandoned 12/31/1997	6000	215	0
22130930	Gas Well	32.3214638, -97.8284661	4870	GAS, on schedule	7B	140101	XTO ENERGY INC. [945936]	LOWDEN, R. R.	MCINTOSH (STRAWN)	no record found	1	start of reporting 01/1993	ongoing as of 06/2007	NA	54281	0
22130931	Gas Well	32.3434347, -97.8915549	4327	GAS, on schedule	7B	140570	XTO ENERGY INC. [945936]	NIX, T. H.	MCINTOSH (STRAWN)	no record found	3	start of reporting 01/1993	ongoing as of 06/2007	NA	21054	0
22130970	Injection from Gas	32.3447148, -97.9272189	5681	GAS, on schedule	7B	186674	DIAMONDBACK DISPOSAL, L.P. [217044]	WOOD	MCINTOSH (STRAWN)	no record found	1	start of reporting 01/2001	ongoing as of 06/2007	NA	124	0
22130971	Gas Well	32.3461801, -97.9323135	5279	GAS, on schedule	9	182726	MEEKER, DAN MANAGEMENT, INC. [558967]	WOOD, MESA "A"	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 01/2001	ongoing as of 06/2007	NA	79912	131
22130972	Plugged Gas Well	32.3493688, -97.9348058	5280	GAS, not on schedule	9	183333	SMITH PIPE OF ABILENE [794745]	WOOD, MESA "B"	NEWARK, EAST (BARNETT SHALE)	no record found	2	start of reporting 06/2001	plugged / abandoned 01/22/2007	5280	23864	25
22130973	Gas Well	32.3526714, -97.9323016	5119	GAS, on schedule	9	186399	SMITH PIPE OF ABILENE [794745]	WOOD, MESA “D”	NEWARK, EAST (BARNETT SHALE)	no record found	4	start of reporting 10/2001	ongoing as of 06/2007	NA	5178	25
22130974	Plugged Gas Well	32.3455831, -97.936261	5140	GAS, not on schedule	9	186398	SMITH PIPE OF ABILENE [794745]	WOOD, MESA "C"	NEWARK, EAST (BARNETT SHALE)	no record found	3	start of reporting 10/2001	plugged / abandoned 01/25/2007	5140	30305	25

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Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22130975	Gas Well	32.3459498, -97.9396895	5126	GAS, on schedule	9	189623	REICHMANN PETROLEUM CORP. [699704]	RANDALL, TANYA	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 12/2001	ongoing as of 06/2007	NA	19841	326
22130976	Gas Well	32.2984972, -97.9394205	4890	GAS, on schedule	9	190530	REICHMANN PETROLEUM CORP. [699704]	COLEMAN, JAMES	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 02/2002	ongoing as of 06/2007	NA	16035	133
22130977	Gas Well	32.3030274, -97.9305625	5275	GAS, on schedule	9	190520	REICHMANN PETROLEUM CORP. [699704]	DOVE, WALLACE	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 12/2001	ongoing as of 06/2007	NA	0	0
22130979	Gas Well	32.2969722, -97.9287684	5082	GAS, on schedule	9	196386	REICHMANN PETROLEUM CORP. [699704]	DOVE, WALLACE	NEWARK, EAST (BARNETT SHALE)	no record found	2	start of reporting 02/2002	ongoing as of 06/2007	NA	9858	133
22130980	Gas Well	32.3038079, -97.9433537	5024	GAS, on schedule	9	190521	REICHMANN PETROLEUM CORP. [699704]	CARAWAY	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 02/2002	ongoing as of 06/2007	NA	11552	132
22130981	Gas Well	32.29325, -97.9255817	4912	GAS, on schedule	9	190519	REICHMANN PETROLEUM CORP. [699704]	MORRISON	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 02/2002	ongoing as of 06/2007	NA	11130	129
22130982	Gas Well	32.3407247, -97.7818535	5469	GAS, on schedule	9	207195	BURLINGTON RESOURCES O&G CO LP [109333]	COURTS CLEVELAND	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 09/2004	ongoing as of 06/2007	NA	42665	170
22130984	Gas Well	32.3385707, -97.6570884	5652	GAS, on schedule	9	207843	QUICKSILVER RESOURCES INC. [684830]	PROFESSOR	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 01/2005	ongoing as of 06/2007	NA	318092	952
22130985	Gas Well	32.3438582, -97.6531787	5717	GAS, on schedule	9	207840	QUICKSILVER RESOURCES INC. [684830]	MARY ANN	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 01/2005	ongoing as of 06/2007	NA	296253	941
22130986	Gas Well	32.344291, -97.6858649	5689	GAS, on schedule	9	208631	QUICKSILVER RESOURCES INC. [684830]	TRAPPER JOHN	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 02/2005	ongoing as of 06/2007	NA	360569	1569
22130987	Gas Well	32.3250999, -97.6704859	5658	GAS, on schedule	9	208464	QUICKSILVER RESOURCES INC. [684830]	TRAPPER JOHN	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 03/2005	ongoing as of 06/2007	NA	389992	1116
22130988	Gas Well	32.3260452, -97.6770505	5619	GAS, on schedule	9	208608	QUICKSILVER RESOURCES INC. [684830]	HOT LIPS UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 03/2005	ongoing as of 06/2007	NA	452334	1595
22130990	Gas Well	32.3810757, -97.8055815	5665	GAS, on schedule	9	208534	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 02/2005	ongoing as of 06/2007	NA	332472	3128
22130991	Gas Well	32.2835013, -97.8613406	5045	GAS, on schedule	9	213347	XTO ENERGY INC. [945936]	BRATTEN RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	10	start of reporting 10/2005	ongoing as of 06/2007	NA	193000	815

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Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/ 1981-06/2007 (BBLS)
22130992	Gas Well	32.3399533, -97.667607	5750	GAS, on schedule	9	209404	QUICKSILVER RESOURCES INC. [684830]	ZBAR LAND & CATTLE CO.	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 04/ 2005	ongoing as of 06/2007	NA	365307	1023
22130993	Gas Well	32.340947, -97.6905313	5637	GAS, on schedule	9	208607	QUICKSILVER RESOURCES INC. [684830]	HAWKEYE UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 04/ 2005	ongoing as of 06/2007	NA	330361	2242
22130995	Gas Well	32.3476144, -97.6708401	5631	GAS, on schedule	9	208624	QUICKSILVER RESOURCES INC. [684830]	PROFESSOR UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 04/ 2005	ongoing as of 06/2007	NA	245841	842
22130997	Gas Well	32.3974608, -97.7972513	5509	GAS, on schedule	9	214230	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 10/ 2005	ongoing as of 06/2007	NA	260524	962
22130999	Gas Well	32.3548022, -97.6641001	5677	GAS, on schedule	9	209465	QUICKSILVER RESOURCES INC. [684830]	MARY ANN UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 05/ 2005	ongoing as of 06/2007	NA	250992	746
22131007	Gas Well	32.3407391, -97.6971046	5506	GAS, on schedule	9	212206	QUICKSILVER RESOURCES INC. [684830]	MAJOR BURNS UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/ 2005	ongoing as of 06/2007	NA	176055	860
22131009	Gas Well	32.3444261, -97.6475027	5699	GAS	9	211591	QUICKSILVER RESOURCES INC. [684830]	GIBBS UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	2	start of reporting 06/ 2005	ongoing as of 06/2007	NA	414027	758
22131010	Gas Well	32.3591738, -97.6595389	5652	GAS, on schedule	9	212018	QUICKSILVER RESOURCES INC. [684830]	GIBBS UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/ 2005	ongoing as of 06/2007	NA	407151	993
22131011	Gas Well	32.2975514, -97.935848	4779	GAS, on schedule	9	213271	BURLINGTON RESOURCES O&G CO LP [109333]	CARAWAY UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	2	start of reporting 09/ 2005	ongoing as of 06/2007	NA	13276	0
22131012	Gas Well	32.3268274, -97.6338791	6101	GAS, on schedule	9	211015	BURLINGTON RESOURCES O&G CO LP [109333]	GEORGES CREEK RANCH	(NEWARK, EAST (BARNETT SHALE)	no record found	5 H	start of reporting 06/ 2005	ongoing as of 06/2007	NA	280205	0
22131017	Gas Well	32.3122069, -97.9342365	5038	GAS, on schedule	9	212354	R.P.WILSON, INC [687012]	CARAWAY "B" UNIT	(NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 08/ 2005	ongoing as of 06/2007	NA	62665	637
22131019	Gas Well	32.3296739, -97.6664789	5666	GAS, on schedule	9	212720	QUICKSILVER RESOURCES INC. [684830]	COLONEL FLAGG UNIT	(NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 09/ 2005	ongoing as of 06/2007	NA	321568	991
22131020	Gas Well	32.3450174, -97.6768828	5627	GAS, on schedule	9	212192	QUICKSILVER RESOURCES INC. [684830]	COLONEL FLAGG UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 09/ 2005	ongoing as of 06/2007	NA	225626	994
22131023	Gas Well	32.3509126, -97.7660618	5578	GAS, on schedule	9	213413	CARRIZO OIL & GAS, INC. [135401]	BUZZARD HOLLOW RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 08/ 2005	ongoing as of 06/2007	NA	155009	1136

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Table 2.5.1-204 (Sheet 11 of 15) Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22131028	Gas Well	32.3839643, -97.7993291	5604	GAS, on schedule	9	215346	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 11/2005	ongoing as of 06/2007	NA	132733	324
22131030	Gas Well	32.3291113, -97.6438453	5798	GAS, on schedule	9	213350	QUICKSILVER RESOURCES INC. [684830]	GILLIGAN UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 09/2005	ongoing as of 06/2007	NA	378906	601
22131031	Gas Well	32.275444, -97.8690297	4956	GAS, on schedule	9	216476	XTO ENERGY INC. [945936]	BRATTEN RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	11	start of reporting 11/2005	ongoing as of 06/2007	NA	150952	941
22131033	Gas Well	32.4093447, -97.8081938	5532	GAS, on schedule	9	214309	BURLINGTON RESOURCES O&G CO LP [109333]	BOWSER RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 12/2005	ongoing as of 06/2007	NA	132578	712
22131034	Gas Well	32.3525016, -97.6383943	5751	GAS, on schedule	9	214660	QUICKSILVER RESOURCES INC. [684830]	MOONEY UNIT "A"	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 11/2005	ongoing as of 06/2007	NA	541726	745
22131035	Gas Well	32.353053, -97.6327817	5724	GAS, on schedule	9	214662	QUICKSILVER RESOURCES INC. [684830]	MOONEY UNIT "B"	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 12/2005	ongoing as of 06/2007	NA	413641	826
22131040	Gas Well	32.4283047, -97.6593755	5611	GAS, on schedule	9	214063	YOUNG, MARSHALL R., OIL CO. [949290]	HELTON	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 11/2005	ongoing as of 06/2007	NA	247075	3577
22131043	Gas Well	32.3746476, -97.8351601	5423	GAS, on schedule	9	216477	XTO ENERGY INC. [945936]	DORSEY	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 12/2005	ongoing as of 06/2007	NA	117340	742
22131044	Gas Well	32.3641663, -97.6305901	5859	GAS, on schedule	9	214912	QUICKSILVER RESOURCES INC. [684830]	CAMPOS UNIT "A"	(NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 01/2006	ongoing as of 06/2007	NA	443249	1082
22131046	Gas Well	32.3608413, -97.7583962	5537	GAS, on schedule	9	215376	QUICKSILVER RESOURCES INC. [684830]	HAYWORTH	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 02/2006	ongoing as of 06/2007	NA	79650	307
22131047	Gas Well	32.3675248, -97.7635504	5525	GAS, on schedule	9	217195	QUICKSILVER RESOURCES INC. [684830]	MASSEY	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 02/2006	ongoing as of 06/2007	NA	202513	1328
22131050	Gas Well	32.3650692, -97.7679353	5528	GAS, on schedule	9	216306	QUICKSILVER RESOURCES INC. [684830]	MASSEY	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 03/2006	ongoing as of 06/2007	NA	179414	1297
22131052	Gas Well	32.365355, -97.6383162	5715	GAS, on schedule	9	223721	QUICKSILVER RESOURCES INC. [684830]	CAMPOS UNIT "B"	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 07/2006	ongoing as of 06/2007	NA	285307	473
22131053	Gas Well	32.3827316, -97.6429904	5744	GAS, on schedule	9	221657	QUICKSILVER RESOURCES INC. [684830]	CAMPOS UNIT "B"	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/2006	ongoing as of 06/2007	NA	388616	620
22131054	Injection Well	32.3400478, -97.8863096	8460	GAS, on schedule	9	220318	XTO ENERGY INC. [945936]	PEAK SWD	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 04/2006	ongoing as of 06/2007	NA	0	0

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Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/ 1981-06/2007 (BBLS)
22131056	Gas Well	32.4193048, -97.9008955	5390	GAS, on schedule	9	220340	BURLINGTON RESOURCES O&G CO LP [109333]	COURTS CLEVELAND B	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 06/ 2006	ongoing as of 06/2007	NA	176844	1761
22131064	Shut-In Well (Type Gas)	32.3850968, -97.9762927	5074	GAS, on schedule	9	224760	EOG RESOURCES, INC. [253162]	BIG L RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 01/ 2006	ongoing as of 06/2007	NA	8228	0
22131065	Gas Well	32.4070967, -97.812368	5493	GAS, on schedule	9	217614	BURLINGTON RESOURCES O&G CO LP [109333]	BOWSER RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 02/ 2006	ongoing as of 06/2007	NA	216586	649
22131066	Gas Well	32.3438016, -97.8991553	5352	GAS, on schedule	9	224946	XTO ENERGY INC. [945936]	NIX UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	5 H	start of reporting 08/ 2006	ongoing as of 06/2007	NA	106770	27
22131067	Gas Well	32.417093, -97.9834476	5084	GAS, on schedule	9	221298	XTO ENERGY INC. [945936]	MARSHALL-LOFTIN	NEWARK, EAST (BARNETT SHALE)	no record found	6 H	start of reporting 02/ 2006	ongoing as of 06/2007	NA	93186	190
22131072	Gas Well	32.344382, -97.8559377	5388	GAS, on schedule	9	225990	XTO ENERGY INC. [945936]	CRAVENS	NEWARK, EAST (BARNETT SHALE)	no record found	8 H	start of reporting 03/ 2006	ongoing as of 06/2007	NA	106220	280
22131073	Shut-In Well (Type Gas)	32.3512806, -97.9981383	4742	GAS, on schedule	9	226350	EOG RESOURCES, INC. [253162]	DALE HEAD	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 02/ 2006	ongoing as of 06/2007	NA	0	0
22131076	Gas Well	32.3607453, -97.7525234	5506	GAS, on schedule	9	217194	QUICKSILVER RESOURCES INC. [684830]	BROWN-EYED GIRL UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 04/ 2006	ongoing as of 06/2007	NA	209104	1768
22131079	Gas Well	32.4001076, -97.9576302	5123	GAS, on schedule	9	223644	BURLINGTON RESOURCES O&G CO LP [109333]	UNDERWOOD	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/ 2006	ongoing as of 06/2007	NA	77442	1220
22131080	Gas Well	32.4257679, -97.6550329	5904	GAS, on schedule	9	217604	BURLINGTON RESOURCES O&G CO LP [109333]	KIDD	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 03/ 2006	ongoing as of 06/2007	NA	258449	2398
22131084	Gas Well	32.3942985, -97.6638506	5736	Gas, on schedule	9	219774	QUICKSILVER RESOURCES INC. [684830]	SUGAR CRISP UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 05-2006	ongoing as of 06/2007	NA	309899	2158
22131085	Gas Well	32.4067667, -97.6723061	5741	GAS, on schedule	9	217192	QUICKSILVER RESOURCES INC. [684830]	SUGAR CRISP UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 04/ 2006	ongoing as of 06/2007	NA	266669	3014
22131086	Gas Well	32.2696251, -97.9059267	4683	GAS, on schedule	9	224763	XTO ENERGY INC. [945936]	WANN UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 06/ 2006	ongoing as of 06/2007	NA	189698	131
22131087	Gas Well	32.3576258, -97.8160636	7269	GAS, on schedule	9	220268	BURLINGTON RESOURCES O&G CO LP [109333]	WALLACE	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/ 2006	ongoing as of 06/2007	NA	121154	543
22131091	Gas Well	32.4302122, -97.6568308	5869	GAS, on schedule	9	219758	QUICKSILVER RESOURCES INC. [684830]	CHRISSY UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/ 2006	ongoing as of 06/2007	NA	260078	5310
22131092	Gas Well	32.3231138, -97.6185531	6193	GAS, on schedule	9	219782	QUICKSILVER RESOURCES INC. [684830]	HINTON	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 05/ 2006	ongoing as of 06/2007	NA	118054	443

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Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22131093	Gas Well	32.3454437, -97.6336935	6221	GAS, on schedule	9	219764	QUICKSILVER RESOURCES INC. [684830]	DENVER, BOB UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 06/2006	ongoing as of 06/2007	NA	326578	454
22131094	Gas Well	32.2814334, -97.9571602	4778	GAS, on schedule	9	223795	EOG RESOURCES, INC. [253162]	RHOADES	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 07/2006	ongoing as of 06/2007	NA	19370	481
22131095	Gas Well	32.3434656, -97.6344599	6169	GAS, on schedule	9	221658	QUICKSILVER RESOURCES INC. [684830]	DENVER, BOB UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 06/2006	ongoing as of 06/2007	NA	273013	636
22131097	Gas Well	32.4276794, -97.6530085	5929	GAS, on schedule	9	220267	BURLINGTON RESOURCES O&G CO LP [109333]	KIDD	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 05/2006	ongoing as of 06/2007	NA	351786	5579
22131100	Gas Well	32.3165553, -97.6208048	6072	GAS, on schedule	9	220390	BURLINGTON RESOURCES O&G CO LP [109333]	GEORGES CREEK RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	14 H	start of reporting 06/2006	ongoing as of 06/2007	NA	23445	0
22131107	Gas Well	32.3654988, -97.8746112	5390	GAS, on schedule	9	222405	CARRIZO OIL & GAS, INC. [135401]	BRAWNER UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 08/2006	ongoing as of 06/2007	NA	91535	1177
22131108	Gas Well	32.4016956, -97.8139315	5562	GAS, on schedule	9	220394	BURLINGTON RESOURCES O&G CO LP [109333]	BOWSER RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 06/2006	ongoing as of 06/2007	NA	169980	201
22131109	Injection Well	32.3185452, -97.8566655	8057	OIL, on schedule	9	030743	PINNERGY, LTD. [665701]	PARKER	NEWARK, EAST (BARNETT SHALE)	no record found	1 D	start of reporting 01/2007	ongoing as of 06/2007	NA	0	0
22131118	Gas Well	32.3143157, -97.8750842	5352	GAS, on schedule	9	220360	BURLINGTON RESOURCES O&G CO LP [109333]	COURTS CLEVELAND "C"	NEWARK, EAST (BARNETT SHALE)	no record found	4 H	start of reporting 06/2006	ongoing as of 06/2007	NA	127575	2144
22131129	Shut-In Well (Type Gas)	32.3641094, -97.7420842	5633	GAS, on schedule	9	225854	BURLINGTON RESOURCES O&G CO LP [109333]	HAYWORTH	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 07/2006	ongoing as of 06/2007	NA	63193	554
22131130	Gas Well	32.4145608, -97.6582308	5790	GAS, on schedule	9	224364	BURLINGTON RESOURCES O&G CO LP [109333]	KIDD "A" UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 10/2006	ongoing as of 06/2007	NA	154530	1153
22131133	Gas Well	32.3957178, -97.8086617	5560	GAS, on schedule	9	225847	BURLINGTON RESOURCES O&G CO LP [109333]	EVANS	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 12/2006	ongoing as of 06/2007	NA	154470	769
22131140	Gas Well	32.3942417, -97.8023976	9947	GAS, on schedule	9	224379	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	6 H	start of reporting 07/2006	ongoing as of 06/2007	NA	219664	536
22131141	Gas Well	32.373611, -97.7951804	5530	GAS, on schedule	9	227014	WILLOWBEND INVESTMENTS, INC. [927665]	TRIAD-COOPER	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 12/2006	ongoing as of 06/2007	NA	52906	0
22131143	Gas Well	32.3867553, -97.8047562	5561	GAS, on schedule	9	224372	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	5 H	start of reporting 09/2006	ongoing as of 06/2007	NA	191657	642

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Table 2.5.1-204 (Sheet 14 of 15)
Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
22131144	Gas Well	32.4155656, -97.656206	5770	GAS, on schedule	9	224712	BURLINGTON RESOURCES O&G CO LP [109333]	KIDD "A" UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	4 H	start of reporting 09/2006	ongoing as of 06/2007	NA	138438	964
22131147	Gas Well	32.3793117, -97.8088362	5572	GAS, on schedule	9	224378	BURLINGTON RESOURCES O&G CO LP [109333]	COMANCHE PEAK	NEWARK, EAST (BARNETT SHALE)	no record found	7 H	start of reporting 07/2006	ongoing as of 06/2007	NA	247859	2342
22131152	Gas Well	32.3737215, -97.7185626	5589	GAS, on schedule	9	226345	EOG RESOURCES, INC. [253162]	WELBORN-SIMON	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 09/2006	ongoing as of 06/2007	NA	279	279
22131183	Gas Well	32.3537651, -97.7583743	5553	GAS, on schedule	9	226423	CARRIZO OIL & GAS, INC. [135401]	BUZZARD HOLLOW RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	3 H	start of reporting 12/2006	ongoing as of 06/2007	NA	49661	640
22131189	Gas Well	32.3544536, -97.798377	5392	GAS, on schedule	9	227193	REPUBLIC ENERGY INC. [702645]	ALBERTHAL	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 10/2006	ongoing as of 06/2007	NA	101350	1439
22131218	Gas Well	32.3705787, -97.7190762	5549	GAS, on schedule	9	226335	EOG RESOURCES, INC. [253162]	WELBORN-SIMON	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 12/2006	ongoing as of 06/2007	NA	120919	278
25130324	Gas Well	32.3158412, -97.6190431	5986	GAS, on schedule	9	209024	BURLINGTON RESOURCES O&G CO LP [109333]	GEORGES CREEK RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	2 H	start of reporting 03/2005	ongoing as of 06/2007	NA	506860	112
42530098	Plugged Gas Well	32.3023323, -97.8087006	5605	GAS, not on schedule	7B	101298	DALLAS PRODUCTION, INC. [197690]	KINNARD	MCINTOSH (STRAWN)	3798-3808	1	completion date 03/15/1982	plugged / abandoned 07/10/1985	5605	0	0
42530099	Gas Well	32.2876826, -97.8161648	5100	GAS, on schedule	7B	102339	SALSA GAS PRODUCERS, INC. [744274]	O'CONNER	MCINTOSH (STRAWN)	3410-3430	1	completion date 06/20/1979	ongoing as of 06/2007	NA	0	0
42530100	Gas Well	32.2751, -97.860932	5107	GAS, on schedule	7B	087207	XTO ENERGY INC. [945936]	DAVIS "A"	MCINTOSH (STRAWN)	3942-3954	1	completion date 02/15/1980	ongoing as of 06/2007	NA	160297	18
42530102	Gas Well	32.3073107, -97.6857966	5365	GAS, on schedule	7B	100787	GRISSOM PRODUCTION [334880]	GRISSOM	MCINTOSH (STRAWN)	3758-3770	1	completion date 03/25/1982	no record found		0	0
42530104	Gas Well	32.2573134, -97.8519112	5400	GAS, not on schedule	7B	075842	E.G. OPERATING [238558]	MCDOWELL, H.W. "A"	BRATTEN (STRAWN)	4112-4130	1	completion date 03/13/1978	ongoing as of 06/2007	NA	90353	0
42530104	Gas Well	32.2573134, -97.8519112	5400	GAS, on schedule	7B	143674	XTO ENERGY INC. [945936]	MCDOWELL, H.W. "A"	SOMERVELL (STRAWN)	no record found	1	no record found	no record found		71480	0
42530107	Gas Well	32.268669, -97.8726657	4615	GAS, on schedule	7B	087517	XTO ENERGY INC. [945936]	FRENCH	MCINTOSH (STRAWN)	3910-4074	1	completion date 04/04/1980	ongoing as of 06/2007	NA	411900	43
42530112	Gas Well	32.3188189, -97.6437077	5910	GAS, on schedule	9	214678	BURLINGTON RESOURCES O&G CO LP [109333]	GEORGES CREEK RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	6 H	start of reporting 08/2005	ongoing as of 06/2007	NA	233809	0
42530115	Injection Well	32.263672, -97.7029862	7410	OIL, not on schedule	9	030751	TWJM SERVICES, GP, LLC [875033]	TWJM SERVICES, LP SWD	NEWARK, EAST (BARNETT SHALE)	no record found	2	start of reporting 03/2007	ongoing as of 06/2007	NA	0	0
42530117	Gas Well	32.2456486, -97.6803013	5702	GAS, on schedule	9	223940	QUICKSILVER RESOURCES INC. [684830]	SATCHMO UNIT	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 04/2006	ongoing as of 06/2007	NA	204976	145

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Table 2.5.1-204 (Sheet 15 of 15)
Production and Injection Wells Within 10-Mile Radius of Comanche Peak, Data Reported by the Railroad Commission of Texas (RRC)

API	WELL TYPE	LATITUDE, LONGITUDE (NAD 83)	TOTAL DEPTH (ft)	PRORATION SCHEDULE	DISTRICT	LEASE	OPERATOR [OPERATOR NUMBER]	LEASE NAME	FIELD	PERFORATION DEPTH (ft)	WELL NUMBER	COMPLETION DATE / START OF REPORTING	PLUGGED / ABANDONED / END OF REPORTING	PLUG DEPTH (ft)	TOTAL GW GAS PRODUCTION 01/1981-06/2007 (MCF)	TOTAL CONDENSATE PRODUCTION 01/1981-06/2007 (BBLS)
42530118	Injection Well	32.1852507, -97.8698223	7518	OIL, not on schedule	9	030655	SHALE TANK TRUCK L.P. [768686]	4M SWD	NEWARK, EAST (BARNETT SHALE)	no record found	1	start of reporting 08/2006	ongoing as of 06/2007	NA	0	0
42530121	Gas Well	32.3228638, -97.6245913	6104	GAS, on schedule	9	223649	BURLINGTON RESOURCES O&G CO LP [109333]	GEORGES CREEK RANCH	NEWARK, EAST (BARNETT SHALE)	no record found	10 H	start of reporting 05/2006	ongoing as of 06/2007	NA	95845	0
42530128	Gas Well	32.1996416, -97.8277115	5145	GAS, on schedule	9	226558	EOG RESOURCES, INC. [253162]	RANCHO HIELO	NEWARK, EAST (BARNETT SHALE)	no record found	1 H	start of reporting 07/2006	ongoing as of 06/2007	NA	0	0

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**Table 2.5.1-205 (Sheet 1 of 5)
Time of Event, Location of Event, Best Estimate Body-Wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog**

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1985	2	10	14	15	52.21	36.4330	-98.4120	3.00	0.10	3.01	OGS
1985	5	5	1	39	30.78	34.6640	-97.5290	3.00	0.10	3.01	OGS
1985	9	6	22	17	2.86	35.81	-93.12	3.80	0.10	3.81	PDE
1985	9	18	15	54	4.64	33.55	-97.05	3.30	0.10	3.31	PDE
1985	9	23	1	3	44.10	34.7250	-95.0590	3.30	0.10	3.31	OGS
1985	12	31	18	27	26.12	34.7030	-97.4590	3.00	0.10	3.01	OGS
1986	1	30	22	26	37.07	32.07	-100.69	3.30	0.10	3.31	PDE
1986	3	3	11	45	17.48	35.31	-102.51	3.10	0.10	3.11	PDE
1986	10	20	4	32	49.00	37.92	-101.37	3.00	0.10	3.01	PDE
1987	1	24	16	8	17.01	35.8280	-98.0970	3.40	0.10	3.41	OGS
1987	12	6	17	43	48.18	34.6640	-97.3940	3.00	0.10	3.01	OGS
1987	12	8	1	42	40.28	36.0550	-98.0240	3.70	0.10	3.71	OGS
1989	7	20	6	7	51.54	36.3820	-98.8180	3.10	0.10	3.11	OGS
1990	7	28	7	53	33.75	34.6000	-93.3760	3.01	0.41	3.20	ANSS
1990	08	03	15	31	40.32	32.2050	-100.6925	3.35	0.30	3.45	NMT
1990	9	16	21	13	33.38	34.8550	-95.5770	3.20	0.10	3.21	OGS
1990	10	11	11	7	22.14	34.7770	-97.5030	3.60	0.10	3.61	OGS
1990	11	15	11	44	41.63	34.7610	-97.5500	4.00	0.10	4.01	OGS
1990	11	15	11	45	35.06	35.6030	-93.0420	3.50	0.41	3.69	ANSS
1991	1	24	5	0	26.90	36.38	-97.30	3.00	0.10	3.01	PDE

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Table 2.5.1-205 (Sheet 2 of 5) Time of Event, Location of Event, Best Estimate Body-Wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1991	7	20	23	38	19.21	28.91	-98.04	3.60	0.10	3.61	PDE
1992	1	2	11	45	35.61	32.33	-103.10	5.00	0.10	5.01	PDE
1992	08	26	03	24	51.16	32.2093	-102.5920	3.15	0.30	3.25	NMT
1992	12	17	7	18	5.65	34.7300	-97.5410	3.80	0.10	3.81	OGS
1993	1	14	17	6	10.45	35.5950	-98.2750	3.20	0.10	3.21	OGS
1993	4	9	12	29	19.17	28.81	-98.12	4.30	0.10	4.31	PDE
1993	5	7	17	50	37.70	34.7380	-97.5410	3.10	0.10	3.11	OGS
1993	5	16	15	30	19.39	28.81	-98.17	3.00	0.10	3.01	PDE
1993	9	29	2	1	19.06	35.87	-102.98	3.30	0.10	3.31	PDE
1993	10	19	16	59	52.41	36.5460	-98.1730	3.10	0.10	3.11	OGS
1993	11	30	03	07	36.28	35.8088	-103.1567	3.26	0.30	3.37	NMT
1993	12	05	00	58	24.06	27.9877	-102.0607	4.03	0.30	4.13	NMT
1993	12	05	03	35	14.14	27.8975	-102.0582	3.43	0.30	3.53	NMT
1994	4	16	7	20	29.99	34.6630	-97.7130	3.10	0.10	3.11	OGS
1994	4	29	3	28	58.68	36.25	-98.09	3.00	0.10	3.01	PDE
1995	1	18	15	51	39.90	34.7120	-97.5420	4.20	0.10	4.21	OGS
1995	4	5	5	31	16.23	35.20	-99.03	3.00	0.10	3.01	PDE
1995	4	14	0	32	56.17	30.28	-103.35	5.82	0.10	5.83	PDE
1995	4	14	2	19	38.50	30.30	-103.35	3.30	0.10	3.31	PDE
1995	4	15	14	33	29.51	30.27	-103.32	4.00	0.10	4.01	PDE

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Table 2.5.1-205 (Sheet 3 of 5)
Time of Event, Location of Event, Best Estimate Body-Wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1995	6	1	1	6	15.70	30.30	-103.35	3.50	0.10	3.51	PDE
1995	6	1	4	49	27.70	34.1340	-96.6830	3.30	0.10	3.31	OGS
1995	9	15	0	31	33.26	36.87	-98.69	4.10	0.10	4.11	PDE
1995	11	12	17	45	59.40	30.30	-103.35	3.60	0.10	3.61	PDE
1995	12	1	14	37	43.00	35.1550	-98.8970	3.00	0.10	3.01	OGS
1996	3	25	6	43	46.86	35.61	-102.60	3.50	0.10	3.51	PDE
1996	11	23	10	54	18.50	35.0400	-100.5040	3.09	0.41	3.28	ANSS
1997	2	12	23	53	10.77	34.9470	-100.8900	3.09	0.41	3.28	ANSS
1997	2	15	9	8	55.46	34.9730	-100.5690	3.25	0.41	3.45	ANSS
1997	3	16	19	7	28.00	34.2700	-93.4900	3.42	0.41	3.61	ANSS
1997	5	31	3	26	41.34	33.1820	-95.9660	3.42	0.41	3.61	ANSS
1997	9	6	23	38	0.91	34.66	-96.43	4.50	0.10	4.51	PDE
1997	9	6	23	38	1.99	34.6760	-96.4990	4.40	0.10	4.41	OGS
1997	10	19	11	12	09.74	32.3347	-103.9360	3.11	0.30	3.21	NMT
1998	1	2	15	47	16.43	37.8280	-103.4080	3.42	0.41	3.61	ANSS
1998	4	15	10	33	42.42	30.19	-103.30	3.60	0.10	3.61	PDE
1998	4	27	15	22	46.25	35.4530	-102.3830	3.25	0.41	3.45	ANSS
1998	4	28	14	13	1.27	34.7550	-98.4470	4.20	0.10	4.21	OGS
1998	7	7	18	44	44.46	34.7190	-97.5890	3.25	0.41	3.45	ANSS
1998	7	14	5	38	48.75	35.3440	-103.4730	3.01	0.41	3.20	ANSS

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Table 2.5.1-205 (Sheet 4 of 5) Time of Event, Location of Event, Best Estimate Body-Wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1998	10	30	17	41	21.42	36.7710	-97.6230	3.50	0.10	3.51	OGS
1998	10	30	17	41	22.20	36.8000	-97.6000	3.50	0.41	3.69	ANSS
1999	10	25	23	19	51.68	36.9462	-100.0700	3.00	0.10	3.01	OGS
1999	10	25	23	19	58.37	36.8460	-99.6590	3.09	0.41	3.28	ANSS
2000	1	14	10	39	34.94	34.6735	-95.0949	3.00	0.10	3.01	OGS
2000	8	7	17	19	8.00	35.3920	-101.8120	3.33	0.41	3.53	ANSS
2000	8	7	18	34	9.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS
2000	8	7	21	36	21.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS
2000	8	10	13	39	50.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS
2000	8	17	1	8	5.45	35.39	-101.81	3.90	0.10	3.91	PDE
2000	12	16	22	8	54.00	35.40	-101.80	3.90	0.10	3.91	PDE
2001	3	30	17	13	55.60	37.9330	-93.3270	3.17	0.41	3.37	ANSS
2001	6	2	1	55	53.72	32.3340	-103.1410	3.33	0.41	3.53	ANSS
2001	7	24	14	2	35.00	37.7000	-97.0000	3.09	0.41	3.28	ANSS
2001	8	4	1	13	28.00	34.4200	-93.2300	3.25	0.41	3.45	ANSS
2001	11	22	0	7	8.02	31.7860	-102.6310	3.17	0.41	3.37	ANSS
2002	2	8	16	7	13.84	34.6514	-98.3021	3.80	0.10	3.81	OGS
2002	5	31	9	57	9.87	34.9997	-97.6228	3.00	0.10	3.01	OGS
2002	5	31	9	57	10.02	34.0250	-97.6190	3.33	0.41	3.53	ANSS
2002	6	19	12	14	20.30	36.57	-103.03	3.70	0.10	3.71	PDE

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Table 2.5.1-205 (Sheet 5 of 5)
Time of Event, Location of Event, Best Estimate Body-Wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
2002	10	20	2	18	14.06	34.2140	-96.1810	3.60	0.10	3.61	OGS
2003	4	7	10	2	12.51	33.8920	-97.6950	3.01	0.41	3.20	ANSS
2003	9	24	15	2	9.09	35.2770	-101.7420	3.33	0.41	3.53	ANSS
2004	4	22	16	13	2.25	34.8040	-97.6770	3.01	0.41	3.20	ANSS
2004	6	8	0	15	8.38	34.0410	-97.3070	3.70	0.10	3.71	OGS
2004	6	8	0	15	9.99	34.23	-97.25	3.50	0.10	3.51	PDE
2004	6	10	12	30	9.86	34.2360	-97.2670	3.01	0.41	3.20	ANSS
2004	11	22	23	42	13.45	34.8640	-97.6720	3.09	0.41	3.28	ANSS
2004	11	30	23	59	34.00	36.9400	-93.8900	3.01	0.41	3.20	ANSS
2005	2	6	15	59	14.48	34.2380	-95.2380	3.50	0.10	3.51	OGS
2005	4	3	14	39	16.97	28.3930	-100.3050	3.50	0.41	3.69	ANSS
2005	4	22	5	17	4.09	34.1790	-95.1920	3.09	0.41	3.28	ANSS
2006	2	18	5	49	41.45	35.6720	-101.7940	3.50	0.41	3.69	ANSS
2006	3	28	23	55	11.49	35.3630	-101.8710	3.09	0.41	3.28	ANSS
2006	4	5	18	46	23.14	34.0690	-97.3140	3.09	0.41	3.28	ANSS
2006	4	8	18	8	35.23	31.9540	-101.4190	3.01	0.41	3.20	ANSS
2006	10	6	22	13	16.78	34.12	-97.62	3.50	0.10	3.51	PDEW

(Reference 2.5-335), (Reference 2.5-369)

Table 2.5.1-206 Deleted

Table 2.5.1-207 Deleted

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Table 2.5.2-201 (Sheet 1 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1985	2	10	14	15	52.21	36.4330	-98.4120	3.00	0.10	3.01	OGS
1985	5	5	1	39	30.78	34.6640	-97.5290	3.00	0.10	3.01	OGS
1985	9	6	22	17	2.86	35.81	-93.12	3.80	0.10	3.81	PDE
1985	9	18	15	54	4.64	33.55	-97.05	3.30	0.10	3.31	PDE
1985	9	23	1	3	44.10	34.7250	-95.0590	3.30	0.10	3.31	OGS
1985	12	31	18	27	26.12	34.7030	-97.4590	3.00	0.10	3.01	OGS
1986	1	30	22	26	37.07	32.07	-100.69	3.30	0.10	3.31	PDE
1986	3	3	11	45	17.48	35.31	-102.51	3.10	0.10	3.11	PDE
1986	10	20	4	32	49.00	37.92	-101.37	3.00	0.10	3.01	PDE
1987	1	24	16	8	17.01	35.8280	-98.0970	3.40	0.10	3.41	OGS
1987	12	6	17	43	48.18	34.6640	-97.3940	3.00	0.10	3.01	OGS
1987	12	8	1	42	40.28	36.0550	-98.0240	3.70	0.10	3.71	OGS
1989	7	20	6	7	51.54	36.3820	-98.8180	3.10	0.10	3.11	OGS
1990	7	28	7	53	33.75	34.6000	-93.3760	3.01	0.41	3.20	ANSS
1990	08	03	15	31	40.32	32.2050	-100.6925	3.35	0.30	3.45	NMT
1990	9	16	21	13	33.38	34.8550	-95.5770	3.20	0.10	3.21	OGS
1990	10	11	11	7	22.14	34.7770	-97.5030	3.60	0.10	3.61	OGS

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Table 2.5.2-201 (Sheet 2 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1990	11	15	11	44	41.63	34.7610	-97.5500	4.00	0.10	4.01	OGS
1990	11	15	11	45	35.06	35.6030	-93.0420	3.50	0.41	3.69	ANSS
1991	1	24	5	0	26.90	36.38	-97.30	3.00	0.10	3.01	PDE
1991	7	20	23	38	19.21	28.91	-98.04	3.60	0.10	3.61	PDE
1992	1	2	11	45	35.61	32.33	-103.10	5.00	0.10	5.01	PDE
1992	08	26	03	24	51.16	32.2093	-102.5920	3.15	0.30	3.25	NMT
1992	12	17	7	18	5.65	34.7300	-97.5410	3.80	0.10	3.81	OGS
1993	1	14	17	6	10.45	35.5950	-98.2750	3.20	0.10	3.21	OGS
1993	4	9	12	29	19.17	28.81	-98.12	4.30	0.10	4.31	PDE
1993	5	7	17	50	37.70	34.7380	-97.5410	3.10	0.10	3.11	OGS
1993	5	16	15	30	19.39	28.81	-98.17	3.00	0.10	3.01	PDE
1993	9	29	2	1	19.06	35.87	-102.98	3.30	0.10	3.31	PDE
1993	10	19	16	59	52.41	36.5460	-98.1730	3.10	0.10	3.11	OGS
1993	11	30	03	07	36.28	35.8088	-103.1567	3.26	0.30	3.37	NMT
1993	12	05	00	58	24.06	27.9877	-102.0607	4.03	0.30	4.13	NMT
1993	12	05	03	35	14.14	27.8975	-102.0582	3.43	0.30	3.53	NMT
1994	4	16	7	20	29.99	34.6630	-97.7130	3.10	0.10	3.11	OGS

CP COL 2.5(1)

Table 2.5.2-201 (Sheet 3 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1994	4	29	3	28	58.68	36.25	-98.09	3.00	0.10	3.01	PDE
1995	1	18	15	51	39.90	34.7120	-97.5420	4.20	0.10	4.21	OGS
1995	4	5	5	31	16.23	35.20	-99.03	3.00	0.10	3.01	PDE
1995	4	14	0	32	56.17	30.28	-103.35	5.71	0.10	5.71	PDE
1995	4	14	2	19	38.50	30.30	-103.35	3.30	0.10	3.31	PDE
1995	4	15	14	33	29.51	30.27	-103.32	4.00	0.10	4.01	PDE
1995	6	1	1	6	15.70	30.30	-103.35	3.50	0.10	3.51	PDE
1995	6	1	4	49	27.70	34.1340	-96.6830	3.30	0.10	3.31	OGS
1995	9	15	0	31	33.26	36.87	-98.69	4.10	0.10	4.11	PDE
1995	11	12	17	45	59.40	30.30	-103.35	3.60	0.10	3.61	PDE
1995	12	1	14	37	43.00	35.1550	-98.8970	3.00	0.10	3.01	OGS
1996	3	25	6	43	46.86	35.61	-102.60	3.50	0.10	3.51	PDE
1996	11	23	10	54	18.50	35.0400	-100.5040	3.09	0.41	3.28	ANSS
1997	2	12	23	53	10.77	34.9470	-100.8900	3.09	0.41	3.28	ANSS
1997	2	15	9	8	55.46	34.9730	-100.5690	3.25	0.41	3.45	ANSS
1997	3	16	19	7	28.00	34.2700	-93.4900	3.42	0.41	3.61	ANSS
1997	5	31	3	26	41.34	33.1820	-95.9660	3.42	0.41	3.61	ANSS

CP COL 2.5(1)

Table 2.5.2-201 (Sheet 4 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
1997	9	6	23	38	0.91	34.66	-96.43	4.50	0.10	4.51	PDE
1997	9	6	23	38	1.99	34.6760	-96.4990	4.40	0.10	4.41	OGS
1997	10	19	11	12	09.74	32.3347	-103.9360	3.11	0.30	3.21	NMT
1998	1	2	15	47	16.43	37.8280	-103.4080	3.42	0.41	3.61	ANSS
1998	4	15	10	33	42.42	30.19	-103.30	3.60	0.10	3.61	PDE
1998	4	27	15	22	46.25	35.4530	-102.3830	3.25	0.41	3.45	ANSS
1998	4	28	14	13	1.27	34.7550	-98.4470	4.20	0.10	4.21	OGS
1998	7	7	18	44	44.46	34.7190	-97.5890	3.25	0.41	3.45	ANSS
1998	7	14	5	38	48.75	35.3440	-103.4730	3.01	0.41	3.20	ANSS
1998	10	30	17	41	21.42	36.7710	-97.6230	3.50	0.10	3.51	OGS
1998	10	30	17	41	22.20	36.8000	-97.6000	3.50	0.41	3.69	ANSS
1999	10	25	23	19	51.68	36.9462	-100.0700	3.00	0.10	3.01	OGS
1999	10	25	23	19	58.37	36.8460	-99.6590	3.09	0.41	3.28	ANSS
2000	1	14	10	39	34.94	34.6735	-95.0949	3.00	0.10	3.01	OGS
2000	8	7	17	19	8.00	35.3920	-101.8120	3.33	0.41	3.53	ANSS
2000	8	7	18	34	9.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS
2000	8	7	21	36	21.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS

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Table 2.5.2-201 (Sheet 5 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
2000	8	10	13	39	50.00	35.3920	-101.8120	3.09	0.41	3.28	ANSS
2000	8	17	1	8	5.45	35.39	-101.81	3.90	0.10	3.91	PDE
2000	12	16	22	8	54.00	35.40	-101.80	3.90	0.10	3.91	PDE
2001	3	30	17	13	55.60	37.9330	-93.3270	3.17	0.41	3.37	ANSS
2001	6	2	1	55	53.72	32.3340	-103.1410	3.33	0.41	3.53	ANSS
2001	7	24	14	2	35.00	37.7000	-97.0000	3.09	0.41	3.28	ANSS
2001	8	4	1	13	28.00	34.4200	-93.2300	3.25	0.41	3.45	ANSS
2001	11	22	0	7	8.02	31.7860	-102.6310	3.17	0.41	3.37	ANSS
2002	2	8	16	7	13.84	34.6514	-98.3021	3.80	0.10	3.81	OGS
2002	5	31	9	57	9.87	34.9997	-97.6228	3.00	0.10	3.01	OGS
2002	5	31	9	57	10.02	34.0250	-97.6190	3.33	0.41	3.53	ANSS
2002	6	19	12	14	20.30	36.57	-103.03	3.70	0.10	3.71	PDE
2002	10	20	2	18	14.06	34.2140	-96.1810	3.60	0.10	3.61	OGS
2003	4	7	10	2	12.51	33.8920	-97.6950	3.01	0.41	3.20	ANSS
2003	9	24	15	2	9.09	35.2770	-101.7420	3.33	0.41	3.53	ANSS
2004	4	22	16	13	2.25	34.8040	-97.6770	3.01	0.41	3.20	ANSS
2004	6	8	0	15	8.38	34.0410	-97.3070	3.70	0.10	3.71	OGS

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**Table 2.5.2-201 (Sheet 6 of 6)
Updated Seismicity Catalog for CPNPP 3 & 4 With Time of Event, Location of Event, Best Estimate Body-wave Magnitude (Emb), Estimate of Standard Deviation of Magnitude (Smb), Uniform Magnitude (Rmb), and Source Catalog**

Year	Mon	Day	Hr	Min	Sec	Lat	Lon	Emb	Smb	Rmb	Cat
2004	6	8	0	15	9.99	34.23	-97.25	3.50	0.10	3.51	PDE
2004	6	10	12	30	9.86	34.2360	-97.2670	3.01	0.41	3.20	ANSS
2004	11	22	23	42	13.45	34.8640	-97.6720	3.09	0.41	3.28	ANSS
2004	11	30	23	59	34.00	36.9400	-93.8900	3.01	0.41	3.20	ANSS
2005	2	6	15	59	14.48	34.2380	-95.2380	3.50	0.10	3.51	OGS
2005	4	3	14	39	16.97	28.3930	-100.3050	3.50	0.41	3.69	ANSS
2005	4	22	5	17	4.09	34.1790	-95.1920	3.09	0.41	3.28	ANSS
2006	2	18	5	49	41.45	35.6720	-101.7940	3.50	0.41	3.69	ANSS
2006	3	28	23	55	11.49	35.3630	-101.8710	3.09	0.41	3.28	ANSS
2006	4	5	18	46	23.14	34.0690	-97.3140	3.09	0.41	3.28	ANSS
2006	4	8	18	8	35.23	31.9540	-101.4190	3.01	0.41	3.20	ANSS
2006	10	6	22	13	16.78	34.12	-97.62	3.50	0.10	3.51	PDEW

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Table 2.5.2-202 (Sheet 1 of 2)
Summary of Bechtel Group Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	Pa[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
39	Oklahoma Aulacogen	143	89	0.20	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 3 [0.33]	Yes
BZ2	Texas Platform	0	0	1.0	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 3 [0.33]	Yes
38	Ouachita	205	125	0.25	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 4 [0.33]	Yes
BZ3	North Great Plains	143	89	1.0	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 3 [0.33]	Yes
C04	Combination Zone	143	89	NA	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 4 [0.33]	Yes
40	Meers Fault	268	166	0.70	5.4 [0.1] 6.0 [0.4] 6.6 [0.4] 7.5 [0.1]	1 [0.33] 2 [0.34] 4 [0.33]	NA - replaced

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Table 2.5.2-202 (Sheet 2 of 2)
Summary of Bechtel Group Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	Pa[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
65	El Reno	315	196	0.35	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 4 [0.33]	No
BZ1	Gulf Coast	219	136	1.0	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 3 [0.33]	No
55	S.E. Oklahoma	235	146	0.15	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	1 [0.33] 2 [0.34] 4 [0.33]	No

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI, 1989a).

d) Smoothing options (EPRI, 1989a):
1 = constant a, constant b, no b prior;
2 = low smoothing on a, high smoothing on b, no b prior;
3 = low smoothing on a, low smoothing on b, no b prior;
4 = low smoothing on a, low smoothing on b, weak b prior of 1.05;
Weights on magnitude intervals are [1.0, 1.0, 1.0, 1.0, 1.0, 1.0, 1.0].

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

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Table 2.5.2-203 (Sheet 1 of 2)
Summary of Dames & Moore Seismic Source Zones

Source	Description	Distance[(a)] (km)	(mi)	Pa[(b)]	M_{max} (m_b) and Wts.[(c)]	Smoothing Options and Wts.[(d)]	Contributes to 99% of Hazard[(e)]
20	Southern Coastal Margin	134	83	1.0	5.3 [0.8] 7.2 [0.2]	1 [0.75] 2 [0.25]	Yes
25	Ouachitas Fold Belt	42	26	0.35	5.5 [0.8] 7.2 [0.2]	1 [0.75] 2 [0.25]	Yes
25a	Kink in Ouachita Fold Belt	121	75	0.65	5.7 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	Yes
28	S. Oklahoma Aulacogen	147	91	0.44	6.0 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	Yes
28b	Default for S. Oklahoma Aulacogen	113	70	0.56	5.0 [0.8] 7.2 [0.2]	1 [0.75] 2 [0.25]	Yes
67	New Mexico	0	0	1.0	5.5 [0.8] 7.2 [0.2]	1 [0.75] 2 [0.25]	Yes
C08	Combination Zone	42	26	NA	5.5 [0.8] 7.2 [0.2]	1 [0.75] 2 [0.25]	Yes
29	B-W-M Fault	160	100	0.31	6.0 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	No
30	A/W Uplift	170	110	0.42	6.0 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	No

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Table 2.5.2-203 (Sheet 2 of 2)
Summary of Dames & Moore Seismic Source Zones

Source	Description	Distance[(a)] (km)	Distance[(a)] (mi)	Pa[(b)]	M_{max} (m_b) and Wts.[(c)]	Smoothing Options and Wts.[(d)]	Contributes to 99% of Hazard[(e)]
32	Ardmore Basin	230	140	0.51	6.0 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	No
33	Anadarko Basin	266	165	1.0	5.8 [0.75] 7.2 [0.25]	1 [0.34] 2 [0.11] 3 [0.41] 4 [0.14]	No
31	Mt. View/Meers	210	130	0.45	6.0 [0.75] 7.2 [0.25]	3 [0.75] 4 [0.25]	NA - replaced

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI 1989a).

d) Smoothing options (EPRI, 1989a):

1 = no smoothing on a, no smoothing on b, strong b prior of 1.04:

2 = no smoothing on a, no smoothing on b, weak b prior of 1.04;

3 = constant a, constant b, strong b prior of 1.04;

4 = constant a, constant b, weak b prior of 1.04;

Weights on magnitude intervals are [0.1, 0.2, 0.4, 1.0, 1.0, 1.0, 1.0].

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

CP COL 2.5(1)

Table 2.5.2-204
Summary of Law Engineering Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	Pa[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
124	New Mexico –Texas Block	0	0	1.0	4.9 [0.3] 5.5 [0.5] 5.8 [0.2]	1a [1.0]	Yes
26	Oklahoma Aulacogen-Arbuckle Wichita Rift	150	93	0.6	5.0 [0.2] 5.2 [0.5 6.8 [0.3]	1a [1.0]	Yes
119	Eastern Mid-Continent	151	94	1.0	4.6 [0.3] 5.0 [0.3] 5.5 [0.4]	1a [1.0]	No
120	Western Mid-Continent	300	190	1.0	4.9 [0.5] 5.5 [0.5]	3a [1.0]	No
126	South Coastal Block	148	92	1.0	4.6 [0.9] 4.9 [0.1]	1a [1.0]	No

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI, 1989a).

d) Smoothing options (EPRI, 1989a):

1a = high smoothing on a, constant b, strong b prior of 1.05;

Weights on magnitude intervals are all 1.0.

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

CP COL 2.5(1)

Table 2.5.2-205
Summary of Rondout Associates Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	Pa[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
16	S. Oklahoma Aulacogen-Ouachita Mts.	129	80	1.0	5.8 [0.15] 6.5 [0.60] 6.8 [0.25]	1 [1.0]	Yes
C02	Grenville Crust	0	0	NA	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	3 [1.0]	Yes
23	Nemaha-Anadark	230	140	1.0	6.6 [0.2] 6.8 [0.6] 7.0 [0.2]	1 [1.0]	Yes
51	Gulf Coast to Bahamas Fracture Zone	92	57	1.0	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	3 [1.0]	Yes
52	Pre-Grenville Precambrian Craton	290	180	1.0	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	3 [1.0]	No

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI, 1989a).

d) Smoothing options (EPRI, 1989a):

1 = constant a of -1.590, constant b of 1.020

2 = constant a of -1.350, constant b of 0.960

3 = low smoothing on a, constant b, strong b prior of 1.0.

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

CP COL 2.5(1)

Table 2.5.2-206
Summary of Weston Geophysical Corporation Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	P*[b]	M_{max} (Em_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
109	Southwest	0	0	1.0	5.4 [0.33] 6.0 [0.49] 6.6 [0.18]	1a [0.2] 2a [0.8]	Yes
C31	Combination Zone	0	0	NA	5.4 [0.33] 6.0 [0.49] 6.6 [0.18]	1a [0.7] 2a [0.3]	Yes
36	Ancestral Rockies	137	85	1.0	5.4 [0.43] 6.0 [0.41] 6.6 [0.16]	1b [0.3] 2b [0.7]	Yes
107	Gulf Coast	128	79	1.0	5.4 [0.71] 6.0 [0.29]	1a [0.2] 2a [0.8]	Yes
37	Delaware Basin	230	140	0.81	5.4 [0.33] 6.0 [0.49] 6.6 [0.18]	1b [0.3] 2b [0.7]	No

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity for earthquakes with magnitudes greater than the minimum magnitude of mb 5.0 (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI, 1989a).

d) Smoothing options (EPRI, 1989a):

1a = constant a, constant b, medium b prior of 1.0;

1b = constant a, constant b, medium b prior of 0.9;

2a = medium smoothing on a, medium smoothing on b, medium b prior of 1.0.

2b = medium smoothing on a, medium smoothing on b, medium b prior of 0.9.

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

CP COL 2.5(1)

Table 2.5.2-207 (Sheet 1 of 2)
Summary of Woodward-Clyde Consultants Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	P*[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
BG44	Central US Backgrounds	0	0	NA	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	1 [0.25] 6 [0.25] 7 [0.25] 8 [0.25]	Yes
46	S. Oklahoma Aulacogen	161	100	0.084	5.7 [0.33] 6.8 [0.34] 7.2 [0.33]	3 [0.33] 4 [0.34] 5 [0.33]	Yes
46a	S. Oklahoma Aulacogen	161	100	0.083	5.7 [0.33] 6.8 [0.34] 7.2 [0.33]	3 [0.33] 4 [0.34] 5 [0.33]	Yes
49	Meers Fault	262	163	0.85	6.8 [0.33] 7.3 [0.34] 7.5 [0.33]	2^+ [1.0]	NA - replaced
52	E. Oklahoma Seismic Zone	238	148	0.4	5.4 [0.33] 6.0 [0.34] 6.5 [0.33]	3 [0.33] 4 [0.34] 5 [0.33]	No

CP COL 2.5(1)

Table 2.5.2-207 (Sheet 2 of 2)
Summary of Woodward-Clyde Consultants Seismic Source Zones

Source	Description	Distance[a] (km)	Distance[a] (mi)	P*[b]	M_{max} (m_b) and Wts.[c]	Smoothing Options and Wts.[d]	Contributes to 99% of Hazard[e]
48	S. Oklahoma Gravity Anomaly	211	131	0.263	5.7 [0.33] 6.5 [0.34] 7.1 [0.33]	3 [0.33] 4 [0.34] 5 [0.33]	Yes

a) Shortest distance between CPNPP 3 & 4 and source zone.

b) Probability of activity for earthquakes with magnitudes greater than the minimum magnitude of mb 5.0 (EPRI, 1989a).

c) Maximum earthquake magnitude (M_{max}) in body-wave magnitude (m_b) and weighting (Wts.) (EPRI, 1989a).

d) Smoothing options (EPRI, 1989a):

1 = low smoothing on a, high smoothing on b, no b prior;

3 = high smoothing on a, high smoothing on b, moderate b prior of 1.0.

4 = high smoothing on a, high smoothing on b, moderate b prior of 0.9.

5 = high smoothing on a, high smoothing on b, moderate b prior of 0.8.

6 = low smoothing on a, high smoothing on b, moderate b prior of 1.0;

7 = low smoothing on a, high smoothing on b, moderate b prior of 0.9;

8 = low smoothing on a, high smoothing on b, moderate b prior of 0.8;

9 = use “a” and “b” from homogeneous solution for source zone 46 with smoothing option 4.

Weights on magnitude intervals are all 1.0.

e) Whether or not the source contributes to 99% of the hazard at CPNPP Units 3 and 4.

CP COL 2.5(1)

Table 2.5.2-208
Comparison of PGA Hazard Results

PGA comparison

Ampl.	Mean			Median			0.85 fractile		
(cm/s^2)	EPRI-SOG	2007	% diff	EPRI-SOG	2007	% diff	EPRI-SOG	2007	% diff
50	4.26E-05	4.59E-05	7.7%	1.91E-05	2.40E-05	25.6%	8.71E-05	9.55E-05	9.6%
100	1.06E-05	1.16E-05	9.4%	4.62E-06	6.92E-06	49.7%	1.83E-05	2.09E-05	14.2%
250	1.23E-06	1.38E-06	12.4%	4.60E-07	7.08E-07	53.9%	2.02E-06	2.07E-06	2.2%
500	1.41E-07	1.64E-07	16.4%	3.17E-08	5.89E-08	85.7%	2.26E-07	2.34E-07	3.7%

CP COL 2.5(1)

Table 2.5.2-209
Comparison of 1 Hz SV Hazard Results

1 Hz SV comparison

Ampl. (cm/s)	Mean			Median			0.85 fractile		
	EPRI-SOG	2007	% diff	EPRI-SOG	2007	% diff	EPRI-SOG	2007	% diff
1	2.50E-04	2.60E-04	4.0%	3.96E-05	5.31E-05	34.1%	4.53E-04	3.43E-04	-24.3%
5	1.42E-05	1.56E-05	9.9%	4.15E-07	9.02E-07	117.3%	1.15E-05	1.16E-05	1.0%
10	3.08E-06	3.50E-06	13.5%	3.86E-08	1.26E-07	226.2%	2.27E-06	3.02E-06	33.0%
20	5.74E-07	6.66E-07	16.0%	9.08E-10	7.16E-09	688.7%	4.15E-07	5.37E-07	29.4%

CP COL 2.5(1)

Table 2.5.2-210
Mmax Update for EPRI Team Sources

Team	Source Zone	Original Mmax Distribution and Weights (EPRI, 1989)	Updated Mmax Distribution and Weights
Bechtel	Background (BZI)	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	6.1 [0.1] 6.4 [0.4] 6.6 [0.1] 6.7 [0.4]
Dames & Moore	South Coastal Margin (zone 20)	5.3 [0.8] 7.2 [0.2]	5.5 [0.8] 7.2 [0.2]
Law Engineering	New Mexico-Texas Block (zone 124)	4.9 [0.3] 5.5 [0.5] 5.8 [0.2]	5.0 [0.3] 5.5 [0.5] 5.8 [0.2]
Law Engineering	South Coastal Block (zone 126)	4.6 [0.9] 4.9 [0.1]	5.5 [0.9] 5.7 [0.1]
Rondout	Gulf Coast to Bahamas Fracture zone (zone 51)	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	6.1 [0.3] 6.3 [0.55] 6.5 [0.15]
Weston	Gulf Coast (zone 107)	5.4 [0.71] 6.0 [0.29]	6.6 [0.89] 7.2 [0.11]

Table 2.5.2-211 Deleted

CP COL 2.5(1)

Table 2.5.2-212
Meers Fault Characterization from 2002 USGS National Seismic Hazard Maps (Frankel et al., 2002)

Probability of Activity	1
Recurrence Model	Characteristic
Characteristic Magnitude	Mw 7.0
Characteristic Return Period	4545 years
Dip	89°
Dip Direction	SW
Sense of Slip	Strike slip
Rupture Top	0 km
Rupture Bottom	15 km
Width	15 km
Length	35 km
Fault Trace Coordinates (Lat., Lon.)	(34.85°, -98.64°) (34.75°, -98.40°) (34.73°, -98.33°) (34.71°, -98.29°)

CP COL 2.5(1)

Table 2.5.2-213
Updated Seismic Source Characterization of the Meers Fault

Probability of Activity	1
Recurrence Model	Characteristic
Characteristic Magnitude	6.7 [0.2][a], 6.85 [0.6][a], 7.0 [0.2][a]
Characteristic Return Period	See logic tree in Figure 4 of TXUT-001-PR-003
Dip	89°
Dip Direction	SW
Rupture Top	0 km
Rupture Bottom	15 to 20 km
Width	15 to 20 km
Length	26 to 37 km
Fault Trace Coordinates (Lat., Lon.)	(34.85°, -98.64°) (34.71°, -98.29°)

a) [ ] = percentage % of 100 for each magnitude weighted in the model

CP COL 2.5(1)

Table 2.5.2-214 (Sheet 1 of 2)
Rio Grande Rift Faults Modeled as Discrete Fault Sources

Fault Name	Recurrence Rate (EQs/yr)	Magnitude (Mw)
Puye fault	4.0140E-05	6.6
Sawyer Canyon fault	5.4280E-05	6.2
La Canada del Amagre fault zone	9.5530E-05	6.5
Embudo fault	3.7700E-05	7.2
Lobato Mesa fault zone	6.3390E-05	6.6
Canones fault	2.0724E-05	6.8
Black Mesa fault zone	3.4270E-05	6.5
Gallina fault	1.8790E-05	6.9
Southern Sangre de Cristo fault	5.7220E-05	7.4
Northern Sangre de Cristo fault	1.0040E-04	7.5
Southern Sawatch fault	4.6820E-05	7.0
West Lobo Valley fault zone	1.7700E-05	7.2
West Indio Mountains fault	4.8600E-05	6.7
Caballo fault	7.8790E-05	7.0
West Eagle Mountains-Red Hills fault	1.5140E-05	6.7
Amargosa fault	6.5170E-05	7.2
East Baylor Mountain - Carizzo Mountain fault	5.3200E-06	7.0
Arroyo Diablo fault	2.4520E-05	6.4
East Sierra Diablo fault	1.6510E-05	6.9
Campo Grande fault	3.6540E-05	7.0
Acala fault	2.4770E-04	6.1
West Delaware Mountains fault zone	2.8590E-05	6.7
East Franklin Mountains fault	8.1530E-05	7.0
Organ Mountains fault	1.4976E-04	6.8
San Andres Mountains fault	3.9120E-05	7.5
Alamogordo fault	3.9970E-05	7.5
Caballo fault	3.7440E-05	6.6
La Jencia fault	2.3120E-05	6.8
Hubbell Springs fault	5.3650E-05	7.0
Tijeras-Canoncito fault	3.2820E-05	7.3
County Dump fault	3.3260E-05	6.9

CP COL 2.5(1)

Table 2.5.2-214 (Sheet 2 of 2)
Rio Grande Rift Faults Modeled as Discrete Fault Sources

Fault Name	Recurrence Rate (EQs/yr)	Magnitude (Mw)
Zia fault	4.2010E-05	6.8
San Francisco fault	6.6380E-05	6.8
San Felipe fault zone	3.1180E-05	7.0
La Bajada fault	4.9530E-05	7.0
Jemez-San Ysidro fault	1.2850E-05	7.1
Picuris-Pecos fault	2.1030E-05	7.4
Nacimiento fault	9.9400E-06	7.3
Nambe fault	1.6790E-05	7.0
Pajarito fault	5.7380E-05	7.0
Pojoaque fault	1.6260E-05	7.0

CP COL 2.5(1)

Table 2.5.2-215 (Sheet 1 of 2)
Surface Trace Coordinates of Rio Grande Rift Faults

Fault Name	Longitude 1	Latitude 1	Longitude 2	Latitude 2
Puye fault	-106.158	35.893	-106.154	36.064
Sawyer Canyon fault	-106.254	35.908	-106.281	35.979
La Canada del Amagre fault zone	-106.242	36.023	-106.211	36.170
Embudo fault	-105.599	36.329	-106.224	36.035
Lobato Mesa fault zone	-106.276	36.207	-106.300	36.041
Canones fault	-106.529	36.081	-106.319	36.284
Black Mesa fault zone	-105.963	36.220	-106.121	36.125
Gallina fault	-106.901	36.220	-106.791	36.525
Southern Sangre de Cristo fault	-105.503	37.178	-105.597	36.328
Northern Sangre de Cristo fault	-105.994	38.393	-105.369	37.006
Southern Sawatch fault	-106.245	38.563	-106.211	38.930
West Lobo Valley fault zone	-104.604	30.466	-104.807	30.939
West Indio Mountains fault	-105.029	30.667	-105.136	30.838
Caballo fault	-105.527	31.095	-105.284	30.779
West Eagle Mountains-Red Hills fault	-105.269	31.003	-105.085	30.857
Amargosa fault	-105.555	30.874	-106.047	31.314
East Baylor Mountain - Carizzo Mountain fault	-104.905	30.952	-104.723	31.285
Arroyo Diablo fault	-105.720	31.306	-105.637	31.202
East Sierra Diablo fault	-104.873	31.224	-104.871	31.517
Campo Grande fault	-106.033	31.495	-105.629	31.292
Acala fault	-105.938	31.411	-105.888	31.360

CP COL 2.5(1)

Table 2.5.2-215 (Sheet 2 of 2) Surface Trace Coordinates of Rio Grande Rift Faults

Fault Name	Longitude 1	Latitude 1	Longitude 2	Latitude 2
West Delaware Mountains fault zone	-104.819	31.669	-104.716	31.467
East Franklin Mountains fault	-106.487	31.605	-106.447	32.011
Organ Mountains fault	-106.490	32.191	-106.486	32.417
San Andres Mountains fault	-106.486	32.417	-106.412	33.437
Alamogordo fault	-106.120	33.493	-105.924	32.520
Caballo fault	-107.266	33.114	-107.253	32.923
La Jencia fault	-107.074	34.011	-107.166	34.263
Hubbell Springs fault	-106.509	34.998	-106.563	34.616
Tijeras-Canoncito fault	-105.881	35.479	-106.507	34.987
County Dump fault	-106.775	35.008	-106.749	35.326
Zia fault	-106.843	35.189	-106.748	35.471
San Francisco fault	-106.321	35.488	-106.470	35.292
San Felipe fault zone	-106.607	35.312	-106.584	35.683
La Bajada fault	-106.302	35.702	-106.214	35.346
Jemez-San Ysidro fault	-106.788	35.420	-106.634	35.833
Picuris-Pecos fault	-105.609	36.329	-105.879	35.479
Nacimiento fault	-106.857	35.485	-106.901	36.220
Nambe fault	-105.883	36.021	-105.852	35.591
Pajarito fault	-106.297	35.646	-106.225	36.034
Pojoaque fault	-106.004	36.088	-106.062	35.671

CP COL 2.5(1)

Table 2.5.2-216
Summary of Rio Grand Rift Fault Source Characterization

Trace Coordinates	Table 2.5.2-215
Dip, Dip Direction	90°, NA
Recurrence Model	Characteristic Earthquake
Recurrence Rate (EQs/yr)	Table 2.5.2-CF12
Magnitude (Mw) and weights	Take magnitude from Table 2.5.2-214 and use Mw –0.2 [0.2][(a)], Mw [0.6][(a)], Mw +0.2 [0.2][(a)] with weights in parentheses
Probability of Activity	1.0

a) [ ] = percentage % of 100 for each magnitude weighted in the model

CP COL 2.5(1)

Table 2.5.2-217
Rio Grande Rift Point Source Characterization

Point location (Lon., Lat.)	(-102.671°, 29.796°)
Recurrence Model	Characteristic Earthquake
Return Period (yrs) and weights	14,500 [0.4][(a)], 37,500 [0.4][(a)], 119,000 [0.2][(a)]
Magnitude (Mw) and weights	6.3 [0.1][(a)], 6.65 [0.3][(a)], 6.95 [0.4][(a)], 7.3 [0.2][(a)]
Probability of Activity	1.0

a) [ ] = percentage % of 100 for each magnitude or period weighted in the model

CP COL 2.5(1)

Table 2.5.2-218
Cheraw Fault Source Characterization

Trace Coordinates (Lon., Lat.)	(-103.22°, 38.43°), (-103.59°, 38.15°)
Dip, Dip Direction	90°, NA
Recurrence Model	Characteristic Earthquake
Recurrence Rate	1.148e-4 per year
Magnitude (Mw) and weights	6.8 [0.2][a], 7.0 [0.6][a], 7.2 [0.2][a]
Probability of Activity	1.0

a) [ ] = percentage % of 100 for each magnitude weighted in the model

CP COL 2.5(1)

Table 2.5.2-219
Values of Mean and Median Rock UHRS (in g) for 10^{-4} and 10^{-5}

freq	10^{-4} mean UHRS	10^{-4} median UHRS	10^{-5} mean UHRS	10^{-5} median UHRS
100	0.0516	0.0353	0.127	0.0815
25	0.0127	0.0728	0.370	0.193
10	0.105	0.0810	0.263	0.187
5	0.0944	0.0743	0.222	0.163
2.5	0.0761	0.0543	0.173	0.113
1	0.0500	0.0277	0.123	0.0554
0.5	0.0380	0.0155	0.116	0.0301

CP COL 2.5(1)

Table 2.5.2-220
Mean Magnitudes and Distances from Deaggregation

	1E-4, 5 and 10 Hz	1E-4, 1 and 2.5 Hz	1E-5, 5 and 10 Hz	1E-5, 1 and 2.5 Hz	1E-6, 5 and 10 Hz	1E-6, 1 and 2.5 Hz
M	6.9	7.3	6.7	7.4	6.1	7.4
R	300	540	180	550	46	470
M (r >100 km)	7.0	7.3	7.1	7.5	7.2	7.6
R (r >100 km)	400	570	430	630	440	680

CP COL 2.5(1)

Table 2.5.2-221
Deaggregation of 10^{-4} High Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	3.3	0.2	0.1	0.0	0.0	0.0	0.0	0.0
20-40 km	3.1	0.3	0.1	0.0	0.0	0.0	0.0	0.0
40-60 km	1.4	0.3	0.1	0.0	0.0	0.0	0.0	0.0
60-80 km	0.6	0.2	0.1	0.0	0.0	0.0	0.0	0.0
80-100 km	0.4	0.2	0.1	0.0	0.0	0.0	0.0	0.0
100-200 km	1.3	1.0	1.0	0.3	0.2	0.0	0.0	0.0
200-300 km	0.3	0.4	0.7	51.0	0.2	0.0	0.0	0.0
>300 km	0.0	0.1	0.2	0.3	6.4	24.3	1.6	0.0

Table 2.5.2-222
CP COL 2.5(1)
Deaggregation of 10^{-4} Low Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	0.8	0.1	0.0	0.0	0.0	0.0	0.0	0.0
20-40 km	0.3	0.1	0.1	0.0	0.0	0.0	0.0	0.0
40-60 km	0.1	0.1	0.1	0.0	0.0	0.0	0.0	0.0
60-80 km	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
80-100 km	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
100-200 km	0.0	0.1	0.3	0.2	0.1	0.0	0.0	0.0
200-300 km	0.0	0.1	0.2	32.4	0.1	0.0	0.0	0.0
>300 km	0.0	0.0	0.1	0.5	13.4	47.7	2.8	0.0

CP COL 2.5(1)

Table 2.5.2-223
Deaggregation of 10^{-5} High Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	15.0	1.3	0.5	0.0	0.0	0.0	0.0	0.0
20-40 km	5.0	1.2	0.6	0.1	0.0	0.0	0.0	0.0
40-60 km	0.9	0.4	0.4	0.1	0.1	0.0	0.0	0.0
60-80 km	0.2	0.2	0.2	0.1	0.1	0.0	0.0	0.0
80-100 km	0.1	0.1	0.2	0.1	0.1	0.0	0.0	0.0
100-200 km	0.3	0.4	0.8	0.4	0.4	0.1	0.0	0.0
200-300 km	0.1	0.1	0.3	39.2	0.3	0.0	0.0	0.0
>300 km	0.0	0.0	0.1	0.1	3.4	24.7	2.3	0.0

CP COL 2.5(1)

Table 2.5.2-224
Deaggregation of 10^{-5} Low Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	1.8	0.5	0.3	0.0	0.0	0.0	0.0	0.0
20-40 km	0.2	0.2	0.2	0.0	0.0	0.0	0.0	0.0
40-60 km	0.0	0.1	0.1	0.0	0.0	0.0	0.0	0.0
60-80 km	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
80-100 km	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
100-200 km	0.0	0.0	0.2	0.2	0.2	0.1	0.0	0.0
200-300 km	0.0	0.0	0.1	23.9	0.2	0.0	0.0	0.0
>300 km	0.0	0.0	0.0	0.2	9.3	57.5	4.3	0.0

CP COL 2.5(1)

Table 2.5.2-225
Deaggregation of 10^{-6} High Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	42.6	6.5	3.1	0.3	0.1	0.0	0.0	0.0
20-40 km	4.0	2.1	1.7	0.3	0.2	0.0	0.0	0.0
40-60 km	0.3	0.3	0.5	0.2	0.2	0.0	0.0	0.0
60-80 km	0.0	0.1	0.2	0.1	0.2	0.0	0.0	0.0
80-100 km	0.0	0.0	0.1	0.1	0.1	0.0	0.0	0.0
100-200 km	0.0	0.1	0.4	0.3	0.6	0.2	0.0	0.0
200-300 km	0.0	0.0	0.1	18.5	0.2	0.0	0.0	0.0
>300 km	0.0	0.0	0.0	0.0	1.0	13.3	1.9	0.0

CP COL 2.5(1)

Table 2.5.2-226
Deaggregation of 10^{-6} Low Frequencies

Percent contribution by M-R bin

	5.25	5.75	6.25	6.75	7.25	7.75	8.25	8.75
0-20 km	3.8	1.7	1.4	0.2	0.1	0.0	0.0	0.0
20-40 km	0.2	0.3	0.5	0.1	0.1	0.0	0.0	0.0
40-60 km	0.0	0.0	0.2	0.1	0.1	0.0	0.0	0.0
60-80 km	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0
80-100 km	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0
100-200 km	0.0	0.0	0.1	0.2	0.4	0.1	0.0	0.0
200-300 km	0.0	0.0	0.0	17.1	0.2	0.0	0.0	0.0
>300 km	0.0	0.0	0.0	0.1	5.7	60.5	6.2	0.0

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Table 2.5.2-227 (Sheet 1 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

			Stratigraphy			
			Top of Layer Depth from YG[3]	Mean Elv Top	Mean Elv Std Dev Top	Mean Thickness
	Unit	Lithology	(ft)	(MSL, ft)	(ft)	(ft)
	Fill Concrete	To be placed as needed from top of layer C	N/A	N/A	N/A	-
	Compacted Fill	Fill for excavation	0.0	822.0	N/A	3.0
			3.0	819.0	N/A	17.0
			20.0	802.0	N/A	20.0
	Fill/Residuum	Fill/Residuum/weathered limestone	-	847.0	N/A	-
Shallow Site Profile[1]	A	Limestone (will be removed)	-	834.0	12.1	36.0
	B1	Shale (will be removed)	24.0	798.0	1.8	8.0
	B2	Shale with limestone (will be removed)	32.0	790.0	1.8	8.0
	C	Limestone (foundation layer)	40.0	782.0	1.8	65.0
	D	Shale	105.0	717.0	1.5	3.0
	E1	Limestone	108.0	714.0	1.6	24.0
	E2	Limestone	132.0	690.0	1.0	34.0
	E3	Limestone	166.0	656.0	1.0	34.0
	F	Limestone with interbedded shales and sand	200.0	622.0	2.2	29.0
	G	Sandstone	229.0	593.0	4.0	80.0
	H	Shale	309.0	513.0	5.2	62.0
	I	Sandstone	371.0	451.0	3.3	63.0
	Strawn Group (MW)	Shales with sandstone and limestone beds	434.0	388.0	26.0	2202.0

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Table 2.5.2-227 (Sheet 2 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

Deep Site Profile [2]	Atoka Sand	Sands and shales interbedded	2636.0	-1814.0	417.0	1995.0
	Smithwick	Shale	4631.0	-3809.0	34.0	123.0
	Big Saline	Conglomerate and sandstones	4754.0	-3932.0	122.0	41.0
	Marble Falls	Limestone	4795.0	-3973.0	37.0	223.0
	Barnett	Shale	5018.0	-4196.0	145.0	247.0
	Ellenburger	Limestone	5265.0	-4443.0	73.0	>3000

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Table 2.5.2-227 (Sheet 3 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

		Velocity[4]						
		Vs			Vp			
		Mean Vs	+Variability[4]	-Variability[4]	Mean Vp	+Variability[4]	-Variability[4]	Poisson's
	Unit	(ft/sec)	(ft/sec)	(ft/sec)	(ft/sec)	(ft/sec)	(ft/sec)	Ratio[8]
	Fill Concrete	6800.0	7300.0	6300.0	-	-	-	0.20
		650.0	975.0	325.0	-	-	-	0.35
		800.0	1200.0	400.0	-	-	-	0.35
	Compacted Fill	1000.0	1500.0	500.0	-	-	-	0.35
	Fill/Residuum	-	-	-	-	-	-	-
Shallow Site Profile[1]	A	3548.0	4435.0	2661.0	8788.0	10985.0	6591.0	0.40
	B1	2609.0	3261.3	1956.8	6736.0	8420.0	5052.0	0.41
	B2	2716.0	3395.0	2037.0	7640.0	9550.0	5730.0	0.43
	C	5685.0	7106.3	4263.8	11324.0	14155.0	8493.0	0.33
	D	3019.0	3773.8	2264.3	8312.0	10390.0	6234.0	0.42
	E1	4943.0	6178.8	3707.3	10486.0	13107.5	7864.5	0.36
	E2	6880.0	8600.0	5160.0	13164.0	16455.0	9873.0	0.31
	E3	4042.0	5052.5	3031.5	9255.0	11568.8	6941.3	0.38
	F	3061.0	3826.3	2295.8	7927.0	9908.8	5945.3	0.41
	G	3290.0	4112.5	2467.5	7593.0	9491.3	5694.8	0.38
	H	3429.0	4286.3	2571.8	8188.0	10235.0	6141.0	0.39
	I	3092.0	3865.0	2319.0	7686.0	9607.5	5764.5	0.40
	Strawn Group (MW)	5546.0	6932.5	4159.5	10627.0	13283.8	7970.3	0.32

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Table 2.5.2-227 (Sheet 4 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

Deep Site Profile [2]	Atoka Sand	7642.0	10011.0	5273.0	13921.0	18236.5	9605.5	0.28
	Smithwick	5557.0	7279.7	3834.3	10894.0	14271.1	7516.9	0.32
	Big Saline	10247.0	13423.6	7070.4	18004.0	23585.2	12422.8	0.26
	Marble Falls	10520.0	13781.2	7258.8	19740.0	25859.4	13620.6	0.30
	Barnett	7783.0	10195.7	5370.3	12858.0	16844.0	8872.0	0.21
	Ellenburger	10906.0	14286.9	7525.1	20382.0	26700.4	14063.6	0.30

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Table 2.5.2-227 (Sheet 5 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

		Unit Weight[9]		Shear Modulus[10]	Minimum C_v for Shear Modulus[14]		G_{max} Variation		Low Strain D_s Damping[11]	Variation with Strain Relation	Low Strain D_c Damping[13]
		Wet	Dry	Mean			LB	UB			
	Unit	(pcf)	(pcf)	(ksi)	LB	UB	$[G_{max}/(1+C_v)]$ (ksi)	$[G_{max}x(1+C_v)]$ (ksi)	(%)	-	(%)
	Fill Concrete	150.0	140.0	1495.9	-	-	-	-	-	N/A	-
		125.0	-	11.4	-	-	-	-	1.5	Fig. 2.5.2-232[15]	0.8
		125.0	-	17.3	-	-	-	-	1.5	Fig. 2.5.2-232[15]	0.8
	Compacted Fill	125.0	-	27.0	-	-	-	-	1.1	Fig. 2.5.2-232[15]	0.6
	Fill/Residuum	-	-		-	-	-	-	-	-	-
Shallow Site Profile[1]	A	145.0	135.0	393.7	0.8	0.6	218.7	629.9	1.8	-	0.9
	B1	135.0	117.0	198.2	0.8	0.6	110.1	317.1	2.0	-	1.0
	B2	135.0	117.0	214.8	0.8	0.6	119.3	343.7	2.0	-	1.0
	C	155.0	148.0	1080.4	0.8	0.6	600.2	1728.6	1.8	-	0.9
	D	135.0	117.0	265.4	0.8	0.6	147.4	424.6	2.0	-	1.0
	E1	155.0	149.0	816.8	0.8	0.6	453.8	1306.9	1.8	-	0.9
	E2	155.0	149.0	1582.3	0.8	0.6	879.1	2531.7	1.8	-	0.9
	E3	150.0	142.0	528.5	0.8	0.6	293.6	845.6	1.8	-	0.9
	F	130.0	112.0	262.7	0.8	0.6	145.9	420.3	2.0	-	1.0
	G	135.0	120.0	315.1	0.8	0.6	175.1	504.2	2.0	-	1.0
	H	140.0	130.0	355.0	0.8	0.6	197.2	568.0	2.0	-	1.0
	I	145.0	132.0	299.0	0.8	0.6	166.1	478.4	2.0	-	1.0
	Strawn Group (MW)	150.0	-	995.0	0.8	0.6	552.8	1592.0	1.8	-	0.9

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Table 2.5.2-227 (Sheet 6 of 6)[16]
Dynamic Properties of Subsurface Rock Materials

Deep Site Profile [2]	Atoka Sand	150.0	-	1890.0	1.0	1.0	945.0	3780.0	1.0	-	0.5
	Smithwick	150.0	-	1000.0	1.0	1.0	500.0	2000.0	1.0	-	0.5
	Big Saline[12]	150.0	-	3400.0	1.0	1.0	1700.0	6800.0	1.0	-	0.5
	Marble Falls	150.0	-	3580.0	1.0	1.0	1790.0	7160.0	0.8	-	0.4
	Barnett	150.0	-	1960.0	1.0	1.0	980.0	3920.0	1.0	-	0.5
	Ellenburger	150.0	-	3850.0	1.0	1.0	1925.0	7700.0	0.8	-	0.4

Notes:

1 Shallow Site Profile derived from site specific data.

2 Deep Velocity Profile derived from regional wells.

3 Depth calculated from the difference between Yard Grade (822 ft MSL (Mean Sea Level)) and the average elevation of top of layer.

4 The selected Variability for Velocity is +/-25% for shallow profile, +/-50% for the compacted fill, +/-31% for deep profile, and +/-500 fps for fill concrete.

5 Yard Grade is the elevation to which the site will be cut = 822 ft MSL.

6 Foundation Unit is the top of Layer C on which all critical structures will be founded (either directly or backfilled with concrete).

7 Max and Min elevation tops not available for deep site profile, which yielded only one estimate for the top each horizon.

8 Poisson's Ratio for Shallow Site Profile calculated from Vs and Vp suspension measurements. Deep Site Profile values estimated from deep regional well Vp data.

9 Unit weight values for Layers A through G estimated based on results of the laboratory tests. Values for Layers H, I and Strawn (MW) estimated from FSAR Table 2.5.4-5G and based on lithology.

10 G_{max} calculated based on suspension Vs or estimated Vs for Deep Site Profile Materials.

11 Low Strain Damping Ratio in Shear estimated from lithology for Shallow Site Profile through discussion with Dr. Ken Stokoe. Deep Site Profile values based on comparison of Vs and lithology of shallow site layers.

12 Standard deviation in elevation of the top of Big Saline and top Atoka estimated from average standard deviation for other layer elevations.

13 Damping Ratio in unconstrained compression, D_c should be taken as $0.5D_s$ with a maximum value of 5%.

14 Recommended minimum C_v (shear modulus variation factor) values are based on +/-25% variation in V_s or Min values recommended by DCD (0.5 if test data are available or 1.0 if test data are not available), whichever is higher.

15 EPRI Curves shown on FSAR Figure 2.5.2-232 were used for non-linear response of the compacted fill layers.

16 The soil properties presented in Table 2.5.2-227 are site-specific for developing the site GMRS and FIRS for comparison to the CSDRS. The soil properties and variations for SSI analysis are discussed in FSAR Chapter 3 Appendix 3NN.

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Table 2.5.2-228
Values of Horizontal 10^{-5} UHRS and GMRS

Horizontal UHRS and GMRS values with site amplification (revised σ, CAV)

Freq	10^{-5}	GMRS
100	0.0826	0.0372
25	0.0928	0.0418
10	0.113	0.0509
5	0.121	0.0545
2.5	0.162	0.0729
1	0.100	0.0450
0.5	0.0789	0.0355

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Table 2.5.2-229
Values of Horizontal 10^{-5} UHRS and FIRS

Horizontal UHRS and FIRS values with site amplification (revised σ, CAV)

	FIRS2		FIRS3		FIRS4		FIRS4_CoV50	
Freq	10^{-5}	FIRS	10^{-5}	FIRS	10^{-5}	FIRS	10^{-5}	FIRS
100	0.0849	0.0382	0.101	0.0455	0.151	0.0680	0.148	0.0666
25	0.0980	0.0441	0.132	0.0594	0.194	0.873	0.190	0.0853
10	0.120	0.0540	0.208	0.0936	0.288	0.130	0.308	0.139
5	0.125	0.0563	0.149	0.0671	0.480	0.216	0.412	0.185
2.5	0.159	0.0716	0.177	0.0797	0.271	0.122	0.308	0.139
1	0.105	0.0473	0.118	0.0531	0.162	0.0729	0.170	0.0764
0.5	0.0830	0.0374	0.097	0.0437	0.132	0.0594	0.133	0.0597

Table 2.5.2-230
Calculation of Duration and Effective Strain Ratio for Rock Input Motions Considered in Site Response Calculations

Case	Magnitude M	Distance R (km)	Seismic Moment Mo (dyn-cm)	Corner Frequency fc (Hz)	Duration T (sec)	Eff Strain Ratio
1E-4 HF	6.9	300	2.51E+26	0.13	22.46	0.59
1E-4 BB	7.3	570	1.00E+27	0.08	40.32	0.63
1E-5 BB	7.4	620	1.41E+27	0.08	44.26	0.64
1E-6 BB	7.5	660	2.00E+27	0.07	47.88	0.65

Table 2.5.2-231
Amplification Factors for the GMRS/FIRS1 Site Column

	Amplification Factor for 10^{-4}		Amplification Factor for 10^{-5}		Amplification Factor for 10^{-6}	
Freq (Hz)	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.
0.1	1.10	0.06	1.10	0.06	1.10	0.06
0.125	1.14	0.08	1.14	0.08	1.14	0.08
0.15	1.18	0.11	1.19	0.11	1.19	0.11
0.2	1.30	0.16	1.30	0.16	1.30	0.16
0.3	1.46	0.17	1.46	0.18	1.46	0.18
0.4	1.43	0.17	1.43	0.17	1.43	0.17
0.5	1.37	0.17	1.37	0.17	1.37	0.17
0.6	1.36	0.16	1.36	0.16	1.36	0.16
0.7	1.37	0.14	1.37	0.14	1.38	0.14
0.8	1.39	0.11	1.40	0.12	1.40	0.12
0.9	1.39	0.10	1.39	0.10	1.39	0.10
1	1.41	0.12	1.38	0.11	1.37	0.11
1.25	1.60	0.16	1.61	0.17	1.61	0.17
1.5	1.75	0.19	1.75	0.19	1.74	0.19
2	1.71	0.13	1.71	0.13	1.71	0.13
2.5	1.44	0.16	1.42	0.15	1.41	0.14
3	1.12	0.17	1.12	0.17	1.12	0.16
4	0.83	0.16	0.84	0.15	0.83	0.15
5	0.74	0.15	0.75	0.14	0.74	0.14
6	0.72	0.17	0.73	0.17	0.71	0.17
7	0.66	0.20	0.66	0.19	0.64	0.21
8	0.59	0.20	0.60	0.19	0.57	0.21
9	0.56	0.19	0.56	0.19	0.52	0.21
10	0.55	0.20	0.55	0.19	0.51	0.22
12.5	0.54	0.26	0.54	0.26	0.49	0.30
15	0.52	0.24	0.51	0.25	0.47	0.29
20	0.42	0.18	0.39	0.19	0.34	0.24
25	0.37	0.15	0.33	0.16	0.27	0.20
30	0.35	0.14	0.32	0.14	0.26	0.17
35	0.35	0.13	0.32	0.13	0.26	0.16
40	0.36	0.12	0.33	0.12	0.26	0.14
45	0.37	0.11	0.34	0.11	0.27	0.13
50	0.39	0.11	0.36	0.11	0.29	0.13
60	0.44	0.10	0.42	0.10	0.34	0.11
70	0.53	0.10	0.52	0.10	0.41	0.11
80	0.63	0.10	0.63	0.10	0.50	0.11
90	0.73	0.10	0.73	0.10	0.59	0.11
100	0.79	0.10	0.81	0.10	0.66	0.11

Table 2.5.2-232
Amplification Factors for the FIRS2 Site Column

	Amplification Factor for 10^{-4}		Amplification Factor for 10^{-5}		Amplification Factor for 10^{-6}	
Freq (Hz)	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.
0.1	1.09	0.05	1.09	0.05	1.09	0.06
0.125	1.12	0.08	1.12	0.08	1.12	0.08
0.15	1.16	0.10	1.16	0.10	1.16	0.11
0.2	1.26	0.16	1.26	0.16	1.26	0.16
0.3	1.42	0.18	1.42	0.18	1.43	0.18
0.4	1.44	0.17	1.44	0.17	1.44	0.17
0.5	1.40	0.18	1.40	0.18	1.40	0.18
0.6	1.37	0.15	1.37	0.15	1.38	0.15
0.7	1.37	0.13	1.37	0.13	1.37	0.13
0.8	1.39	0.10	1.39	0.10	1.39	0.10
0.9	1.41	0.11	1.41	0.11	1.40	0.11
1	1.45	0.14	1.41	0.13	1.41	0.13
1.25	1.64	0.19	1.65	0.19	1.65	0.19
1.5	1.83	0.18	1.83	0.18	1.83	0.18
2	1.72	0.14	1.72	0.13	1.72	0.13
2.5	1.38	0.17	1.36	0.16	1.36	0.15
3	1.07	0.18	1.08	0.18	1.08	0.17
4	0.80	0.16	0.81	0.15	0.81	0.15
5	0.75	0.17	0.76	0.17	0.74	0.17
6	0.73	0.20	0.74	0.19	0.72	0.20
7	0.66	0.24	0.67	0.23	0.64	0.24
8	0.59	0.25	0.60	0.25	0.57	0.27
9	0.56	0.26	0.56	0.25	0.53	0.28
10	0.56	0.26	0.56	0.25	0.52	0.29
12.5	0.57	0.32	0.57	0.32	0.53	0.36
15	0.54	0.30	0.52	0.31	0.48	0.36
20	0.42	0.22	0.39	0.23	0.34	0.28
25	0.37	0.19	0.34	0.21	0.28	0.25
30	0.36	0.20	0.33	0.21	0.27	0.25
35	0.36	0.19	0.33	0.20	0.27	0.25
40	0.37	0.17	0.34	0.17	0.27	0.20
45	0.37	0.15	0.35	0.15	0.28	0.18
50	0.39	0.14	0.37	0.15	0.29	0.17
60	0.45	0.13	0.43	0.13	0.34	0.15
70	0.54	0.13	0.53	0.12	0.42	0.14
80	0.64	0.12	0.64	0.12	0.51	0.14
90	0.74	0.12	0.74	0.12	0.60	0.14
100	0.80	0.12	0.82	0.12	0.67	0.14

Table 2.5.2-233
Amplification Factors for the FIRS3 Site Column

	Amplification Factor for 10^{-4}		Amplification Factor for 10^{-5}		Amplification Factor for 10^{-6}	
Freq (Hz)	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.
0.1	1.09	0.07	1.09	0.07	1.09	0.07
0.125	1.13	0.10	1.13	0.10	1.13	0.10
0.15	1.17	0.14	1.17	0.14	1.17	0.14
0.2	1.26	0.19	1.26	0.19	1.26	0.19
0.3	1.39	0.19	1.39	0.19	1.39	0.19
0.4	1.39	0.16	1.39	0.16	1.39	0.16
0.5	1.37	0.17	1.36	0.17	1.36	0.17
0.6	1.35	0.16	1.35	0.16	1.35	0.16
0.7	1.35	0.13	1.35	0.13	1.36	0.13
0.8	1.40	0.11	1.40	0.11	1.40	0.11
0.9	1.44	0.12	1.43	0.12	1.43	0.12
1	1.46	0.14	1.41	0.13	1.41	0.13
1.25	1.60	0.20	1.61	0.20	1.60	0.20
1.5	1.78	0.18	1.78	0.18	1.77	0.18
2	1.65	0.15	1.66	0.15	1.66	0.1
2.5	1.35	0.23	1.34	0.21	1.34	0.20
3	1.10	0.22	1.11	0.21	1.10	0.21
4	0.84	0.18	0.85	0.17	0.85	0.17
5	0.80	0.21	0.81	0.20	0.80	0.20
6	0.79	0.23	0.80	0.22	0.79	0.23
7	0.77	0.29	0.77	0.28	0.76	0.29
8	0.74	0.33	0.75	0.32	0.72	0.34
9	0.76	0.37	0.77	0.37	0.74	0.39
10	0.81	0.38	0.82	0.38	0.79	0.40
12.5	0.88	0.35	0.88	0.35	0.86	0.37
15	0.74	0.36	0.72	0.37	0.69	0.41
20	0.57	0.33	0.55	0.35	0.51	0.40
25	0.46	0.26	0.42	0.28	0.37	0.33
30	0.41	0.22	0.37	0.23	0.32	0.27
35	0.40	0.21	0.37	0.22	0.31	0.25
40	0.41	0.20	0.38	0.21	0.31	0.24
45	0.42	0.20	0.39	0.20	0.32	0.23
50	0.44	0.19	0.41	0.19	0.34	0.22
60	0.50	0.18	0.48	0.17	0.40	0.20
70	0.60	0.17	0.59	0.17	0.49	0.19
80	0.71	0.17	0.71	0.16	0.59	0.19
90	0.82	0.16	0.83	0.16	0.69	0.19
100	0.89	0.16	0.92	0.16	0.77	0.19

Table 2.5.2-234
Amplification Factors for the FIRS4 Site Column

	Amplification Factor for 10^{-4}		Amplification Factor for 10^{-5}		Amplification Factor for 10^{-6}	
Freq (Hz)	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.
0.1	1.10	0.05	1.10	0.05	1.10	0.05
0.125	1.13	0.07	1.13	0.07	1.13	0.07
0.15	1.18	0.10	1.18	0.10	1.18	0.10
0.2	1.28	0.15	1.28	0.15	1.29	0.15
0.3	1.45	0.18	1.45	0.18	1.46	0.18
0.4	1.43	0.17	1.44	0.17	1.45	0.17
0.5	1.37	0.17	1.37	0.17	1.39	0.18
0.6	1.35	0.15	1.36	0.16	1.38	0.17
0.7	1.39	0.14	1.39	0.14	1.43	0.16
0.8	1.45	0.13	1.46	0.14	1.50	0.17
0.9	1.49	0.14	1.50	0.14	1.55	0.20
1	1.54	0.15	1.51	0.15	1.58	0.22
1.25	1.80	0.19	1.83	0.19	1.94	0.25
1.5	1.98	0.22	2.03	0.23	2.15	0.30
2	1.93	0.16	2.01	0.20	2.14	0.26
2.5	1.63	0.25	1.70	0.29	1.79	0.31
3	1.42	0.32	1.52	0.36	1.58	0.35
4	1.50	0.49	1.53	0.44	1.52	0.40
5	1.85	0.49	1.76	0.44	1.55	0.40
6	2.00	0.41	1.77	0.40	1.43	0.45
7	1.80	0.41	1.55	0.44	1.23	0.49
8	1.54	0.44	1.32	0.46	1.06	0.51
9	1.31	0.44	1.13	0.43	0.90	0.45
10	1.12	0.36	0.99	0.34	0.79	0.36
12.5	0.99	0.29	0.88	0.29	0.70	0.31
15	0.94	0.31	0.81	0.30	0.64	0.34
20	0.76	0.31	0.63	0.32	0.48	0.35
25	0.66	0.27	0.53	0.27	0.39	0.29
30	0.61	0.24	0.50	0.23	0.37	0.25
35	0.61	0.23	0.50	0.22	0.37	0.23
40	0.61	0.22	0.51	0.20	0.38	0.21
45	0.62	0.21	0.53	0.20	0.40	0.20
50	0.65	0.20	0.56	0.19	0.42	0.19
60	0.74	0.19	0.66	0.18	0.50	0.18
70	0.89	0.19	0.81	0.17	0.61	0.18
80	1.06	0.19	0.98	0.17	0.74	0.17
90	1.22	0.19	1.14	0.17	0.87	0.17
100	1.33	0.19	1.26	0.17	0.97	0.17

Table 2.5.2-235
Amplification Factors for the FIRS4_CoV50 Site Column

	Amplification Factor for 10^{-4}		Amplification Factor for 10^{-5}		Amplification Factor for 10^{-6}	
Freq (Hz)	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.	Median	Logarithmic Std. Dev.
0.1	1.10	0.05	1.10	0.05	1.11	0.05
0.125	1.13	0.07	1.13	0.07	1.14	0.07
0.15	1.18	0.10	1.18	0.10	1.19	0.10
0.2	1.28	0.15	1.29	0.15	1.30	0.15
0.3	1.45	0.18	1.46	0.18	1.48	0.19
0.4	1.44	0.17	1.45	0.17	1.48	0.20
0.5	1.38	0.18	1.39	0.19	1.44	0.24
0.6	1.36	0.16	1.39	0.18	1.45	0.25
0.7	1.40	0.15	1.44	0.19	1.51	0.26
0.8	1.47	0.14	1.52	0.21	1.60	0.27
0.9	1.52	0.15	1.57	0.23	1.66	0.29
1	1.57	0.17	1.60	0.23	1.70	0.30
1.25	1.86	0.22	1.96	0.29	2.04	0.33
1.5	2.07	0.29	2.15	0.32	2.20	0.31
2	2.06	0.29	2.11	0.31	2.11	0.29
2.5	1.76	0.37	1.73	0.35	1.73	0.33
3	1.54	0.45	1.49	0.38	1.48	0.38
4	1.43	0.49	1.38	0.46	1.30	0.47
5	1.57	0.53	1.44	0.50	1.19	0.43
6	1.57	0.47	1.36	0.44	1.11	0.47
7	1.43	0.45	1.25	0.47	1.01	0.50
8	1.30	0.43	1.12	0.44	0.93	0.51
9	1.23	0.41	1.06	0.45	0.87	0.52
10	1.15	0.40	1.00	0.43	0.80	0.50
12.5	1.01	0.34	0.86	0.36	0.68	0.43
15	0.91	0.32	0.77	0.35	0.60	0.41
20	0.73	0.30	0.60	0.31	0.45	0.36
25	0.64	0.28	0.51	0.29	0.39	0.36
30	0.62	0.27	0.50	0.28	0.37	0.32
35	0.60	0.24	0.49	0.24	0.37	0.27
40	0.60	0.23	0.50	0.22	0.37	0.24
45	0.61	0.22	0.52	0.21	0.38	0.21
50	0.64	0.21	0.55	0.20	0.41	0.20
60	0.73	0.21	0.64	0.19	0.48	0.19
70	0.87	0.20	0.79	0.19	0.59	0.19
80	1.04	0.20	0.95	0.18	0.72	0.18
90	1.19	0.20	1.11	0.18	0.84	0.18
100	1.30	0.20	1.23	0.18	0.94	0.18

Table 2.5.2-236
1E-5 and GMRS Amplitudes for GMRS Elevation, Horizontal and Vertical

Amplitudes for GMRS elevation			
Frequency (Hz)	Horizontal 1E-5 UHRS (g)	Hoizontal GMRS (g)	Vertical GMRS (g)
100	8.26E-02	3.72-E-02	3.72E-02
90	8.33E-02	3.75E-02	3.75E-02
80	8.42E-02	3.79E-02	3.79E-02
75	8.46E-02	3.81E-02	3.81E-02
70	8.51E-02	3.83E-02	3.83E-02
60	8.62E-02	3.88E-02	3.88E-02
50	8.76E-02	3.94E-02	3.94E-02
40	8.92E-02	4.01E-02	4.01E-02
30	9.14E-02	4.11E-02	4.11E-02
25	9.28E-02	4.18E-02	4.18E-02
20	9.74E-02	4.38E-02	4.38E-02
15	1.04-E01	4.66E-02	4.66E-02
12.5	1.08E-01	4.85E-02	4.85E-02
10	1.13E-01	5.09E-02	5.09E-02
9	1.14E-01	5.14E-02	5.14E-02
8	1.16E-01	5.20E-02	5.20E-02
7.5	1.16E-01	5.23E-02	5.23E-02
7	1.17E-01	5.27E-02	5.27E-02
6	1.19E-01	5.35E-02	5.34E-02
5	1.21E-01	5.45E-02	5.44E-02
4	1.42E-01	6.39E-02	6.38E-02
3	1.58E-01	7.13E-02	6.11E-02
2.5	1.62E-01	7.29E-02	5.21E-02
2	1.54E-01	6.94E-02	4.93E-02
1.8	1.50E-01	6.75E-02	4.78E-02
1.5	1.36E-01	6.14E-02	4.32E-02
1.25	1.20E-01	5.41E-02	3.79E-02
1	1.00E-01	4.50E-02	3.13E-02
0.9	9.65E-02	4.34E-02	3.01E-02
0.8	9.27E-02	4.17E-02	2.88E-02
0.7	8.85E-02	3.98E-02	2.74E-02
0.6	8.40E-02	3.78E-02	2.59E-02
0.5	7.89E-02	3.55E-02	2.42E-02
0.4	6.13E-02	2.76E-02	1.87E-02
0.3	4.19E-02	1.89E-02	1.27E-02
0.2	2.03E-02	9.12E-03	6.09E-03
0.15	1.14E-02	5.11E-03	3.42E-03
0.125	7.84E-03	3.53E-03	2.63E-03
0.1	4.95E-03	2.23E-03	1.49E-03

Table 2.5.2-237 (Sheet 1 of 2)
1E-5 and FIRS Amplitudes for FIRS Elevations, Horizontal and Vertical

	FIRS2 (g)			FIRS3 (g)		
Frequency (Hz)	1E-5 UHRS	Horizontal FIRS2	Vertical FIRS2	1E-5 UHRS	Horizontal FIRS 3	Vertical FIRS3
100	8.49E-02	3.82E-02	3.82E-02	1.01E-01	4.55E-02	4.55E-02
90	8.58E-02	3.86E-02	3.86E-02	1.03E-01	4.64E-02	4.64E-02
80	8.69E-02	3.91E-02	3.91E-02	1.05E-01	4.75E-02	4.75E-02
75	8.75E-02	3.94E-02	3.94E-02	1.07E-01	4.80E-02	4.80E-02
70	8.81E-02	3.96E-02	3.96E-02	1.08E-01	4.87E-02	4.87E-02
60	8.95E-02	4.03E-02	4.03E-02	1.11E-01	5.02E-02	5.02E-02
50	9.12E-02	4.10E-02	4.10E-02	1.15E-01	5.20E-02	5.20E-02
40	9.33E-02	4.20E-02	4.20E-02	1.21E-01	5.42E-02	5.42E-02
30	9.62E-02	4.33E-02	4.33E-02	1.27E-01	5.73E-02	5.73E-02
25	9.80E-02	4.41E-02	4.41E-02	1.32E-01	5.94E-02	5.94E-02
20	1.03E-01	4.63E-02	4.63E-02	1.55E-01	6.96E-02	6.96E-02
15	1.10E-01	4.94E-02	4.94E-02	1.93E-01	8.67E-02	8.67E-02
12.5	1.14E-01	5.14E-02	5.14E-02	2.02E-01	9.10E-02	9.10E-02
10	1.20E-01	5.40E-02	5.40E-02	2.08E-01	9.36E-02	9.36E-02
9	1.21E-01	5.43E-02	5.43E-02	1.99E-01	8.97E-02	8.97E-02
8	1.22E-01	5.47E-02	5.47E-02	1.89E-01	8.52E-02	8.52E-02
7.5	1.22E-01	5.49E-02	5.49E-02	1.84E-01	8.27E-02	8.27E-02
7	1.23E-01	5.51E-02	5.51E-02	1.78E-01	8.01E-02	8.00E-02
6	1.24E-01	5.56E-02	5.56E-02	1.65E-01	7.41E-02	7.41E-02
5	1.25E-01	5.63E-02	5.62E-02	1.49E-01	6.71E-02	6.70E-02
4	1.43E-01	6.44E-02	6.44E-02	1.66E-01	7.45E-02	7.44E-02
3	1.57E-01	7.06E-02	6.05E-02	1.77E-01	7.95E-02	6.81E-02
2.5	1.59E-01	7.16E-02	5.12E-02	1.77E-01	7.97E-02	5.69E-02
2	1.55E-01	6.97E-02	4.95E-02	1.73E-01	7.79E-02	5.53E-02
1.8	1.52E-01	6.84E-02	4.85E-02	1.70E-01	7.65E-02	5.42E-02
1.5	1.40E-01	6.30E-02	4.44E-02	1.57E-01	7.06E-02	4.97E-02
1.25	1.25E-01	5.62E-02	3.93E-02	1.40E-01	6.30E-02	4.41E-02
1	1.05E-01	4.73E-02	3.29E-02	1.18E-01	5.31E-02	3.70E-02
0.9	1.01E-01	4.56E-02	3.16E-02	1.15E-01	5.16E-02	3.58E-02
0.8	9.73E-02	4.38E-02	3.03E-02	1.11E-01	4.99E-02	3.45E-02
0.7	9.30E-02	4.19E-02	2.88E-02	1.07E-01	4.81E-02	3.31E-02
0.6	8.83E-02	3.97E-02	2.72E-02	1.02E-01	4.60E-02	3.16E-02
0.5	8.30E-02	3.74E-02	2.55E-02	9.72E-02	4.37E-02	2.98E-02
0.4	6.35E-02	2.86E-02	1.94E-02	7.34E-02	7.34E-02	2.24E-02
0.3	4.23E-02	1.90E-02	1.28E-02	4.96E-02	4.96E-02	1.50E-02
0.2	2.03E-02	9.13E-03	6.10E-03	2.45E-02	2.45E-02	7.37E-03
0.15	1.15E-02	5.17E-03	3.46E-03	1.39E-02	1.39E-02	4.19E-03
0.125	7.97E-03	3.58E-03	2.40E-03	9.62E-03	9.62E-03	2.89E-03
0.1	5.05E-03	2.27E-03	1.52E-03	6.09E-03	6.09E-03	1.83E-03

Table 2.5.2-237 (Sheet 2 of 2)
1E-5 and FIRS Amplitudes for FIRS Elevations, Horizontal and Vertical

	FIRS4 (g)			FIRS4-CoV50 (g)		
Frequency (Hz)	1E-5 UHRS	Horizontal FIRS4	Vertical FIRS4	1E-5 UHRS	Horizontal FIRS4-CoV50	Vertical FIRS4-CoV50
100	1.51E-01	6.80E-02	6.80E-02	1.48E-01	6.66E-02	6.66E-02
90	1.54E-01	6.93E-02	6.93E-02	1.51E-01	6.79E-02	6.79E-02
80	1.57E-01	7.07E-02	7.07E-02	1.54E-01	6.93E-02	6.93E-02
75	1.59E-01	7.16E-02	7.16E-02	1.56E-01	7.01E-02	7.01E-02
70	1.61E-01	7.25E-02	7.25E-02	1.58E-01	7.10E-02	7.10E-02
60	1.66E-01	7.45E-02	7.45E-02	1.62E-01	7.30E-02	7.30E-02
50	1.71E-01	7.70E-02	7.70E-02	1.67E-01	7.54E-02	7.54E-02
40	1.78E-01	8.02E-02	8.02E-02	1.74E-01	7.84E-02	7.84E-02
30	1.88E-01	8.45E-02	8.45E-02	1.83E-01	8.25E-02	8.25E-02
25	1.94E-01	8.73E-02	8.73E-02	1.90E-01	8.53E-02	8.53E-02
20	2.14E-01	9.61E-02	9.61E-02	2.13E-01	9.60E-02	9.60E-02
15	2.42E-01	1.09E-01	1.09E-01	2.49E-01	1.12E-01	1.12E-01
12.5	2.62E-01	1.18E-01	1.18E-01	2.74E-01	1.23E-01	1.23E-01
10	2.88E-01	1.30E-01	1.30E-01	3.08E-01	1.39E-01	1.39E-01
9	3.25E-01	1.46E-01	1.46E-01	3.30E-01	1.48E-01	1.48E-01
8	3.64E-01	1.64E-01	1.64E-01	3.52E-01	1.58E-01	1.58E-01
7.5	3.83E-01	1.73E-01	1.73E-01	3.62E-01	1.63E-01	1.63E-01
7	4.03E-01	1.81E-01	1.81E-01	3.73E-01	1.68E-01	1.68E-01
6	4.43E-01	1.99E-01	1.99E-01	3.94E-01	1.77E-01	1.77E-01
5	4.80E-01	2.16E-01	2.16E-01	4.12E-01	1.85E-01	1.85E-01
4	4.10E-01	1.84E-01	1.84E-01	3.83E-01	1.72E-01	1.72E-01
3	3.23E-01	1.45E-01	1.25E-01	3.40E-01	1.53E-01	1.31E-01
2.5	2.71E-01	1.22E-01	8.72E-02	3.08E-01	1.39E-01	9.91E-02
2	2.58E-01	1.16E-01	8.25E-02	2.86E-01	1.29E-01	9.14E-02
1.8	2.47E-01	1.11E-01	7.88E-02	2.71E-01	1.22E-01	8.64E-02
1.5	2.23E-01	1.01E-01	7.08E-02	2.41E-01	1.08E-01	7.64E-02
1.25	1.96E-01	8.81E-02	6.17E-02	2.08E-01	9.37E-02	6.57E-02
1	1.62E-01	7.29E-02	5.07E-02	1.70E-01	7.64E-02	5.32E-02
0.9	1.57E-01	7.07E-02	4.90E-02	1.64E-01	7.36E-02	5.11E-02
0.8	1.52E-01	6.82E-02	4.72E-02	1.57E-01	7.06E-02	4.88E-02
0.7	1.46E-01	6.56E-02	4.52E-02	1.50E-01	6.73E-02	4.64E-02
0.6	1.39E-01	6.27E-02	4.30E-02	1.42E-01	6.37E-02	4.37E-02
0.5	1.32E-01	5.94E-02	4.05E-02	1.33E-01	5.97E-02	4.07E-02
0.4	1.03E-01	4.62E-02	3.13E-02	1.03E-01	4.61E-02	3.13E-02
0.3	6.99E-02	3.15E-02	2.11E-02	6.95E-02	3.13E-02	2.10E-02
0.2	3.34E-02	1.50E-02	1.00E-02	3.31E-02	1.49E-02	9.94E-03
0.15	1.88E-02	8.47E-03	5.66E-03	1.86E-02	8.39E-03	5.61E-03
0.125	1.30E-02	5.85E-03	3.91E-03	1.29E-02	5.80E-03	3.88E-03
0.1	8.24E-03	3.71E-03	2.48E-03	8.17E-03	3.67E-03	2.46E-03

CP COL 2.5(1)

Table 2.5.4-201 (Sheet 1 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B1000	32°18'07.51921"N	97°47'41.80881"W	6793576	2186347	851.01	400.00
B1001	32°18'09.59563"N	97°47'42.07001"W	6793786	2186324	842.38	300.40
B1002	32°18'07.75819"N	97°47'40.05882"W	6793601	2186497	844.66	300.20
B1003	32°18'05.11572"N	97°47'41.12093"W	6793333	2186408	861.69	336.50
B1003 offset	32°18'05.11572"N[b]	97°47'41.12093"W[b]	6793333[b]	2186408[b]	861.77	400.5
B1004	32°18'07.31768"N	97°47'43.97424"W	6793554	2186162	859.70	301.50
B1005	32°18'07.55611"N	97°47'43.58808"W	6793579	2186195	857.02	225.00
B1006	32°18'08.79831"N	97°47'43.34929"W	6793579	2186195	847.72	225.00
B1006E	32°18'08.79831"N[b]	97°47'43.02538"W[b]	6793704[b]	2186242[b]	840.70	75.40
B1007	32°18'08.94015"N	97°47'42.02591"W	6793719	2186328	843.62	226.70
B1008	32°18'07.65947"N	97°47'41.05563"W	6793590	2186412	847.56	225.20
B1008 offset	32°18'07.65947"N[b]	97°47'41.05563"W[b]	6793590[b]	2186412[b]	847.54	8.70
B1009I	32°18’06.42029”N	97°47’41.58920”W	6793465	2186367	857.20	204.50
B1010	32°18’06.30251”N	97°47’42.63420”W	6793452	2186277	861.44	224.80
B1011I	32°18’07.38565”N	97°47’42.79987”W	6793562	2186262	854.71	204.60
B1011I offset	32°18’07.38565”N[b]	97°47’42.79987”W[b]	6793562[b]	2186262[b]	855.31	15.00
B1012	32°18’08.28312”N	97°47’35.91922”W	6793656	2186852	844.04	550.00

CP COL 2.5(1)

Table 2.5.4-201 (Sheet 2 of 10) Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B1012 offset	32°18’08.28312”N[b]	97°47’35.91922”W[b]	6793656[b]	2186852[b]	844.04	5.50
B1013	32°18'05.63875"N	97°47'42.83107"W	6793385	2186261	863.53	150.00
B1014	32°18'08.08134"N	97°47'42.94056"W	6793632	2186250	851.20	150.00
B1015	32°18'05.88650"N	97°47'40.88214"W	6793411	2186428	858.09	151.40
B1015 offset	32°18'05.88650"N[b]	97°47'40.88214"W[b]	6793411[b]	2186428[b]	858.80	15.50
B1016	32°18’04.81020”N	97°47’42.69342”W	6793302	2186273	865.59	201.20
B1017	32°18’03.31719”N	97°47’42.04443”W	6793151	2186330	866.37	150.00
B1018	32°18’09.21359”N	97°47’43.53870”W	6793746	2186198	846.36	152.40
B1019	32°18’08.45527”N	97°47’43.81029”W	6793669	2186175	852.22	200.00
B1020	32°18’06.71293”N	97°47’44.33772”W	6793493	2186131	861.94	152.40
B1021	32°18’07.84854”N	97°47’39.21954”W	6793611	2186569	845.18	150.00
B1022	32°18’06.25714”N	97°47’43.42690”W	6793447	2186209	863.48	150.00
B1023	32°18’05.48582”N	97°47’44.14647”W	6793369	2186148	865.68	150.50
B1024	32°18’05.69714”N	97°47’41.93881”W	6793392	2186337	861.65	110.00
B1025	32°18’04.93101”N	97°47’40.85351”W	6793315	2186431	861.52	151.00

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Table 2.5.4-201 (Sheet 3 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B1025 offset	32°18’04.93101”N[b]	97°47’40.85351”W[b]	6793315[b]	2186431[b]	861.69	16.00
B1026	32°18’04.25347”N	97°47’41.61808”W	6793247	2186365	865.13	110.80
B1027	32°18’03.43182”N	97°47’40.55214”W	6793164	2186458	863.62	150.00
B1028	32°18’09.96343”N	97°47’42.20648”W	6793823	2186312	842.06	75.50
B1029	32°18’08.25662”N	97°47’41.89926”W	6793650	2186339	846.88	150.00
B1030	32°18’07.74588”N	97°47’44.59729”W	6793597	2186108	858.58	150.00
B1031	32°18’06.22031”N	97°47’44.51535”W	6793443	2186116	864.21	100.60
B1032	32°18’04.92264”N	97°47’43.54930”W	6793312	2186200	867.32	100.50
B1033	32°18’04.82882”N	97°47’44.29315”W	6793303	2186136	866.92	76.20
B1034	32°18’02.70848”N	97°47’41.75126”W	6793090	2186356	858.46	150.00
B1035	32°18’02.78195”N	97°47’39.80404”W	6793098	2186523	856.11	100.00
B1036	32°18’04.75500”N	97°47’40.08035”W	6793298	2186498	858.65	75.00
B1037	32°18’06.16864”N	97°47’40.27994”W	6793440	2186479	852.84	151.30
B1038	32°18’09.15105”N	97°47’40.86652”W	6793741	2186427	843.53	150.00
B1039	32°18’06.47144”N	97°47’39.33504”W	6793471	2186560	847.10	75.00
B1040	32°18’06.86973”N	97°47’39.74260”W	6793511	2186525	847.04	100.50
B1041	32°18’07.28642”N	97°47’39.13745”W	6793554	2186577	845.30	149.80
B1042	32°18’06.28423”N	97°47’38.94404”W	6793453	2186594	846.41	151.00

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Table 2.5.4-201 (Sheet 4 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B1043	32°18'10.97505"N	97°47'46.05367"W	6793923	2185981	824.51	76.80
B1044	32°18'11.26805"N	97°47'43.12536"W	6793954	2186232	838.21	75.70
B1045	32°18'11.45493"N	97°47'42.45811"W	6793973	2186289	838.17	75.80
B1046	32°18'11.75242"N	97°47'39.61640"W	6794005	2186533	837.25	75.00
B1047	32°18'10.45134"N	97°47'45.24071"W	6793870	2186051	831.11	150.00
B1048	32°18'10.67112"N	97°47'43.85016"W	6793893	2186170	835.78	101.50
B1049	32°18'10.88198"N	97°47'41.69399"W	6793916	2186355	840.41	100.00
B1050	32°18'11.06395"N	97°47'40.26851"W	6793935	2186477	838.50	150.00
B1051	32°18'09.66696"N	97°47'45.88176"W	6793791	2185996	842.69	75.00
B1052	32°18'10.11028"N	97°47'42.99168"W	6793837	2186244	842.51	75.00
B1053	32°18'10.17969"N	97°47'42.28628"W	6793844	2186305	841.35	75.80
B1054	32°18'10.53981"N	97°47'39.42754"W	6793882	2186550	839.66	75.50
B1055	32°18'08.18710"N	97°47'45.48654"W	6793641	2186031	854.80	50.00
B1056	32°18'08.94327"N	97°47'39.29146"W	6793721	2186562	843.03	51.00
B1057	32°18'06.72009"N	97°47'45.31534"W	6793493	2186047	861.14	50.00
B1058	32°18'07.47110"N	97°47'39.04390"W	6793573	2186585	845.22	51.00
B1059	32°18'04.98000"N	97°47'44.55224"W	6793318	2186114	866.15	50.00
B1060	32°18'05.06516"N	97°47'43.76893"W	6793327	2186181	866.96	50.00

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Table 2.5.4-201 (Sheet 5 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B1061	32°18’05.59134”N	97°47’39.50493”W	6793382	2186546	852.77	51.00
B1062	32°18’05.67223”N	97°47’38.75965”W	6793391	2186610	848.38	50.80
B1063	32°18’12.57354”N	97°47’36.94881”W	6794089	2186761	837.16	100.00
B1064	32°18’11.43676”N	97°47’36.77404”W	6793975	2186777	839.05	100.50
B1065	--[b]	--[b]	6793654[b]	2186082[b]	--[b]	145.80
B2000	32°18’08.98718”N	97°47’30.07842”W	6793731	2187353	844.03	401.10
B2001	32°18’11.01368”N	97°°47’30.36614”W	6793936	2187327	837.89	301.00
B2001 offset	32°18’11.01368”N[b]	97°47’30.36614”W[b]	6793936[b]	2187327[b]	837.71	9.60
B2002	32°18'09.20743"N	97°47'28.37129"W	6793754	2187499	837.21	302.00
B2003	32°18'06.70815"N	97°47'29.67923"W	6793501	2187389	840.99	400.30
B2003 offset	32°18'06.70815"N[b]	97°47'29.67923"W[b]	6793501[b]	2187389[b]	841.14	253.00
B2004	32°18'08.74045"N	97°47'32.27067"W	6793705	2187165	850.53	300.00
B2005	32°18'08.98107"N	97°47'31.88572"W	6793729	2187198	848.72	226.00
B2006	32°18'10.22084"N	97°47'31.29884"W	6793855	2187248	844.99	226.00
B2007	32°18'10.39277"N	97°47'30.27188"W	6793873	2187336	841.23	225.00
B2008	32°18'09.21386"N	97°47'29.14203"W	6793754	2187433	840.62	225.00

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Table 2.5.4-201 (Sheet 6 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B2008 offset	32°18'09.21386"N[b]	97°47'29.14203"W[b]	6793754[b]	2187433[b]	840.92	13.00
B2008E	32°18'09.21386"N[b]	97°47'29.14203"W[b]	--[b]	--[b]	817.5	67.00
B2009I	32°18'07.84148"N	97°47'29.82984"W	6793615	2187375	842.97	204.60
B2009I offset	32°18'07.84148"N[b]	97°47'29.82984"W[b]	6793615[b]	2187375[b]	843.18	10.50
B2010	32°18’07.73186”N	97°47’30.85096”W	6793604	2187288	846.41	226.40
B2011I	32°18’08.80851”N	97°47’31.14886”W	6793712	2187261	847.08	204.50
B2011I offset	32°18’08.80851”N[b]	97°47’31.14886”W[b]	6793712[b]	2187261[b]	846.73	8.50
B2012	32°18’07.04791”N	97°47’31.13925”W	6793534	2187263	847.19	150.00
B2013	32°18’09.54492”N	97°47’31.24423”W	6793787	2187253	846.96	150.50
B2014	32°18’07.30404”N	97°47’29.17122”W	6793561	2187432	839.41	150.00
B2015	32°18’06.22925”N	97°47’31.00136”W	6793452	2187276	846.68	200.00
B2016	32°18’04.64128”N	97°47’30.28038”W	6793292	2187339	841.04	150.20
B2016 offset	32°18’04.64128”N[b]	97°47’30.28038”W[b]	6793292[b]	2187339[b]	841.04[b]	8.50
B2017	32°18’10.69107”N	97°47’31.88619”W	6793902	2187197	845.37	150.00
B2018	32°18’09.87162”N	97°47’32.12241”W	6793819	2187177	848.690	8.10

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Table 2.5.4-201 (Sheet 7 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B2018 offset	32°18’09.87162”N[b]	97°47’32.12241”W[b]	6793819[b]	2187177[b]	848.69	200.00
B2019	32°18’08.11069”N	97°47’32.64972”W	6793641	2187133	852.57	150.70
B2020	32°18’09.27433”N	97°47’27.42284”W	6793762	2187581	832.25	150.00
B2021	32°18’07.65632”N	97°47’31.73443”W	6793596	2187212	849.56	150.00
B2022	32°18’06.89117”N	97°47’32.44375”W	6793518	2187152	851.98	150.00
B2023	32°18’07.14539”N	97°47’30.13802”W	6793545	2187349	843.55	110.00
B2024	32°18’06.24608”N	97°47’29.41622”W	6793454	2187412	838.15	150.30
B2025	32°18’05.73697”N	97°47’29.93430”W	6793403	2187368	840.13	110.00
B2026	32°18’04.80417”N	97°47’28.85847”W	6793309	2187461	833.49	149.00
B2027	32°18’08.26494”N	97°47’26.33985”W	6793660	2187674	827.00	75.00
B2028	32°18’06.56476”N	97°47’26.04534”W	6793489	2187701	823.74	75.50
B2029	32°18’09.69373”N	97°47’30.20691”W	6793802	2187342	843.58	151.00
B2030	32°18’09.28154”N	97°47’33.33612”W	6793759	2187073	852.55	151.40
B2030 offset	32°18’09.28154”N[b]	97°47’33.33612”W[b]	6793759[b]	2187073[b]	852.55[b]	8.50
B2031	32°18’07.65554”N	97°47’32.80039”W	6793595	2187120	853.14	100.00
B2032	32°18’06.18999”N	97°47’32.54177”W	6793447	2187144	852.99	150.00
B2033	32°18’06.27388”N	97°47’31.85974”W	6793456	2187202	850.69	76.00
B2034	32°18’03.96063”N	97°47’30.02668”W	6793223	2187361	839.12	150.90

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Table 2.5.4-201 (Sheet 8 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B2035	32°18’04.21968”N	97°47’28.00470”W	6793250	2187534	829.82	100.20
B2036	32°18’06.15506”N	97°47’28.34813”W	6793446	2187504	832.35	75.00
B2037	32°18’07.56081”N	97°47’28.59029”W	6793588	2187482	836.49	150.60
B2038	32°18’10.57050”N	97°47’29.13727”W	6793892	2187433	836.95	151.00
B2038 offset	32°18’10.57050”N[b]	97°47’29.13727”W[b]	6793892[b]	2187433[b]	839.95	9.00
B2039	32°18’07.71152”N	97°47’27.86599”W	6793603	2187544	833.49	76.00
B2040	32°18’08.34422”N	97°47’28.05260”W	6793667	2187527	835.14	100.50
B2041	32°18’08.70943”N	97°47’27.42697”W	6793704	2187581	831.76	151.40
B2041A	32°18’08.70943”N[b]	97°47’27.42697”W[b]	6793704[b]	2187581[b]	832.05	150.00
B2042	32°18’07.71444”N	97°47’27.25919”W	6793604	2187596	830.75	90.00
B2042A	32°18’07.71444”N[b]	97°47’27.25919”W[b]	6793604[b]	2187596[b]	830.66	150.00
B2043	32°18’07.04321”N	97°47’32.35975”W	6793533	2187159	851.71	55.00
B2044	32°18’12.39045”N	97°47’34.36032”W	6794072	2186983	843.46	75.20
B2044 offset	32°18’12.39045”N[b]	97°47’34.36032”W[b]	6794072[b]	2186983[b]	843.71	13.50
B2045	32°18’12.73530”N	97°47’31.51690”W	6794109	2187227	832.33	75.00
B2046	32°18’12.84479”N	97°47’30.81327”W	6794120	2187287	828.07	74.50

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**Table 2.5.4-201 (Sheet 9 of 10)
Summary of Exploratory Borings**

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B2047	32°18’13.24168”N	97°47’27.96754”W	6794162	2187531	793.70	75.00
B2048	32°18’11.90693”N	97°47’33.58070”W	6794024	2187051	844.18	151.70
B2048 offset	32°18’11.90693”N[b]	97°47’33.58070”W[b]	6794024[b]	2187051[b]	845.92	16.50
B2049	32°18’12.05915”N	97°47’32.15036”W	6794040	2187173	839.66	101.00
B2050	32°18’12.19402”N	97°47’30.01212”W	6794055	2187357	827.85	100.50
B2051	32°18’12.48880”N	97°47’28.53218”W	6794086	2187483	809.97	100.40
B2052	32°18’11.17840”N	97°47’34.17224”W	6793950	2187000	846.57	75.00
B2053	32°18’11.43072”N	97°47’31.29465”W	6793977	2187247	839.10	75.00
B2053 offset	32°18’11.43072”N[b]	97°47’31.29465”W[b]	6793977[b]	2187247[b]	839.10[b]	9.10
B2054	32°18'11.62191"N	97°47'30.60538"W	6793997	2187306	834.54	75.00
B2055	32°18'11.96274"N	97°47'27.70218"W	6794033	2187555	815.25	75.00
B2056	32°18'09.61661"N	97°47'33.77532"W	6793792	2187035	852.34	50.60
B2057	32°18'10.36396"N	97°47'27.57830"W	6793872	2187567	831.71	50.00
B2058	32°18'08.15106"N	97°47'33.52367"W	6793644	2187058	853.67	50.00
B2059	32°18'08.89353"N	97°47'27.33431"W	6793723	2187589	831.73	50.00
B2060	32°18'06.40808"N	97°47'32.82242"W	6793469	2187119	853.72	50.00
B2061	32°18'06.48996"N	97°47'32.06266"W	6793477	2187185	851.24	50.00
B2062	32°18'07.00447"N	97°47'27.79434"W	6793532	2187551	831.34	51.00

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Table 2.5.4-201 (Sheet 10 of 10)
Summary of Exploratory Borings

Boring No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
B2063	32°18'07.10306"N	97°47'27.04391"W	6793542	2187615	828.15	50.10
B2064	-[b]	-[b]	6793805[b]	2187085[b]	-[b]	149.00
BS1000-ABC	32°18'07.18723"N	97°47'41.55090"W	6793542	2186370	851.96	40.00
BS1001-ABC	32°18'04.92967"N	97°47'41.09384"W	6793315	2186410	861.81	39.90
BS2000-ABC	32°18'08.29646"N	97°47'31.04370"W	6793661	2187271	847.00	40.60
BS2001-ABC	32°18'06.78874"N	97°47'28.45203"W	6793510	2187494	834.39	40.20

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

b) Coordinates are approximate. Values are taken same as or estimated from the primary adjacent boring or approximated based on the data from initial field location survey.

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Table 2.5.4-202
Summary of Test Pits

Test Pit No.	East and West Coordinates			Orientation	Depth (ft)	Dimensions (ft)
	Northing	Easting	Elevation (ft)			
Test Pit A	6,793,760	2,187,762	822.48	N86W	5	55 x 26
	6,793,773	2,187,695	825.83			
Test Pit B	6,793,618	2,187,691	826.65	N71W	9.5	41 x 9
	6,793,631	2,187,646	828.55			
Test Pit C	6,793,417	2,187,625	826.14	N75W	12	42.4 x 4
	6,793,427	2,187,585	828.14			

Notes: Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

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Table 2.5.4-203 (Sheet 1 of 2) Summary of Monitoring Wells

Monitoring Well No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
MW-1200S	32°18'18.85108"N	97°47'49.77300"W	6,794,716	2,185,656	848.97	100.00
MW-1201S	32°18'06.56453"N	97°47'44.74781"W	6,793,478	2,186,096	863.08	100.00
MW-1202S	32°18'03.79866"N	97°47'51.54684"W	6,793,194	2,185,514	853.86	100.20
MW-1203S	32°17'53.39997"N	97°47'47.88713"W	6,792,146	2,185,835	862.42	100.00
MW-1204S-A	32°17'48.78667"N	97°47'31.04428"W	6,791,689	2,187,284	842.18	100.00
MW-1205S	32°18'04.59247"N	97°47'39.42955"W	6,793,281	2,186,554	857.45	100.50
MW-1206S	32°18'14.35354"N	97°47'33.20609"W	6,794,271	2,187,081	833.08	100.20
MW-1207S	32°18'10.55139"N	97°47'34.23116"W	6,793,887	2,186,996	848.57	99.70
MW-1208S	32°18'09.60935"N	97°47'24.06571"W	6,793,797	2,187,869	817.43	100.20
MW-1209S	32°18'11.69119"N	97°47'23.33107"W	6,794,008	2,187,930	808.45	100.10
MW-1210S	32°18'02.34306"N	97°47'22.19926"W	6,793,064	2,188,034	827.92	100.10
MW-1211S	32°18'08.22628"N	97°47'20.88260"W	6,793,659	2,188,143	810.57	100.00
MW-1212S	32°18'04.91836"N	97°47'19.34452"W	6,793,326	2,188,277	819.93	100.50
MW-1213S	32°18'03.33999"N	97°47'31.58713"W	6,793,159	2,187,227	845.55	101.62
MW-1214S	32°18'05.17066"N	97°47'25.09686"W	6,793,348	2,187,783	821.36	101.10

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Table 2.5.4-203 (Sheet 2 of 2) Summary of Monitoring Wells

Monitoring Well No.	Latitude	Longitude	Northing[a]	Easting[a]	Elevation[a] (ft)	Total Depth (ft)
MW-1215S	32°18'08.03859"N	97°47'41.71230"W	6,793,628	2,186,355	847.77	99.80
MW-1216S	32°18'06.35503"N	97°47'30.32786"W	6,793,465	2,187,334	844.04	101.50
MW-1217S	32°18'09.82078"N	97°47'30.49836"W	6,793,815	2,187,317	844.30	101.40
MW-1218S	32°17'45.56710"N	97°47'41.14517"W	6,791,358	2,186,419	835.48	15.00
MW-1219S	32°18'12.86174"N	97°47'38.36713"W	6,794,118	2,186,639	836.35	54.20

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

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Table 2.5.4-204 (Sheet 1 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[a] (ft)	Total Depth (ft)
C1000	32°18'07.46915"N	97°47'41.80421"W	6793571	2186348	850.7	4.46
C1001	32°18'07.81814"N	97°47'39.45497"W	6793607	2186549	845.0	4.46
C1002	32°18'07.13567"N	97°47'44.94923"W	6793535	2186078	861.0	2.16
C1002A	32°18'07.19810"N	97°47'44.90419"W	6793542	2186082	860.9	3.02
C1002B	32°18'07.28156"N	97°47'44.66234"W	6793550	2186103	861.0	2.69
C1002C	32°18'07.16413"N	97°47'45.03758"W	6793538	2186070	860.7	2.36
C1002D	32°18'07.39319"N	97°47'44.87384"W	6793561	2186084	860.3	1.71
C1003	32°18'05.62311"N	97°47'43.03034"W	6793384	2186244	864.0	3.74
C1003A	32°18'05.69469"N	97°47'43.06085"W	6793391	2186241	863.7	3.94
C1003B	32°18'05.79108"N	97°47'43.02665"W	6793401	2186244	863.0	3.87
C1004	32°18'03.97817"N	97°47'43.99060"W	6793217	2186162	869.2	0.46
C1004A	32°18'03.92245"N	97°47'43.97221"W	6793211	2186164	869.0	3.02
C1004B	32°18'03.82792"N	97°47'43.94327"W	6793202	2186167	868.6	0.85
C1005	32°18'04.3128"N[b]	97°47'41.2548"W[b]	--[b]	--[b]	864.0	4.26
C1005A	32°18'04.3128"N[b]	97°47'41.2548"W[b]	--[b]	--[b]	864.0	0.85
C1005B	32°18'04.3128"N[b]	97°47'41.2548"W[b]	--[b]	--[b]	864.0	3.48

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Table 2.5.4-204 (Sheet 2 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[(a)] (ft)	Total Depth (ft)
C1006	32°18'04.45836"N	97°47'39.34929"W	6793268	2186561	856.7	4.53
C1006A	32°18'04.44597"N	97°47'39.43033"W	6793267	2186554	857.2	4.59
C1006B	32°18'04.42505"N	97°47'39.57432"W	6793264	2186541	857.7	4.72
C1007	32°18'09.37358"N	97°47'41.62853"W	6793763	2186362	842.6	12.07
C1008	32°18'09.55938"N	97°47'42.63558"W	6793782	2186275	843.0	0.92
C1008A	32°18'09.51316"N	97°47'42.67642"W	6793777	2186272	843.6	0.52
C1008B	32°18'09.40089"N	97°47'42.77354"W	6793765	2186263	844.1	1.57
C1008C	32°18'09.83801"N	97°47'42.26216"W	6793810	2186307	842.1	11.55
C1009	32°18'01.58286"N	97°47'41.91899"W	6792976	2186342	868.1	6.10
C1010N	32°18'03.36480"N	97°47'39.77309"W	6793157	2186525	860.8	7.74
C1010NA	32°18'03.37766"N	97°47'39.85330"W	6793158	2186518	861.1	2.36
C1010NB	32°18'03.39935"N	97°47'39.95695"W	6793161	2186509	861.6	2.36
C1010S	32°18'01.72466"N	97°47'39.92874"W	6792991	2186513	866.7	6.04
C1011	32°18'07.67466"N	97°47'40.56916"W	6793592	2186454	845.9	1.44
C1011A	32°18'07.65212"N	97°47'40.65110"W	6793590	2186447	846.2	1.44
C2000	32°18'09.03191"N	97°47'30.06497"W	6793736	2187354	844.0	2.03
C2000A	32°18'09.02956"N	97°47'30.02838"W	6793735	2187357	843.8	1.77

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Table 2.5.4-204 (Sheet 3 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[a] (ft)	Total Depth (ft)
C2000B	32°18'09.02395"N	97°47'29.93741"W	6793735	2187365	843.4	1.64
C2001	32°18'09.08754"N	97°47'28.85642"W	6793742	2187458	839.2	1.12
C2001A	32°18'09.16386"N	97°47'28.85478"W	6793750	2187458	839.2	1.18
C2002	32°18'08.56861"N	97°47'33.23249"W	6793687	2187083	853.2	3.61
C2002A	32°18'08.57208"N	97°47'33.28851"W	6793687	2187078	853.4	2.03
C2002B	32°18'08.56274"N	97°47'33.04688"W	6793686	2187099	853.0	2.36
C2003	32°18'07.02715"N	97°47'31.32887"W	6793532	2187247	847.7	4.26
C2003A	32°18'06.98111"N	97°47'31.27617"W	6793528	2187252	847.4	4.20
C2003B	32°18'06.92189"N	97°47'31.20619"W	6793522	2187258	847.2	3.87
C2004	32°18'05.47837"N	97°47'31.43934"W	6793376	2187239	847.9	3.94
C2004A	32°18'05.48708"N	97°47'31.49789"W	6793376	2187234	848.3	4.33
C2004B	32°18'05.50223"N	97°47'31.60136"W	6793378	2187225	848.8	5.12
C2005	32°18'05.70195"N	97°47'29.54029"W	6793399	2187402	837.4	8.20
C2005A	32°18'05.68533"N	97°47'29.50530"W	6793398	2187405	837.2	5.58
C2005B	32°18'05.65596"N	97°47'29.44588"W	6793395	2187410	836.8	4.79
C2006	32°18'05.93090"N	97°47'27.64577"W	6793424	2187564	828.7	14.10
C2006A	32°18'05.83897"N	97°47'27.51879"W	6793414	2187575	828.1	13.58

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Table 2.5.4-204 (Sheet 4 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[a] (ft)	Total Depth (ft)
C2007	32°18'10.79568"N	97°47'29.78233"W	6793914	2187377	837.4	8.33
C2007A	32°18'10.80112"N	97°47'29.82874"W	6793914	2187373	837.4	8.40
C2007B	32°18'10.92151"N	97°47'29.71231"W	6793927	2187383	835.9	7.08
C2008	32°18'03.62261"N	97°47'29.96209"W	6793189	2187367	838.2	8.00
C2008A	32°18'03.60531"N	97°47'29.94363"W	6793187	2187368	838.0	10.69
C2009	32°18'03.89761"N	97°47'27.84007"W	6793218	2187549	829.4	2.82
C2009A	32°18'03.93021"N	97°47'27.81519"W	6793221	2187551	829.2	6.23
C2009B	32°18'03.99134"N	97°47'27.77162"W	6793227	2187555	828.8	3.15
C3000	32°18'10.87947"N	97°47'39.51747"W	6793917	2186542	839.1	21.45
C3001	32°18'12.78812"N	97°47'36.43490"W	6794111	2186805	836.9	5.18
C3001A	32°18'12.82762"N	97°47'36.48193"W	6794115	2186801	836.8	3.80
C3001B	32°18'12.74652"N	97°47'36.56386"W	6794107	2186794	836.8	5.25
C3001C	32°18'12.76515"N	97°47'36.40872"W	6794109	2186807	836.9	5.44
C3002	32°18'03.69725"N	97°47'46.45279"W	6793187	2185951	865.2	3.02
C3002A	32°18'03.53639"N	97°47'46.47550"W	6793171	2185950	865.8	3.15
C3002B	32°18'03.45046"N	97°47'46.48356"W	6793162	2185949	866.4	2.56
C3003	32°18'04.8168"N[b]	97°47'37.0392"W[b]	--[b]	--[b]	851.5	4.99

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Table 2.5.4-204 (Sheet 5 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[(a)] (ft)	Total Depth (ft)
C3003A	32°18'04.8168"N[(b)]	97°47'37.0392"W[(b)]	--[(b)]	--[(b)]	851.5	5.12
C3003B	32°18'04.8168"N[(b)]	97°47'37.0392"W[(b)]	--[(b)]	--[(b)]	851.5	5.38
C3004	32°18'05.16097"N	97°47'34.15022"W	6793342	2187006	856.6	5.12
C3004A	32°18'05.11118"N	97°47'34.05133"W	6793337	2187015	856.4	3.87
C3004B	32°18'05.04304"N	97°47'33.91382"W	6793330	2187027	855.9	4.53
C3005	32°18'04.1004"N[(b)]	97°47'26.9808"W[(b)]	--[(b)]	--[(b)]	827.0	12.92
C3006	32°18'06.19656"N	97°47'25.60251"W	6793452	2187739	820.8	14.69
C3006A	32°18'06.17056"N	97°47'25.49724"W	6793449	2187748	820.5	21.45
C3007	32°18'06.42765"N	97°47'23.62223"W	6793476	2187909	817.4	5.05
C3007A	32°18'06.42974"N	97°47'23.56749"W	6793476	2187914	817.2	23.03
C3007B	32°18'06.44050"N	97°47'23.34877"W	6793478	2187932	816.8	36.80
C3008	32°18'09.54543"N	97°47'25.26984"W	6793790	2187765	821.9	3.67
C3008A	32°18'09.57800"N	97°47'25.20881"W	6793793	2187771	821.7	8.99
C3008B	32°18'09.59475"N	97°47'25.17571"W	6793795	2187773	821.6	8.99
C3008C	32°18'09.66918"N	97°47'25.10302"W	6793803	2187780	821.2	8.53
C3008D	32°18'09.68595"N	97°47'24.95674"W	6793805	2187792	820.6	12.33
C3009	32°18'02.28463"N	97°47'26.54828"W	6793056	2187661	826.7	14.37

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Table 2.5.4-204 (Sheet 6 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[(a)] (ft)	Total Depth (ft)
C3009A	32°18'02.40512"N	97°47'26.49725"W	6793068	2187665	826.3	13.05
C3010	32°18'03.87799"N	97°47'24.61092"W	6793218	2187826	822.4	10.69
C3010A	32°18'03.96058"N	97°47'24.69594"W	6793226	2187819	822.1	30.04
C3011	32°18'05.71834"N	97°47'22.59034"W	6793405	2187998	815.1	16.73
C3011A	32°18'05.77744"N	97°47'22.71005"W	6793411	2187988	815.4	24.34
C3012	32°18'07.64735"N	97°47'20.42722"W	6793601	2188182	808.9	26.31
C3012A	32°18'07.64735"N	97°47'20.42722"W	6793601	2188182	808.9	29.72
C3013	32°18'03.29316"N	97°47'23.70785"W	6793159	2187904	825.3	1.12
C3013A	32°18'03.19417"N	97°47'23.96737"W	6793149	2187882	825.4	3.02
C3013B	32°18'03.23664"N	97°47'23.84093"W	6793153	2187892	825.5	23.68
C3014	32°18'07.22787"N	97°47'29.62913"W	6793553	2187393	841.3	3.35
C3014A	32°18'07.31630"N	97°47'29.58516"W	6793562	2187397	841.2	3.21
C3015	32°18'03.74591"N	97°47'19.51591"W	6793207	2188263	822.6	1.64
C3015A	32°18'03.83164"N	97°47'19.57168"W	6793216	2188258	822.5	2.43
C3015B	32°18'04.08274"N	97°47'19.50723"W	6793241	2188264	821.7	3.08
C3016	32°18'07.73279"N	97°47'25.56746"W	6793607	2187741	823.7	25.06
C3017	32°18'09.41571"N	97°47'23.09021"W	6793778	2187953	814.0	2.69

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Table 2.5.4-204 (Sheet 7 of 7)
Summary of Cone Penetration Test Soundings

CPT No.	Latitude	Longitude	Northing	Easting	Elevation[a] (ft)	Total Depth (ft)
C3017A	32°18'09.39781"N	97°47'23.03822"W	6793777	2187957	814.0	30.83
C3018	32°18'06.53302"N	97°47'22.50891"W	6793487	2188004	815.0	36.8

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

b) Coordinates are approximate. Values are estimated from the primary adjacent sounding or approximated based on the data from initial field location survey.

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Table 2.5.4-205 (Sheet 1 of 2)
Summary of Seismic Refraction Survey Lines

Line No.	No. of Spreads	Geophone Spacing (ft)	Shots Per Spread	Sum of Individual Spread Lengths (ft)	Total Line Length (ft)	Survey Line Starting Coordinates			Survey Line Ending Coordinates		
						Easting	Northing	Elevation (ft)	Easting	Northing	Elevation (ft)
1	2	15	7 & 7	630	630	2,185,958	6,793,158	866.7	2,186,572	6,793,297	853.3
2	1	15	7	345	345	2,186,113	6,793,541	861.0	2,186,453	6,793,592	846.1
3	1	10	7	230	230	2,186,414	6,793,119	864.6	2,186,427	6,793,348	860.2
4	1	15	6	255	255	2,186,568	6,793,866	839.9	2,186,504	6,794,113	834.8
5	1	12	6	252	252	2,186,827	6,794,050	838.4	2,186,748	6,794,286	833.0
6	3	15	7, 7, & 6	930	840	2,187,447	6,793,094	831.0	2,187,323	6,793,922	839.4
7	2	15	7 & 7	690	690	2,187,712	6,793,096	825.5	2,188,182	6,793,601	808.8
8	2	15	7 & 7	690	675	2,187,903	6,793,159	825.5	2,187,369	6,793,572	842.1
9	2	15	7 & 7	690	645	2,188,265	6,793,200	822.9	2,187,755	6,793,596	823.7
10	1	15	8	345	345	2,188,213	6,793,577	809.3	2,187,939	6,793,787	814.3
10-S1	1	15	5	345	345	2,188,213	6,793,577	809.3	2,187,939	6,793,787	814.3
10-S2	1	15	4	345	345	2,188,213	6,793,577	809.3	2,187,939	6,793,788	814.3
11	1	15	7	285	285	2,186,535	6,793,260	858.3	2,186,819	6,793,272	853.4
12	3	15	7, 7, & 7	1080	1035	2,186,997	6,793,341	856.7	2,188,021	6,793,490	814.5
13	1	10	7	230	230	2,186,902	6,793,611	846.8	2,186,675	6,793,643	843.3

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Table 2.5.4-205 (Sheet 2 of 2)
Summary of Seismic Refraction Survey Lines

Line No.	No. of Spreads	Geophone Spacing (ft)	Shots Per Spread	Sum of Individual Spread Lengths (ft)	Total Line Length (ft)	Survey Line Starting Coordinates			Survey Line Ending Coordinates		
						Easting	Northing	Elevation (ft)	Easting	Northing	Elevation (ft)
14	3	15	7, 7, & 7	945	855	2,187,050	6,793,681	852.9	2,187,895	6,793,808	815.5
15	1	15	7	345	345	2,186,366	6,793,433	859.6	2,186,314	6,793,773	843.1

Notes: Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

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Table 2.5.4-206 (Sheet 1 of 3)
Summary of In Situ Packer Tests

Boring No.	Test No.	Test Mid-Interval Depth (ft)	Test Mid-Interval Elevation (ft)	Borehole Size	Test Interval Length (ft)	Average Permeability[a] (cm/s)	Average Permeability[b] (cm/s)
B-1016	1	152	719.43	HQ	5	3.95×10^{-08}	1.40×10^{-08}
B-1016	2	135	736.43	HQ	5	1.37×10^{-08}	5.48×10^{-09}
B-1016	3	110	761.43	HQ	5	1.06×10^{-08}	4.98×10^{-09}
B-1016	4	91	780.43	HQ	5	9.47×10^{-09}	4.83×10^{-09}
B-1016	5	69	802.43	HQ	5	4.03×10^{-08}	2.48×10^{-08}
B-1016	6	45	826.43	HQ	5	0.00	0.00
B-1016	7	30	841.43	HQ	5	0.00	0.00
B-1019	1	175	677.71	HQ	5	4.75×10^{-09}	1.30×10^{-09}
B-1019	2	133	719.71	HQ	5	4.56×10^{-08}	1.89×10^{-08}
B-1019	3	110	742.71	HQ	5	7.00×10^{-09}	3.30×10^{-09}
B-1019	4	95	757.71	HQ	5	1.00×10^{-08}	4.72×10^{-09}
B-1019	5	67	785.71	HQ	5	2.51×10^{-08}	1.51×10^{-08}
B-1019	6	57	795.71	HQ	5	0.00	0.00
B-1019	7	27	825.71	HQ	5	0.00	0.00

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Table 2.5.4-206 (Sheet 2 of 3)
Summary of In Situ Packer Tests

Boring No.	Test No.	Test Mid-Interval Depth (ft)	Test Mid-Interval Elevation (ft)	Borehole Size	Test Interval Length (ft)	Average Permeability[(a)] (cm/s)	Average Permeability[(b)] (cm/s)
B-1021	1	130	715.58	HQ	5	4.18×10^{-09}	1.74×10^{-09}
B-1021	2	101	744.58	HQ	5	1.86×10^{-08}	9.45×10^{-09}
B-1021	3	90	755.58	HQ	5	2.51×10^{-08}	1.33×10^{-08}
B-1021	4	65	780.58	HQ	5	0.00	0.00
B-1021	5	49	796.58	HQ	5	0.00	0.00
B-1021	6	24	821.58	HQ	5	0.00	0.00
B-2015	1	165	682.18	HQ	5	2.08×10^{-08}	7.31×10^{-09}
B-2015	2	133	714.18	HQ	5	2.78×10^{-08}	1.07×10^{-08}
B-2015	3	110	737.18	HQ	5	3.83×10^{-08}	1.74×10^{-08}
B-2015	4	95	752.18	HQ	5	2.24×10^{-08}	1.14×10^{-08}
B-2015	5	70	777.18	HQ	5	8.89×10^{-09}	5.64×10^{-09}
B-2015	6	58	789.18	HQ	5	1.30×10^{-09}	9.57×10^{-10}
B-2015	7	20	827.18	HQ	5	1.35×10^{-08}	1.16×10^{-08}
B-2018	1	154	695.07	HQ	5	3.05×10^{-09}	1.04×10^{-09}
B-2018	2	123	726.07	HQ	5	0.00	0.00

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Table 2.5.4-206 (Sheet 3 of 3)
Summary of In Situ Packer Tests

Boring No.	Test No.	Test Mid-Interval Depth (ft)	Test Mid-Interval Elevation (ft)	Borehole Size	Test Interval Length (ft)	Average Permeability[a] (cm/s)	Average Permeability[b] (cm/s)
B-2018	3	110	739.07	HQ	5	2.18×10^{-08}	9.95×10^{-09}
B-2018	4	96	753.07	HQ	5	0.00	0.00
B-2018	5	63	786.07	HQ	5	5.93×10^{-09}	3.73×10^{-09}
B-2018	6	56	793.07	HQ	5	0.00	0.00
B-2018	7	24	825.07	HQ	5	2.19×10^{-08}	1.91×10^{-08}
B-2020	1	131.5	701.3	HQ	5	1.06×10^{-08}	4.12×10^{-09}
B-2020	2A	116	716.8	HQ	5	7.89×10^{-09}	3.59×10^{-09}
B-2020	3A	91	741.8	HQ	5	1.03×10^{-08}	5.28×10^{-09}
B-2020	4	72	760.8	HQ	5	6.38×10^{-07}	4.00×10^{-07}
B-2020	5	47	785.8	HQ	5	6.19×10^{-08}	5.41×10^{-08}
B-2020	6	37	795.8	HQ	5	8.31×10^{-08}	8.34×10^{-08}

a) Permeability estimated based on Lugeon water test procedure

b) Permeability estimated based on the procedure recommended by U. S. Department of the Interior, Bureau of Reclamation (Engineering Geology Field Manual)

CP COL 2.5(1)

Table 2.5.4-207 (Sheet 1 of 4)
Summary of Borehole Geophysical Testing

				Tool and Survey Depth Range (ft)					
Boring No.	Surface Elevation[a] (ft)	Boring Total Depth (ft)	Run No.	Elog/ Gamma	Suspension	Caliper/ Gamma	Acoustic	Down-hole	Deviation
1000	851.01	400.0	1	374.0 - 35.0	1.6 - 342.9	355.0 - 0.0	306.0 - 0	2.5 - 135.0	--
1001	842.38	300.4	1	299.1 - 16.0	1.6 - 285.4	290.0 - 0.0	--	--	290.0 - 0.0
1002	844.66	300.2	1	288.0 - 35.0	1.6 - 275.6	285.0 - 0.0	--	--	221.0 - 1.1
			2	--	--	--	--	--	285.0 - 0.0
1003	861.69	336.5	1	290.0 - 34.8	269.0 - 1.6	290.1 - 1.4	--	--	290.2 - 2.8
			2	354.6 - 274.0	262.5 - 339.6	353.8 - 268.3	--	--	354.0 - 279.0
1004	859.70	301.5	1	299.1 - 16.0	1.6 - 285.4	290.0 - 0.0	--	--	290.0 - 0.0
1005	857.02	225.0	1	--	--	--	219.0 - 0.0	--	--
1008	847.56	225.2	1	--	--	--	224.0 - 0.0	--	--
1009-I	857.20	204.5	1	--	--	--	224.0 - 0.0	--	--
1011-I	854.71	204.6	1	--	--	--	88.0 - 5.0	--	--
			2	--	--	--	102.0 - 75.0	--	--
			3	--	--	--	200.0 - 5.0	--	--
			4	--	--	--	223.0 - 188.0	--	--

CP COL 2.5(1)

Table 2.5.4-207 (Sheet 2 of 4)
Summary of Borehole Geophysical Testing

Boring No.	Surface Elevation[(a)] (ft)	Boring Total Depth (ft)	Run No.	Tool and Survey Depth Range (ft)					
				Elog/ Gamma	Suspension	Caliper/ Gamma	Acoustic	Down-hole	Deviation
1012	844.04	550.0	1	342.6 - 30.0	415.0 - 1.6	427.4 - 6.0	--	--	398.5 - 2.6
			2	430.0 - 296.6	413.4 - 536.4	550.5 - 399.5	--	--	550.3 - 396.4
			3	310.0 - 37.0	--	--	--	--	--
			4	550.2 - 380.0	--	--	--	--	--
1013	863.53	150.0	1	--	--	--	149.4 - 3.9	--	--
1014	851.20	150.0	1	--	--	--	149.8 - 3.7	--	--
1015	858.09	151.4	1	--	--	--	149.0 - 4.0	--	--
1029	846.88	150.0	1	--	--	--	149.5 - 3.8	--	--
1047	831.11	150.0	1	--	4.9 - 136.2	--	--	--	3.5 - 147.6
			2	--	--	--	--	--	149.5 - 3.8
1050	838.50	150.0	1	--	57.4 - 136.2	--	--	--	1.7 - 149.0
			2	--	6.6 - 57.4	--	--	--	148.5 - 1.6
			3	--	--	--	--	--	3.2 - 65.0
			4	--	--	--	--	--	65.0 - 3.2
2000	844.03	401.1	1	393.0 - 238.1	1.6 - 380.6	391.0 - 0.0	392.0 - 0.0	5.0 - 144.0	--
			2	392.0 - 34.0	--	--	--	--	--

CP COL 2.5(1)

Table 2.5.4-207 (Sheet 3 of 4)
Summary of Borehole Geophysical Testing

Boring No.	Surface Elevation[a] (ft)	Boring Total Depth (ft)	Run No.	Tool and Survey Depth Range (ft)					
				Elog/ Gamma	Suspension	Caliper/ Gamma	Acoustic	Down-hole	Deviation
2001	837.89	301.0	1	289.0 - 34.8	275.6 - 1.6	288.5 - 276.4	--	--	267.5 - 3.2
			2	--	--	273.4 - 5.2	--	--	--
2002	837.21	302.0	1	300.0 - 39.0	6.5 - 272.3	288.0 - 2.0	--	--	228.0 - 1.3
2003 OFFSET	841.14	253.0	1	266.6 - 34.9	203.4 - 1.6	340.0 - 2.3	--	--	336.5 - 199.2
			2	336.5 - 2.0	196.9 - 311.7	--	--	--	--
2004	850.53	300.0	1	291.0 - 35.0	275.6 - 1.6	287.0 - 0.4	--	--	288.9 - 2.8
2005	848.72	226.0	1	--	--	--	224.0 - 0.0	--	--
2008	840.62	225.0	1	--	--	--	197.9 - 3.8	--	--
			2	--	--	--	35.5 - 3.8	--	--
2009-I	842.97	204.6	1	--	--	--	224.4 - 2.1	--	--
2011-I	847.08	204.5	1	--	--	--	224.0 - 2.5	--	--
2012	847.19	150.0	1	--	--	--	149.0 - 3.9	--	--
2014	839.41	150.0	1	--	--	--	149.3 - 4.3	--	--
2029	843.58	151.0	1	--	--	--	149.7 - 3.6	--	--
2048	844.18	151.7	1	--	6.6 - 137.8	--	--	--	4.5 - 151.2
			2	--	--	--	--	--	151.0 - 3.5

CP COL 2.5(1)

Table 2.5.4-207 (Sheet 4 of 4)
Summary of Borehole Geophysical Testing

				Tool and Survey Depth Range (ft)					
Boring No.	Surface Elevation[(a)] (ft)	Boring Total Depth (ft)	Run No.	Elog/ Gamma	Suspension	Caliper/ Gamma	Acoustic	Down-hole	Deviation
2051	809.97	100.4	1	--	4.6 - 86.9	--	--	--	3.7 - 100.0
			2	--	--	--	--	--	99.8 - 1.7

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

CP COL 2.5(1)

Table 2.5.4-208 (Sheet 1 of 4)
Summary of In Situ Pressuremeter Testing

Boring No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type	Engineering Layer	unload/reload Shear Modulus, G_1 (psi)	initial Shear Modulus, G_0 (psi)
1013	823.1	40.4	Limestone	A	850,000	220,000
1013	821.6	41.9	Limestone	A	13,000,000	300,000
1013	806.2	57.3	Shale	A	220,000	77,000
1013	804.7	58.8	Limestone	A	600,000	330,000
1013	793.3	70.2	Shale	B	160,000	73,000
1013	791.8	71.7	Shale	B	170,000	90,000
1013	773.3	90.2	Limestone	C	700,000	260,000
1013	771.8	91.7	Limestone	C	300,000	300,000
1013	750.6	112.9	Limestone	C	720,000	440,000
1013	749.1	114.4	Limestone	C	2,000,000	500,000
1014	816.7	34.5	Limestone	A	2,000,000	450,000
1014	815.2	36.0	Limestone	A	780,000	300,000
1014	796.7	54.5	Shale	B	750,000	185,000
1014	795.2	56.0	Shale	B	65,000	27,000
1014	776.7	74.5	Limestone	C	1,000,000	364,000
1014	775.2	76.0	Limestone	C	750,000	300,000
1014	756.7	94.5	Limestone	C	1,100,000	370,000

CP COL 2.5(1)

Table 2.5.4-208 (Sheet 2 of 4)
Summary of In Situ Pressuremeter Testing

Boring No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type	Engineering Layer	unload/reload Shear Modulus, G_1 (psi)	initial Shear Modulus, G_0 (psi)
1014	755.2	96.0	Limestone	C	1,700,000	650,000
1015	811.9	46.2	Limestone	A	670,000	200,000
1015	809.4	48.7	Shale	A	340,000	23,000
1015	795.8	62.3	Shale	B	88,000	30,000
1015	794.3	63.8	Limestone	B	850,000	200,000
1015	770.5	87.6	Limestone	C	853,000	420,000
1015	769.0	89.1	Limestone	C	850,000	350,000
1015	750.8	107.3	Limestone	C	570,000	136,000
1015	749.3	108.8	Limestone	C	720,000	111,000
1029	815.9	31.0	Limestone	A	750,000	300,000
1029	814.4	32.5	Limestone	A	750,000	300,000
1029	795.9	51.0	Shale	B	380,000	60,000
1029	794.4	52.5	Shale	B	410,000	410,000
1029	791.4	55.5	Shale	B	260,000	130,000
1029	789.9	57.0	Shale	B	143,000	60,000
1029	775.9	71.0	Limestone	C	1,300,000	350,000
1029	774.4	72.5	Limestone	C	1,100,000	600,000

CP COL 2.5(1)

Table 2.5.4-208 (Sheet 3 of 4) Summary of In Situ Pressuremeter Testing

Boring No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type	Engineering Layer	unload/reload Shear Modulus, G_1 (psi)	initial Shear Modulus, G_0 (psi)
2012	816.6	30.6	Shale	A	450,000	120,000
2012	815.1	32.1	Limestone	A	850,000	200,000
2012	792.2	55.0	Shale	B	130,000	23,000
2012	790.7	56.5	Shale	B	170,000	74,000
2012	776.9	70.3	Limestone	C	850,000	300,000
2012	775.4	71.8	Limestone	C	780,000	426,000
2012	756.7	90.5	Limestone	C	420,000	260,000
2012	755.2	92.0	Limestone	C	850,000	420,000
2014	816.4	23.0	Shale	A	45,000	16,000
2014	814.9	24.5	Limestone	A	600,000	160,000
2014	795.9	43.5	Limestone	A	850,000	430,000
2014	794.4	45.0	Limestone	B	600,000	220,000
2014	775.9	63.5	Limestone	C	1,700,000	350,000
2014	774.4	65.0	Limestone	C	1,300,000	350,000
2014	755.9	83.5	Limestone	C	850,000	350,000
2014	754.4	85.0	Limestone	C	750,000	224,000
2029	816.1	27.5	Limestone	A	780,000	350,000

CP COL 2.5(1)

Table 2.5.4-208 (Sheet 4 of 4)
Summary of In Situ Pressuremeter Testing

Boring No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type	Engineering Layer	unload/reload Shear Modulus, G_1 (psi)	initial Shear Modulus, G_0 (psi)
2029	814.6	29.0	Limestone	A	677,000	400,000
2029	806.6	37.0	Shale	A	220,000	60,000
2029	805.1	38.5	Limestone	A	1,000,000	300,000
2029	788.7	54.9	Limestone	B	800,000	220,000
2029	787.2	56.4	Shale	B	260,000	130,000
2029	776.3	67.3	Limestone	C	1,000,000+	--
2029	774.8	68.8	Limestone	C	1,000,000	350,000
2029	756.1	87.5	Limestone	C	850,000	370,000
2029	754.6	89.0	Limestone	C	2,600,000	750,000

CP COL 2.5(1)

Table 2.5.4-209 (Sheet 1 of 13)
Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1003-off-2	856.7	5.0	Residual Soil	20.5	-	-	-	26	12	-
1032-5	854.8	12.5	Residual Soil	-	-	-	-	25	9	-
1017-4	853.9	12.5	Residual Soil	-	-	-	-	31	17	-
1030-3	851.1	7.5	Residual Soil	21.0	-	-	-	31	16	-
1025-5	849.0	12.5	Residual Soil	-	-	-	-	44	30	-
2032-3	845.5	7.5	Residual Soil	-	-	-	-	60	43	-
2010-off-1	843.9	2.5	Residual Soil	4.8	-	-	-	36	19	-
1014-3	843.7	7.5	Residual Soil	14.4	-	-	-	50	36	-
1008-off-2	842.5	5.0	Residual Soil	11.5	-	-	-	26	12	-
2011-I-off-2	842.1	5.0	Residual Soil	11.5	-	-	-	55	38	-
1018-2	841.4	5.0	Residual Soil	17.8	-	-	-	45	30	-
1014-4	841.2	10.0	Residual Soil	-	-	-	-	27	13	-
2010-off-3	839.9	6.5	Residual Soil	-	-	-	-	26	12	-
2003-off-3	831.0	10.0	Residual Soil	14.6	-	-	-	32	16	-
2001-off-3	830.4	7.5	Residual Soil	-	-	-	-	36	20	-
2040-3	827.6	7.5	Residual Soil	-	-	-	-	32	17	-
2006-07	819.6	25.4	Residual Soil	-	-	-	-	43	26	-
1033-07	838.2	28.8	Shale	9.9	137.2	124.8	27.3	27	14	-
2004-06	829.8	20.2	Shale	12.0	141.3	126.2	26.5	33	18	-
1002-08	821.9	22.8	Shale	-	-	-	-	69	45	-

CP COL 2.5(1)

Table 2.5.4-209 (Sheet 2 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1003-09	821.8	39.9	Shale	18.3	138.1	116.7	32.1	-	-	-
2058-9	821.3	32.4	Shale	-	-	-	-	-	-	2.75
2006-07	820.6	25.4	Shale	17.5	131.7	112.1	34.7	-	-	-
1004-12	820.0	39.7	Shale	18.4	130.2	110.0	35.9	45	28	-
2036-04	818.8	14.2	Shale	17.6	131.6	111.9	34.9	-	-	-
2000-02	817.1	26.9	Shale	21.0	130.9	108.2	37.1	-	-	-
2000a-08	815.2	28.8	Shale	18.5	118.6	100.1	41.7	-	-	-
1000-10	813.6	37.4	Shale	-	-	-	-	42	21	-
1002-10	809.3	35.4	Shale	13.4	137.7	121.4	29.3	-	-	-
2001 @ 28.9'	809.0	28.9	Shale	16.1	117.7	101.3	41.0	-	-	-
2030-07	806.9	45.1	Shale	15.6	141.4	122.3	28.9	-	-	-
2036-07	806.7	26.3	Shale	16.7	133.1	114.0	33.6	48	26	-
1003-12	806.7	55.0	Shale	-	-	-	-	51	25	-
2000-05	802.3	41.7	Shale	22.4	121.6	99.3	42.3	-	-	-
1035-13	799.5	56.6	Shale	19.7	135.9	113.5	34.0	53	34	-
1005-15	797.5	59.5	Shale	12.2	143.0	127.5	25.9	-	-	-
1038-14	797.3	46.2	Shale	17.2	130.4	111.3	35.3	56	44	-
1042-13	797.0	49.4	Shale	-	-	-	-	44	26	-
1058-12	796.8	48.4	Shale	8.5	141.6	130.5	24.0	-	-	-
1041-11	796.3	49.0	Shale	18.1	130.9	110.8	35.4	56	33	-
1003-14	796.3	65.4	Shale	17.4	135.1	115.0	33.1	-	-	-

CP COL 2.5(1)

Table 2.5.4-209 (Sheet 3 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1042-15	795.5	50.9	Shale	13.3	139.8	123.4	26.6	47	29	-
1029-13	795.2	51.7	Shale	20.4	128.3	106.3	37.8	-	-	-
1029-13	795.2	51.7	Shale	-	-	-	-	51	30	-
1041-12	795.0	50.3	Shale	19.2	132.2	110.9	35.5	-	-	-
2042a-12	794.4	36.6	Shale	19.0	130.9	110.0	36.0	-	-	-
2000-07	794.2	49.8	Shale	11.2	144.5	130.0	24.4	-	-	-
2035-10	794.0	36.0	Shale	-	-	-	-	38	23	-
2051-4	794.0	16.0	Shale	17.2	125.0	106.7	37.9	-	-	-
2042a-11	793.9	37.1	Shale	20.3	124.2	103.2	40.0	55	32	-
2003-07	793.9	47.1	Shale	-	-	-	-	46	30	-
1003-15	793.4	68.3	Shale	12.8	147.5	130.8	23.9	-	-	-
2008-09	793.3	47.7	Shale	18.2	133.8	113.2	34.2	-	-	-
1003-off-18	793.3	68.4	Shale	-	-	-	-	43	25	-
1013-15	792.8	70.8	Shale	11.0	142.7	128.5	25.3	-	-	-
1006-13	792.1	55.6	Shale	-	-	-	-	50	33	-
1006-14	791.6	56.1	Shale	14.1	139.1	122.0	29.1	-	-	-
2036-10	790.7	42.3	Shale	14.4	134.0	117.2	31.7	45	29	-
2003-08	790.3	50.7	Shale	14.6	139.0	121.3	29.5	-	-	-
2029-12	789.5	54.6	Shale	15.7	137.0	118.4	31.2	-	-	-
1005-17	788.2	68.8	Shale	17.2	133.1	113.6	33.9	46	27	-
2037-13	788.0	49.0	Shale	18.2	138.4	117.0	32.0	-	-	-

CP COL 2.5(1)

Table 2.5.4-209 (Sheet 4 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1000-15	787.6	63.4	Shale	13.7	135.1	118.8	30.8	40	24	-
1042-15b	787.1	59.4	Shale	-	-	-	-	36	22	-
1042-15bb	787.0	59.5	Shale	-	-	-	-	38	24	-
1042-15c	786.3	60.1	Shale	10.4	144.1	130.6	24.1	-	-	-
1041-13	786.3	59.0	Shale	11.8	143.0	127.9	25.6	-	-	-
1038-16	785.4	58.1	Shale	12.7	141.8	125.8	27.1	38	24	-
2006-14	785.0	61.0	Shale	12.4	142.2	126.5	26.4	-	-	-
1037-12	784.8	68.0	Shale	16.3	128.8	110.8	35.5	56	35	-
2042a-14	783.9	47.1	Shale	16.5	131.7	113.0	34.3	-	-	-
1002-15	783.7	61.0	Shale	14.5	139.4	121.8	29.2	48	31	-
1005-18	783.1	73.9	Shale	12.4	136.2	121.1	29.5	46	29	-
2035-13	782.5	47.5	Shale	-	-	-	-	40	26	-
2034-16	781	58	Shale	-	-	-	-	-	-	2.77
1041-15	777.6	67.7	Shale	6.7	156.5	146.7	13.2	-	-	-
2036-12	775.9	57.1	Shale	16.1	135.3	116.5	32.3	-	-	-
1012-21	735.0	109.0	Shale	-	-	-	-	28	16	-
2002-22	720.4	116.8	Shale	-	-	-	-	27	16	-
1041-27	716.3	129.0	Shale	-	-	-	-	65	43	-
1041-27a	715.9	129.4	Shale	-	-	-	-	71	48	-
1004-33	715.6	144.1	Shale	21.2	131.4	108.4	37.0	-	-	-
2001-22	715.0	123.0	Shale	19.1	168.5	141.5	17.7	-	-	2.78

CP COL 2.5(1)

Table 2.5.4-209 (Sheet 5 of 13)
Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1000-30	714.6	136.4	Shale	11.8	139.3	124.6	27.6	-	-	-
2037-28	712.5	124.5	Shale	22.5	133.1	108.7	36.8	-	-	-
2034-30	711.1	127.9	Shale	11.4	144.0	129.3	24.8	-	-	-
2006-30	705.0	141.0	Shale	8.8	151.9	139.6	18.8	-	-	-
2000-35	652.7	191.3	Shale	14.0	139.0	122.0	29.1	-	-	-
1012-34	650.0	194.0	Shale	16.4	138.3	118.9	30.9	-	-	2.75
2000-36	647.3	196.7	Shale	16.0	135.0	117.0	32.0	-	-	-
2000-37	645.2	198.8	Shale	12.5	133.5	118.7	31.0	-	-	-
1000-39	639.0	212.0	Shale	10.9	148.0	133.5	22.4	-	-	-
1004-51	599.2	260.5	Shale	20.1	131.9	109.8	36.2	-	-	-
1012-41	562.993	281.05	Shale	11.5	136.6	122.5	28.8	-	-	-
1012-42	546.093	297.95	Shale	12.8	129.8	115.1	33.1	-	-	-
1000-50	498.508	352.5	Shale	9.8	148.5	135.2	21.4	-	-	2.75
1012-51	363.493	480.55	Shale	7.4	155.1	144.4	16.0	-	-	-
1012-56	330.793	513.25	Shale	5.5	154.2	146.2	15.0	-	-	2.74
1035-07	831.5	24.6	Limestone	-	164.1	-	-	-	-	-
1005-09	828.0	29.0	Limestone	1.0	160.0	158.3	6.3	-	-	-
1033-09	827.9	39	Limestone	-	-	-	-	-	-	2.71
1030-11	826.6	32.0	Limestone	2.4	162.4	158.6	6.1	-	-	-
2000-01	824.8	19.2	Limestone	1.6	155.4	152.9	9.5	-	-	-
1032-12	824.0	43.3	Limestone	-	164.6	-	-	-	-	-

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Table 2.5.4-209 (Sheet 6 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1037-05	818.2	34.6	Limestone	7.3	150.8	140.6	16.8	-	-	-
1009-I-05	816.7	40.5	Limestone	5.3	156.8	148.9	11.9	-	-	-
2005-08	816.7	32.3	Limestone	-	164.9	-	-	-	-	-
1030-13	815.6	43.0	Limestone	5.3	155.9	148.1	12.4	-	-	-
1003-off-13	815.3	46.4	Limestone	4.3	152.9	146.7	13.2	-	-	-
1004-13	814.2	45.5	Limestone	6.8	151.2	141.6	16.2	-	-	-
2000-03	813.3	30.7	Limestone	4.4	155.6	149.0	11.8	-	-	-
1033-12	813.3	53.6	Limestone	4.0	156.9	150.9	10.7	-	-	-
1029-09	813.3	30.3	Limestone	3.5	158.45	153.14	9.4	-	-	-
1002-09	812.8	31.9	Limestone	5.3	154.2	-	-	-	-	-
1005-12	812.2	44.8	Limestone	5.3	155.0	147.2	12.9	-	-	-
2031-11	810.2	42.9	Limestone	5.3	155.0	147.2	12.9	-	-	-
2036-06	809.2	23.8	Limestone	3.8	155.9	150.2	11.1	-	-	-
1013 @ 55.5'	808.0	55.5	Limestone	3.4	152.1	147.0	13.0	-	-	-
2003-04	808.0	33.0	Limestone	4.1	157.3	151.1	10.6	-	-	2.69
1041-09	807.0	38.0	Limestone	5.8	155.7	147.1	13.0	-	-	-
1013-12	805.3	58.2	Limestone	1.7	162.9	160.3	5.2	-	-	-
1013-19	805.3	58.2	Limestone	4.4	156.8	150.1	11.2	-	-	-
1031-14	803.8	60.4	Limestone	4.6	154.8	147.9	12.5	-	-	-
1007-11	801.3	42.3	Limestone	2.9	159.8	155.3	8.1	-	-	-
2030-08	801.3	50.7	Limestone	4.7	156.2	149.1	11.8	-	-	-

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Table 2.5.4-209 (Sheet 7 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
2037-10	800.3	36.7	Limestone	1.5	160.6	158.2	6.4	-	-	-
2005-11	799.4	49.6	Limestone	3.0	160.4	155.7	7.9	-	-	-
2003-06	798.5	42.5	Limestone	4.2	157.5	151.2	10.5	-	-	-
2014 @ 43.6'	797.1	43.6	Limestone	5.7	152.4	144.3	14.6	-	-	-
1009-I-09	797.0	60.2	Limestone	4.3	157.9	151.4	10.4	-	-	-
2030-11	789.2	63.4	Limestone	6.3	154.6	145.4	14.0	-	-	-
2007-13	786.1	54.9	Limestone	2.9	159.1	155.1	8.2	-	-	-
1034-18	785.1	73.4	Limestone	5.1	160.5	152.8	9.6	-	-	-
1038-17	781.1	62.4	Limestone	3.6	159.2	153.6	9.1	-	-	-
2004-16	779.6	70.9	Limestone	0.9	160.7	159.2	5.8	-	-	-
2042a-15	779.1	51.9	Limestone	2.6	163.4	159.3	5.7	-	-	-
1000-17	779.0	72.0	Limestone	-	151.0	-	-	-	-	-
2012 @ 69'	778.6	69.0	Limestone	3.5	162.3	156.8	7.2	-	-	-
2012-20	778.1	69.5	Limestone	-	157.1	-	-	-	-	-
1014-17	777.5	73.7	Limestone	3.6	154.8	149.5	11.5	-	-	-
2036-13	774.6	57.8	Limestone	4.0	155.5	149.5	11.6	-	-	-
2006-16	774.5	70.5	Limestone	4.2	154.9	148.6	12.0	-	-	-
1032-22	773.6	93.7	Limestone	3.6	157.9	152.4	9.8	-	-	-
1041-16	772.7	72.6	Limestone	6.9	153.0	143.1	15.3	-	-	-
1005-20	772.5	84.5	Limestone	-	156.3			-	-	-
2001-10	772.1	65.8	Limestone	4.6	155.0	148.2	12.3	-	-	-

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Table 2.5.4-209 (Sheet 8 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1013-23	771.7	91.8	Limestone	5.9	152.1	143.6	15.0	-	-	-
1003-off-22	771.6	90.1	Limestone	5.1	154.9	147.3	10.6	-	-	-
1030-22	771.0	87.6	Limestone	4.5	157.1	150.3	11.1	-	-	-
1031-21	770.8	93.4	Limestone	4.8	155.2	148.1	12.3	-	-	-
1034-21	768.9	89.6	Limestone	5.7	154.3	146.0	13.6	-	-	-
2041a-15	768.7	63.3	Limestone	8.0	151.9	140.7	16.7	-	-	-
2002-13R	768.2	68.8	Limestone	3.5	158.5	153.1	9.4	-	-	-
2042a-17	767.8	62.9	Limestone	5.8	152.7	144.3	14.6	-	-	-
1029-18	767.8	75.8	Limestone	6.7	155.01	145.33	14.0	-	-	-
1035-20	767.4	88.7	Limestone	6.8	149.5	140.0	17.2	-	-	-
2038-13	767.1	69.9	Limestone	5.4	153.5	145.6	13.8	-	-	-
1041-17	766.8	78.2	Limestone	4.2	157.2	150.9	10.7	-	-	-
1042-19	765.6	80.9	Limestone	4.2	157.0	150.7	10.8	-	-	-
1010-22	764.4	97	Limestone	-	-	-	-	-	-	2.72
2000-13	763.7	80.3	Limestone	4.5	154.7	148.0	12.4	-	-	-
2000-13a	763.7	80.3	Limestone	1.3	149.3	147.3	12.8	-	-	-
1034-23	761.6	99.4	Limestone	-	160.7	-	-	-	-	-
2006-19	760.8	85.2	Limestone	5.2	155.0	147.3	12.8	-	-	-
1003-off-24	760.7	101.0	Limestone	7.2	149.4	139.5	17.5	-	-	-
2030-17	759.1	93.5	Limestone	6.5	152.8	143.5	15.1	-	-	-
1037-17	757.9	94.9	Limestone	6.1	155.2	146.3	13.4	-	-	-

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Table 2.5.4-209 (Sheet 9 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1042-21	757.5	88.9	Limestone	4.3	159.3	152.8	10.4	-	-	-
2037-19	757.5	79.5	Limestone	-	-	-	-	-	-	2.72
1014-21	756.9	94.3	Limestone	2.2	161.2	157.7	6.7	-	-	-
1004-25	756.0	104.0	Limestone	3.0	160.5	155.9	7.8	-	-	-
1012-17	755.0	89.0	Limestone	2.4	161.0	157.3	6.9	-	-	-
2014-16	752.4	87.0	Limestone	7.4	153.4	142.8	15.5	-	-	-
1041-20	752.0	93.0	Limestone	4.1	157.3	151.1	10.6	-	-	-
2005-21	751.6	97.1	Limestone	5.7	153.7	145.4	14.0	-	-	-
2002-16	750.6	86.4	Limestone	4.4	156.0	149.4	11.6	-	-	-
1038-24	745.7	97.8	Limestone	5.7	154.9	146.5	13.3	-	-	-
2041a-20	745.3	86.7	Limestone	2.7	156.7	152.6	9.7	-	-	-
2000-17A	742.7	101.3	Limestone	0.8	147.4	146.2	13.5	-	-	-
2000-17	742.2	101.8	Limestone	5.1	155.3	147.8	12.5	-	-	-
2000-18	742.1	101.9	Limestone	-	162.2	-	-	-	-	-
1000-24	741.6	109.4	Limestone	-	153.5	-	-	-	-	-
2035-21	739.5	90.3	Limestone	7.8	150.9	140.0	17.2	-	-	-
2042a-23	738.4	92.6	Limestone	3.9	157.7	151.8	10.2	-	-	-
2037-23	737.7	99.3	Limestone	6.4	152.2	143.1	15.3	-	-	-
2035-22	735.9	93.9	Limestone	5.7	154.3	146.0	13.6	-	-	-
2004-25	735.3	114.7	Limestone	5.1	155.3	147.7	12.6	-	-	-
2000-19	732.9	111.1	Limestone	6.9	152.0	142.2	15.9	-	-	-

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Table 2.5.4-209 (Sheet 10 of 13)
Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1003-off-30	731.1	130.6	Limestone	5.1	135.6	146.2	13.5	-	-	-
1012-22	730.0	114.0	Limestone	3.4	155.9	150.7	10.8	-	-	-
1000-27	728.2	122.8	Limestone	9.5	153.1	139.9	17.2	-	-	-
2000-20	726.5	117.5	Limestone	4.9	160.9	153.4	9.2	-	-	-
2002-22	720.2	116.8	Limestone	7.9	149.0	138.0	18.3	-	-	-
2000-22	718.5	125.5	Limestone	4.6	155.9	149.0	11.8	-	-	-
2002-23	716.0	121.0	Limestone	4.2	157.8	151.4	10.4	-	-	-
1002-29	714.3	130.4	Limestone	7.2	153.0	142.6	15.6	-	-	-
1002-30	710.1	134.6	Limestone	18.9	141.4	118.9	29.6	-	-	-
1012-27	705.0	139.0	Limestone	3.6	160.7	155.2	8.2	-	-	-
2000-25	704.2	139.8	Limestone	2.6	161.2	157.1	7.0	-	-	-
1012-28	700.0	144.0	Limestone	3.7	156.5	150.9	10.7	-	-	-
1000-33	699.5	151.5	Limestone	-	-	-	-	-	-	2.72
2000-26	699.0	145.0	Limestone	3.4	157.4	152.2	10.0	-	-	-
2000-27	691.8	152.2	Limestone	5.1	156.1	148.5	12.1	-	-	-
2000-28	688.8	155.2	Limestone	8.2	147.7	137.4	18.7	-	-	-
1004-38	684.4	175.6	Limestone	3.8	157.6	151.8	10.2	-	-	-
2002-31	675.3	161.9	Limestone	3.8	156.6	150.9	10.7	-	-	-
1004-41	674.2	185.5	Limestone	2.9	152.8	148.5	12.1	-	-	-
2000-32	668.2	175.8	Limestone	3.3	158.8	153.8	9.0	-	-	-
1012-33	660.0	184.0	Limestone	3.7	157.8	152.2	9.9	-	-	-

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Table 2.5.4-209 (Sheet 11 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
2000-34	657.8	186.2	Limestone	3.5	158.5	153.2	9.3	-	-	2.70
2002-39	638.2	198.8	Limestone	2.8	161.0	156.5	7.4	-	-	-
1003-off-16	799.6	62.1	Limestone/Shale	9.1	146.3	134.1	21.3	-	-	-
1003-16	787.9	73.8	Limestone/Shale	13.6	140.5	123.6	27.5	-	-	-
1006-13	792.1	55.6	Limestone/Shale	15.9	141.0	121.6	28.7	-	-	-
1012-11	785.0	59.0	Limestone/Shale	9.6	142.5	130.0	23.8	-	-	-
1042-15b	786.7	59.7	Limestone/Shale	8.7	141.4	130.1	23.7	-	-	-
1042-15d	784.5	61.9	Limestone/Shale	14.8	135.6	118.1	30.7	-	-	-
1043-7	793.4	31.1	Limestone/Shale	13.8	137.0	120.4	29.5	-	-	-
1048-14	783.3	52.5	Limestone/Shale	16.1	132.3	114.0	33.3	-	-	-
2014-8	796.7	44.0	Limestone/Shale	10.4	145.6	131.8	22.7	-	-	-
2000-24	708.0	136.0	Limestone/Shale	8.5	149.0	137.4	19.4	-	-	-
1004-46	650.7	209.0	Limestone/Shale	13.2	153.6	135.6	20.5	-	-	-
1004-48	629.9	229.8	Limestone/Shale	13.1	136.7	120.8	29.1	-	-	-
2000-40	628.7	215.3	Limestone/Shale	9.0	145.0	133.0	22.0	-	-	-
2000-72	466.6	377.4	Limestone/Shale	11.7	138.2	123.7	27.4	-	-	-
2000-76	448.9	395.1	Limestone/Shale	10.0	142.0	129.0	24.3	-	-	-
2000-43	614.8	229.2	Sandstone	17.0	132.0	113.0	31.6	-	-	-
2003-35	614.4	226.6	Sandstone	13.3	132.5	117.0	29.4	-	-	-
2003-36	613.1	227.9	Sandstone	18.8	124.4	104.7	36.7	-	-	-
1004-53	579.7	280	Sandstone	-	-	-	-	-	-	2.65

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Table 2.5.4-209 (Sheet 12 of 13) Summary of Index Properties

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity
1012-40	575.0	269.0	Sandstone	13.8	133.0	116.9	29.2	-	-	-
1004-54	569.7	290.0	Sandstone	11.3	136.3	122.5	25.8	-	-	-
1000-46	568.7	282.3	Sandstone	8.4	140.8	129.8	21.4	-	-	-
2000-52	568.5	275.5	Sandstone	14.4	-	-	-	-	-	-
2002-60R	538.7	298.3	Sandstone	17.3	124.3	106.0	35.8	-	-	-
2000-60	529.7	314.3	Sandstone	18.0	131.0	111.0	32.8	-	-	-
2000-64	508.8	335.2	Sandstone	11.0	140.0	126.0	23.7	-	-	-
1012-44	506.0	338.1	Sandstone	8.2	145.4	134.4	18.6	-	-	-
2000-68	488.8	355.2	Sandstone	11.2	146.6	131.8	20.2	-	-	-
1000-51	475.3	375.7	Sandstone	9.3	145.0	132.6	19.7	-	-	-

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TABLE 2.5.4-209 (Sheet 13 of 13)
Summary of Index Properties

Statistical Summary

		Material Type		Moisture Content (%A)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Calculated Porosity (%)	Liquid Limit
Average	Soil	14.5	-	-	-	36	21	-
	Shale	15.0	137.3	119.8	30.3	46	28	2.76
	Limestone	4.7	155.9	149.0	11.8	-	-	2.71
	Limestone/Shale	11.8	141.8	126.9	25.6	-	-	-
	Sandstone	13.2	135.9	120.5	27.1	-	-	2.65
Minimum	Soil	4.8	-	-	-	25	9	-
	Shale	5.5	117.7	99.3	13.2	27	14	2.74
	Limestone	0.8	135.6	118.9	5.2	-	-	2.69
	Limestone/Shale	8.5	132.3	114.0	19.4	-	-	-
	Sandstone	8.2	124.3	104.7	18.6	-	-	2.65
Maximum	Soil	21.0	-	-	-	60	43	-
	Shale	22.5	168.5	146.7	42.3	71	48	2.78
	Limestone	18.9	164.9	160.3	29.6	-	-	2.72
	Limestone/Shale	16.1	153.6	137.4	33.3	-	-	-
	Sandstone	18.8	146.6	134.4	36.7	-	-	2.65
Standard Deviation	Soil	5.0	-	-	-	10	10	-
	Shale	3.9	8.8	10.9	6.4	11	8	0.02
	Limestone	2.2	4.4	6.0	3.5	-	-	0.01
	Limestone/Shale	2.6	5.4	6.8	4.0	-	-	-
	Sandstone	3.5	7.4	10.1	6.1	-	-	-

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Table 2.5.4-210
Summary of Index Properties Statistical Data

Material Type/ Engineering Layer	Moisture Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Atterberg Limits	
				Liquid Limit (%)	Plasticity Index (%)
	Mean ± Stdev [Count]	Mean ± Stdev [Count]	Mean ± Stdev [Count]	Mean ± Stdev [Count]	Mean ± Stdev [Count]
Residual Soil	14.5 ± 5.0 [8]	-	-	36 ± 10 [18]	21 ± 10 [18]
Shale	15.0 ± 3.9 [63]	137.3 ± 8.8 [63]	119.8 ± 10.9 [63]	46 ± 11 [32]	28 ± 8 [32]
Limestone	4.7 ± 2.2 [100]	155.9 ± 4.4 [109]	149.0 ± 6.0 [99]	-	-
Limestone/Shale	11.8 ± 2.6 [15]	141.8 ± 5.4 [15]	126.9 ± 6.8 [15]	-	-
Sandstone	13.2 ± 3.5 [13]	135.9 ± 7.4 [12]	120.5 ± 10.1 [12]	-	-
Layer A	8.8 ± 6.6 [37]	149.1 ± 12.8 [40]	136.7 ± 18.9 [36]	45 ± 12 [9]	26 ± 8 [9]
Layer B	13.1 ± 4.6 [46]	139.6 ± 8.8 [46]	124.0 ± 13.0 [46]	46 ± 6 [18]	29 ± 5 [18]
Layer C	5.1 ± 2.3 [56]	155.0 ± 5.0 [62]	147.8 ± 6.6 [46]	35 ± 11 [3]	21 ± 7 [3]
Layer D	13.9 ± 6.6 [7]	146.7 ± 12.6 [7]	129.5 ± 15.6 [7]	68 ± 3 [2]	46 ± 3 [2]
Layer E	8.0 ± 5.1 [24]	150.6 ± 8.8 [24]	140.1 ± 13.9 [24]	-	-
Layer F	17.3 ± 2.6 [4]	130.2 ± 3.4 [4]	111.1 ± 4.5 [4]	-	-
Layer G	13.2 ± 2.9 [9]	134.0 ± 5.2 [8]	118.7 ± 7.4 [8]	-	-
Layer H	10.0 ± 1.2 [6]	144.3 ± 3.3 [6]	131.1 ± 3.9 [6]	-	-
Layer I	6.5 ± 1.0 [2]	154.7 ± 0.5 [2]	145.3 ± 0.9 [2]	-	-

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Table 2.5.4-211
Summary of Slake Durability Test Results

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Moisture Content (%)	Average Water Temperature (°C)	Slake Durability (%)	Classification
1000-04	840.5	10.5	Shale	7.2	22.4	83.2	Low Durability
1003-05	841.6	20.4	Limestone	5.0	22.4	91.1	Medium Durability
1003-14	796.6	65.4	Shale	17.8	22.6	0.2	Very Low Durability
1033-06	842.9	24.2	Limestone	9.8	22.5	92.0	Medium Durability
1033-09	827.9	39.0	Limestone	1.7	22.6	98.3	High Durability
2031-06	835.0	18.0	Shale	10.9	22.6	66.1	Low Durability
2031-10	816.7	36.3	Limestone	4.0	22.6	98.1	High Durability

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Table 2.5.4-212 (Sheet 1 of 2) Summary of Calcium Carbonate Test Results

Boring No.	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	Calcium Carbonate, $CaCO_3$ (%)
1012	1012-17	755.0	89.0	Limestone	90
1030	1030-11	826.6	32.0	Limestone	86
1032	1032-15	806.8	60.5	Limestone	91
2000	2000-01	824.8	19.2	Limestone	100
2000	2000-02	817.1	26.9	Shale	72
2000	2000-03	813.3	30.7	Limestone	56
2000	2000-13	763.7	80.3	Limestone	75
2000	2000-17	742.1	101.8	Limestone	76
2000	2000-19	732.9	111.1	Shale	77
2000	2000-22	718.5	125.5	Limestone	96
2000	2000-24	708.0	136.0	Shale	36
2000	2000-25	704.2	139.8	Limestone	94
2000	2000-28	688.8	155.2	Limestone	77
2000	2000-32	668.2	175.8	Limestone	95
2000	2000-34	657.8	186.2	Limestone	84
2000	2000-35	652.7	191.3	Shale	17
2000	2000-36	647.3	196.7	Shale	4
2000	2000-40	628.7	215.3	Shale	49
2000	2000-43	614.8	229.2	Sandstone	2
2000	2000-52	568.5	275.5	Sandstone	1
2000	2000-60	529.7	314.3	Sandstone	0
2000	2000-64	508.8	335.2	Sandstone	7
2000	2000-76	448.9	395.1	Shale	3
2002	2002-16	750.6	86.4	Limestone	83

CP COL 2.5(1)

Table 2.5.4-212 (Sheet 2 of 2)
Summary of Calcium Carbonate Test Results

Boring No.	Sample No.	Elevation[(a)] (ft)	Depth (ft)	Material Type	Calcium Carbonate, $CaCO_3$ (%)
2002	2002-23	716.0	121.0	Limestone	89
2003	2003-06	798.5	42.5	Limestone	82
2004	2004-16	779.1	70.9	Limestone	74
2041A	2041A-09	801.5	30.5	Limestone	94

Statistical Summary (Mean ± Stdev [Count])	
Shale	36.9 ± 30.6 [7]
Limestone	84.9 ± 10.9 [17]
Sandstone	2.5 ± 3.1 [4]

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

CP COL 2.5(1)

Table 2.5.4-213 (Sheet 1 of 3) Summary of Petrographic and Photomicrographic Analysis

Boring No. or Location	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Description	Minerals (%)						
					Quartz	Feldspar	Calcite	Dolomite	Clay	Opaques	Others
1012	1012-49	393.6	450.5	Sandstone	85		15				
1012	HS-1	828.0	16.0	Limestone			99			1	< 1
1012	HS-2	822.7	21.3	Limestone	5			88		5	2
1012	HS-3	816.2	27.8	Limestone	2		93			5	
1012	HS-4	799.5	44.5	Limestone	3		45	40	6	6	
1012	HS-5	790.9	53.1	Shale	20			12	58	10	
1012	HS-6	717.3	126.8	Limestone	4		92		2	2	
1012	HS-7	685.4	158.6	Limestone	2		90		2	6	
1012	HS-8	725.8	118.2	Limestone	5		45	45	1	4	< 1
1012	HS-9	637.4	206.7	Limestone	8		20	62		8	2
1012	HS-10	579.7	264.3	Sandstone	60				35	5	< 1
1012	HS-11	534.3	309.7	Sandstone	50				45	5	< 1
1012	HS-12	508.6	335.4	Shale	35			35	24	3	3
1012	HS-13	477.8	366.3	Shale	15				81	2	2
1012	HS-14	462.4	381.7	Sandstone	41	20		20	15	3	1
1012	HS-15	404.2	439.8	Sandstone	70	10		2	15		3
1012	HS-16	394.3	449.7	Sandstone	60	3		37			< 1
1012	HS-17	391.3	452.7	Sandstone	65	< 1		35			< 1

CP COL 2.5(1)

Table 2.5.4-213 (Sheet 2 of 3) Summary of Petrographic and Photomicrographic Analysis

					Minerals (%)						
Boring No. or Location	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Description	Quartz	Feldspar	Calcite	Dolomite	Clay	Opaques	Others
1012	HS-18	384.6	459.4	Shale	30		30	1	30		9
1012	HS-19	349.8	494.2	Sandstone	67	18			15		< 1
1012	HS-20	322.4	521.7	Shale	40				54	2	4
1012	HS-21	297.2	546.8	Shale	30				64	2	4
1031	1031-10	826.5	37.8	Limestone	10		63	20		6	1
1031	1031-16	795.5	68.7	Shale	20			15	55	10	
1032	1032-19	788.5	78.8	Shale	30			20	43	7	
1035	1035-12	806.6	49.5	Limestone	3		87		5	5	< 1
1037	1037-18	754.2	98.6	Limestone	3		78		15	3	1
1041	1041-27	716.3	129.0	Shale	10		2		81	7	
2002	2002-06	802.6	34.6	Shale	15		5		77	3	
2004	2004-07	828.0	22.5	Shale	26			35	35	4	
2004	2004-12	803.3	47.2	Limestone	15		75		5	4	1
2031	2031-15	791.1	62.0	Shale	50			30	13	5	2
2034	2034-09	810.1	29.0	Limestone	2		89		7	2	
2034	2034-23	745.2	93.9	Limestone	3		86		5	6	
2038	2038-09	786.1	50.9	Limestone	20		51	5	15	6	3
STA 34[b]	HS-22	[b]	NA[b]	Limestone			98			< 1	2

CP COL 2.5(1)

Table 2.5.4-213 (Sheet 3 of 3)
Summary of Petrographic and Photomicrographic Analysis

Boring No. or Location	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Description	Minerals (%) Quartz	Feldspar	Calcite	Dolomite	Clay	Opaques	Others
STA 38[c]	HS-23	[c]	NA[c]	Limestone	3		79	15		3	
STA 42-A[d]	HS-24	[d]	NA[d]	Sandstone	80	10			10		< 1
STA 42-A[d]	HS-25	[d]	NA[d]	Sandstone	75	10					15
				Statistical Summary (Mean ± Stdev [Count])							
				Shale	26.8±11.5[12]	--	3.1±8.6[12]	12.3±14.4[12]	51.3±22.7[12]	4.6±3.3[12]	2.0±2.7[12]
				Limestone	5.2±5.3[17]	--	70.0±28.7[17]	16.2±26.7[17]	3.7±4.9[17]	4.2±2.1[17]	0.7±1.0[17]
				Sandstone	65.3±13.4[10]	7.1±7.7[10]	1.5±4.7[10]	9.4±15.3[10]	13.5±15.6[10]	1.3±2.2[10]	1.9±4.7[10]

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

b) STA 34: N32.29319, W097.80386; El. 800+/-13.2 ft (Garmin GPSmap 60X, Coordinate System: NAD27 CONUS)

c) STA 38: N32.29676, W097.79382; El. 813+/-13.9 ft (Garmin GPSmap 60X, Coordinate System: NAD27 CONUS)

d) STA 42-A: N32.30403, W097.84562; El. 1001+/-17.8 ft (Garmin GPSmap 60X, Coordinate System: NAD27 CONUS)

CP COL 2.5(1)

Table 2.5.4-214 (Sheet 1 of 2)
Summary of X-Ray Diffraction Analysis

Results of X-Ray Analysis on Clay-Size Fraction (< 2 μm) of Samples

					Minerals (%)				
Boring No.	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	Expandable Illite / Smectite	Expandable Smectite	Illite	Chlorite	Kaolinite
1012	1012-49	393.6	450.5	Sandstone	5		40	10	45
1031	1031-10	826.5	37.8	Limestone	10		80		10
1031	1031-16	795.5	68.7	Shale	5		60	5	30
1032	1032-19	788.5	78.8	Shale	10		75		15
1035	1035-12	806.6	49.5	Limestone	10		85		5
1037	1037-18	754.2	98.6	Limestone	15		75		10
1041	1041-27	716.3	129.0	Shale		30	55		15
2002	2002-06R	802.6	34.6	Shale	10		80		10
2004	2004-07	828.0	22.5	Shale	10		85		5
2004	2004-12	803.3	47.2	Limestone	10		85		5
2031	2031-15	791.1	62.0	Shale	5		70	5	20
2034	2034-09	810.1	29.0	Limestone	10		90		
2034	2034-23	745.2	93.9	Limestone	10		80		10
2038	2038-09	786.1	50.9	Limestone	5		80		15

Statistical Summary (Mean ± Stdev [Count])

Shale	6.7±4.1[6]	5.0±12.3[6]	70.1±11.6[6]	1.7±2.6[6]	15.8±8.6[6]
Limestone	10.0±2.9[7]	--	82.1±4.9[7]	--	7.9±4.9[7]
Sandstone	--	--	--	--	--

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

CP COL 2.5(1)

Table 2.5.4-214 (Sheet 2 of 2)
Summary of X-Ray Diffraction Analysis

Results of X-Ray Analysis on Bulk Samples

Boring No.	Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	Minerals (%) Quartz	Calcite	Ankerite	Dolomite	Phyllo-Silicates	Pyrite	Sanidine	Albite
1012	1012-49	393.6	450.5	Sandstone	65	30			5			
1031	1031-10	826.5	37.8	Limestone	5	74	13		8			
1031	1031-16	795.5	68.7	Shale	42	3	6		44	2	1	2
1032	1032-19	788.5	78.8	Shale	36		10		47		4	4
1035	1035-12	806.6	49.5	Limestone	3	92			5			
1037	1037-18	754.2	98.6	Limestone	9	85			6			
1041	1041-27	716.3	129.0	Shale	19				76		2	2
2002	2002-06R	802.6	34.6	Shale	18				82			
2004	2004-07	828.0	22.5	Shale	38	1	16		40			
2004	2004-12	803.3	47.2	Limestone	5	89			6			
2031	2031-15	791.1	62.0	Shale	65	1		26	8			
2034	2034-09	810.1	29.0	Limestone	2	92			5			
2034	2034-23	745.2	93.9	Limestone	4	83	4		9			
2038	2038-09	786.1	50.9	Limestone	21	67	3		8	1		

Statistical Summary (Mean ± Stdev [Count])

Shale	36.3±17.3[6]	0.8±1.2[6]	5.3±6.6[6]	4.3±10.6[6]	49.5±26.9[6]	0.3±0.8[6]	1.2±1.6[6]	1.3±1.6[6]
Limestone	7.0±6.6[7]	83.1±9.5[7]	2.9±4.8[7]	--	6.7±1.6[7]	0.14±0.4[7]	----	----
Sandstone	--	--	--	--	--	--	----	----

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

CP COL 2.5(1)

Table 2.5.4-215 (Sheet 1 of 2) Summary of Consolidated-Undrained Triaxial Test with Pore Water Pressure Measurement Results

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Approx. Overbdn. Pressure (a) σ_{vo} (tsf)	Consol. Pressure σ_3 (tsf)	Deviator Stress at Failure σ_1-σ_3 (tsf)	Axial Failure Strain ε_{af} (%)	Pore Pressure at Failure (tsf)	Failure Plane Stresses (for C=0): φ for C = 0 (deg)	Failure Plane Stresses (for C=0): Total Normal Stress σ_n (tsf)	Failure Plane Stresses (for C=0): Shear Strength s (tsf)	p'-q Parameters at Failure(b): q (tsf)	p'-q Parameters at Failure(b): P' (tsf)	Linear Shear Strength Parameters, Effective Stress: C' (tsf)	Linear Shear Strength Parameters, Effective Stress: φ' (deg)	Linear Shear Strength Parameters, Total Stress: C (tsf)	Linear Shear Strength Parameters, Total Stress: φ (deg)
Peak Strength																	
1002-30(c)	710.4	134.6	Shale	9.2	5.0	25.0	0.7	4.38	46	8.6	8.8	12.5	13.1	0.0	73	0.0	51
					10.1	70.6	1.1	8.46	51	18.0	22.2	35.3	36.9				
1029-13(c)	795.2	51.7	Shale	3.5	2.0	2.4	1.3	1.43	22	2.7	1.1	1.2	1.8	0.38	30	0.33	17
					4.0	4.8	1.2	2.65	22	5.5	2.2	2.4	3.7				
					8.0	7.2	1.2	5.11	18	10.5	3.4	3.6	6.5				
1038-16	785.4	58.1	Shale	4.0	4.2	19.0	1.9	0.44	49	7.1	6.8	9.5	13.2	--	--	--	--
2000-07	794.2	49.8	Shale	3.4	3.6	17.4	2.3	0.00	45	6.1	6.1	8.7	12.3	--	--	--	--
2003-35(c)	614.4	226.6	Sandstone	15.4	9.0	70.2	1.9	-0.32	53	16.2	21.2	35.1	44.4	21.5	20	22.9	18
					18.0	95.0	1.8	-0.49	47	31.0	32.7	47.5	66.0				
					32.4	91.2	1.7	1.99	36	51.3	37.0	45.6	76.0				
2003-36(c)	613.1	227.9	Sandstone	15.5	16.2	38.4	2.1	2.89	33	25.0	16.1	19.2	32.6	4.0	29	1.1	31
					32.4	74.0	4.4	-0.64	32	49.7	31.3	37.0	70.1				
Ultimate Strength																	
1029-13(c)	795.2	51.7	Shale	3.5	2.0	--	--	--	--	--	--	--	--	--	--	--	--
					4.0	--	--	--	--	--	--	--	--				
					8.0	7.4	9.0	4.17	15	10.0	2.7	2.7	6.5				

CP COL 2.5(1)

Table 2.5.4-215 (Sheet 2 of 2)
Summary of Consolidated-Undrained Triaxial Test with Pore Water Pressure Measurement Results

									Failure Plane Stresses (for C=0)			p'-q Parameters at Failure(b)		Linear Shear Strength Parameters			
														Effective Stress		Total Stress	
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Approx. Overbdn. Pressure (a) σ_{vo} (tsf)	Consol. Pressure σ_3 (tsf)	Deviator Stress at Failure σ_1-σ_3 (tsf)	Axial Failure Strain ε_{af} (%)	Pore Pressure at Failure (tsf)	φ for C = 0 (deg)	Total Normal Stress σ_n (tsf)	Shear Strength s (tsf)	q (tsf)	P' (tsf)	C' (tsf)	φ' (deg)	C (tsf)	φ (deg)
1038-16	785.4	58.1	Shale	4.0	4.2	13.8	8.7	0.74	38	6.8	5.4	6.9	10.4	--	--	--	--
2000-07	794.2	49.8	Shale	3.4	3.6	9.6	9.5	0.00	35	5.7	3.9	4.8	8.4	--	--	--	--

a) Approximate overburden pressure is calculated based on an average unit weight of 145 pcf.

b) $q=(\sigma_1-\sigma_3)/2$, $p'=(\sigma'_1+\sigma'_3)/2$.

c) One specimen was utilized for a multistage test.

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Table 2.5.4-216 (Sheet 1 of 3)
Summary of Consolidated-Undrained Triaxial Test without Pore Water Pressure Measurement Results

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Approx. Overbdn. Pressure[a] σ_{vo} (tsf)	Consol. Pressure σ_3 (tsf)	Deviator Stress at Failure σ_1-σ_3 (tsf)	Axial Failure Strain ε_{af} (%)	Failure Plane Stresses (for C=0): φ for C = 0 (deg)	Failure Plane Stresses (for C=0): Total Normal Stress σ_n (tsf)	Failure Plane Stresses (for C=0): Shear Strength s (tsf)	p-q Parameters at Failure[b]: q (tsf)	p-q Parameters at Failure[b]: P (tsf)
Peak Strength												
1002-15	783.7	61.0		4.4	4.4	18.6	2.5	43	7.4	6.8	9.3	13.7
1041-13	786.3	59.0	Shale[c]	4.3	8.5	38.8	1.8	44	14.4	13.9	19.4	27.9
1042-15d	784.5	61.9		4.5	2.3	12.8	2.1	48	3.9	4.3	6.4	8.6
1003-16	787.9	73.8		5.4	10.0	57.4	1.7	48	17.5	19.2	28.7	38.8
1006-13	792.1	55.6	Shale[c]	4.0	4.0	23.2	1.6	48	7.0	7.8	11.6	15.6
1006-14	791.6	56.1		4.1	2.0	24.4	2.1	59	3.8	6.3	12.2	14.3
1038-14	797.3	46.2		3.4	7.0	10.2	2.2	25	9.9	4.6	5.1	12.1
1042-15a	795.5	50.9	Shale[c]	3.7	2.0	34.8	1.6	64	3.8	7.7	17.4	19.4
2042a-12	794.1	36.6		2.7	2.7	12.2	2.8	44	4.5	4.4	6.1	8.8
1000-17A	779.0	72.0	Limestone	5.2	5.2	552.8	1.1	79	10.4	53.0	276.4	281.6
1000-24	741.6	109.4	Limestone	7.9	7.9	587.2	0.6	77	15.7	66.9	293.6	301.6
1000-27	728.2	122.8	Limestone	8.9	8.9	285.0	0.9	70	17.3	48.1	142.5	151.4
1000-51	475.3	375.7	Sandstone	27.2	27.2	124.0	0.9	44	46.2	44.5	62.0	89.4
1003-09	821.8	39.9	Shale	2.9	2.9	15.2	0.9	46	5.0	5.2	7.6	10.5
1003-14	796.3	65.4	Shale	4.7	4.7	25.2	0.9	47	8.2	8.7	12.6	17.3
1003-off-16	799.6	62.1	Shale	4.5	4.5	34.6	1.5	53	8.1	10.5	17.3	21.8
1003-off-24	760.7	101.0	Limestone	7.3	7.3	126.8	0.8	64	13.9	28.1	63.4	70.8
1004-33	715.6	144.1	Shale	10.5	5.2	10.8	11.1	31	7.9	4.7	5.4	10.6
1004-46	650.7	209.0	Shale	15.2	15.2	41.6	0.4	35	24.0	16.9	20.8	36.1
1004-51	599.2	260.5	Shale	18.9	18.9	35.6	0.7	29	28.1	15.6	17.8	36.7

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Table 2.5.4-216 (Sheet 2 of 3) Summary of Consolidated-Undrained Triaxial Test without Pore Water Pressure Measurement Results

								Failure Plane Stresses (for C=0)			p-q Parameters at Failure[b]	
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Approx. Overbdn. Pressure[a] σ_{vo} (tsf)	Consol. Pressure σ_3 (tsf)	Deviator Stress at Failure σ_1-σ_3 (tsf)	Axial Failure Strain ε_{af} (%)	φ for C = 0 (deg)	Total Normal Stress σ_n (tsf)	Shear Strength s (tsf)	q (tsf)	P (tsf)
1012-44	506.0	338.1	Shale	24.5	24.5	82.2	0.6	39	39.9	32.0	41.1	65.6
1013-15	792.9	70.6	Shale	5.1	5.1	26.2	1.7	46	8.8	9.1	13.1	18.2
1041-12	795.0	50.3	Shale	3.7	3.6	14.8	4.3	42	6.1	5.5	7.4	11.0
2000-17A	742.7	101.3	Limestone	7.3	7.3	536.0	0.5	77	14.5	61.5	268.0	275.4
2000-27	691.8	152.2	Limestone	11.0	11.1	240.0	1.4	66	21.2	48.2	120.0	131.0
2000-72	466.6	377.4	Shale	27.4	27.4	23.4	1.8	17	35.6	11.1	11.7	39.2
2002-16R	750.8	86.4	Limestone	6.3	6.3	200.0	1.0	70	12.2	33.8	100.0	106.2
2002-22R	720.4	116.8	Shale	8.5	8.4	70.6	2.6	54	15.2	20.8	35.3	43.7
2006-14	784.0	61.0	Shale	4.4	4.4	50.4	1.6	58	8.2	13.3	25.2	29.6
2008-09	792.9	47.7	Shale	3.5	3.5	15.8	2.9	44	5.9	5.7	7.9	11.4
2030-07	807.5	45.1	Shale	3.3	3.3	26.4	1.8	53	5.9	7.9	13.2	16.5
2036-04	818.2	14.2	Shale	1.0	2.1	11.6	2.8	47	3.6	3.9	5.8	7.8
2037-13	787.5	49.0	Shale	3.6	3.6	11.8	2.7	39	5.8	4.6	5.9	9.4
2042a-14a	779.6	51.1	Shale	3.7	3.7	10.2	3.1	35	5.9	4.2	5.1	8.8
Ultimate Strength												
1041-13	786.3	59.0	Shale[c]	4.3	8.5	28.9	5.2	39	13.8	11.2	14.4	22.9
1042-15d	784.5	61.9		4.5	2.3	7.7	4.5	39	3.7	2.9	3.8	6.1
1003-16	787.9	73.8	Shale[c]	5.4	10.0	31.0	7.2	37	16.1	12.3	15.5	25.6
1006-13	792.1	55.6		4.0	4.0	13.7	3.2	39	6.6	5.3	6.8	10.9
1006-14	791.6	56.1		4.1	2.0	11.8	3.5	49	3.5	4.0	6.0	8.0

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Table 2.5.4-216 (Sheet 3 of 3)
Summary of Consolidated-Undrained Triaxial Test without Pore Water Pressure Measurement Results

								Failure Plane Stresses (for C=0)			p-q Parameters at Failure[b]	
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Approx. Overbdn. Pressure[a] σ_{vo} (tsf)	Consol. Pressure σ_3 (tsf)	Deviator Stress at Failure σ_1-σ_3 (tsf)	Axial Failure Strain ε_{af} (%)	φ for C = 0 (deg)	Total Normal Stress σ_n (tsf)	Shear Strength s (tsf)	q (tsf)	P (tsf)
1038-14	797.3	46.2		3.4	7.0	6.5	7.7	19	9.2	3.1	3.3	10.3
1042-15a	795.5	50.9	Shale[c]	3.7	2.0	7.4	3.4	41	3.3	2.8	3.7	5.7
2042a-12	794.1	36.6		2.7	2.7	7.6	5.3	35	4.2	3.0	3.7	6.4
1003-off-16	799.6	62.1	Shale	4.5	4.5	21.4	3.2	45	7.7	7.6	10.7	15.2
1004-46	650.7	209.0	Shale	15.2	15.2	32.4	3.2	31	22.9	13.7	15.9	31.1
1004-51	599.2	260.5	Shale	18.9	18.9	16.9	2.8	18	24.6	7.8	8.2	27.1
1013-15	792.9	70.6	Shale	5.1	5.1	14.3	4.3	36	8.1	5.8	7.1	12.2
2006-14	784.0	61.0	Shale	4.4	4.4	15.3	10.7	39	7.2	5.8	7.4	11.8

a) Approximate overburden pressure is calculated based on an average unit weight of 145 pcf.

b) $q=(\sigma_1-\sigma_3)/2$, $p=(\sigma_1+\sigma_3)/2$.

c) Three separate specimens from about the same elevation range were used for a 3 point test.

CP COL 2.5(1)

Table 2.5.4-217 (Sheet 1 of 2)
Summary of Unconsolidated-Undrained Triaxial Test Results

Sample No.	Elevation (ft)	Depth (ft)	Material Type	L/D Ratio	Confining[a] Pressure, σ_3 (tsf)	Axial Stress at Failure, σ_1 (tsf)	Deviator Stress at Failure, σ_1-σ_3 (tsf)	Axial Failure Strain, ε_{af} (%)	Failure Plane Stresses (for C=0) Normal Stress, σ_n (tsf)	Failure Plane Stresses (for C=0) Shear Strength, s (tsf)	$q = \frac{\sigma_1 - \sigma_3}{2}$ (tsf)	$p = \frac{\sigma_1 + \sigma_3}{2}$ (tsf)
Peak Strength												
1000-46	568.7	282.3	Sandstone	1.9	20.5	70.5	50.0	1.5	31.8	20.9	25.0	45.5
1002-09	812.8	31.9	Limestone	2.1	2.3	206.6	204.3	0.7	4.5	21.3	102.1	104.4
1003-15	793.4	68.3	Shale	2.0	4.9	46.0	41.1	1.6	8.9	12.1	20.5	25.5
1004-41	674.2	185.5	Limestone	2.0	13.5	409.7	396.2	1.0	26.2	69.6	198.1	211.6
1005-15	797.5	59.5	Shale	1.8	4.3	26.0	21.7	2.5	7.4	7.6	10.9	15.2
1005-20	772.5	84.5	Limestone	2.2	6.1	501.1	495.0	0.9	12.1	54.0	247.5	253.6
1032-12	824.0	43.3	Limestone	1.7	3.1	569.5[b]	566.4	1.1	3.1	20.9	284.8	287.9
2000-5	802.3	41.7	Shale	2.0	3.0	7.2	4.2	13.8	4.2	1.9	2.1	5.1
2005-08	816.7	32.3	Limestone	2.1	2.4	500.1	497.7	1.2	4.7	34.3	249.0	251.3
2036-06	809.2	23.8	Limestone	2.4	2.1	220.8	218.7	0.7	4.1	21.1	109.4	111.5
2036-12	775.9	57.1	Shale	2.5	4.1	17.3	13.2	1.7	6.6	5.2	6.6	10.7

CP COL 2.5(1)

Table 2.5.4-217 (Sheet 2 of 2)
Summary of Unconsolidated-Undrained Triaxial Test Results

Sample No.	Elevation (ft)	Depth (ft)	Material Type	L/D Ratio	Confining[a] Pressure, σ_3 (tsf)	Axial Stress at Failure, σ_1 (tsf)	Deviator Stress at Failure, σ_1-σ_3 (tsf)	Axial Failure Strain, ε_{af} (%)	Failure Plane Stresses (for C=0): Normal Stress, σ_n (tsf)	Failure Plane Stresses (for C=0): Shear Strength, s (tsf)	$q = \frac{\sigma_1 - \sigma_3}{2}$ (tsf)	$p = \frac{\sigma_1 + \sigma_3}{2}$ (tsf)
Ultimate Strength												
1000-46	568.7	282.3	Sandstone	1.9	20.5	29.6	9.1	4.9	24.2	4.5	4.6	25.1
1003-15	793.4	68.3	Shale	2.0	4.9	28.8	23.9	3.8	8.3	8.7	12.8	17.7
1005-15	797.5	59.5	Shale	1.8	4.3	20.0	15.7	4.7	7.1	6.0	7.8	12.1
2036-12	775.9	57.1	Shale	2.5	4.1	11.0	6.9	10.2	6.0	3.0	3.4	7.5

a) Confining pressure values are approximately the same as effective overburden pressure values.

b) Sample did not fail.

CP COL 2.5(1)

Table 2.5.4-218 (Sheet 1 of 4)
Summary of Unconfined Compression Test Results

Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	L/D Ratio	Axial Failure Stress, σ_{af} (tsf)	Axial Failure Strain, ε_{af} (LVDT Jacket) (%)	Axial Failure Strain, ε_{apf} (Actuator) (%)	Secant Young's Modulus at 50% σ_{af} (psi)	Tangent Young's Modulus at 50% σ_{af} (psi)	Secant Poisson's Ratio at 50% σ_{af}	Tangent Poisson's Ratio at 50% σ_{af}
1000-39	639.0	212.0	Shale	1.7	104	--	0.57	--	--	--	--
1003-off-22	771.6	90.1	Limestone	2.0	312	--	0.91	-	--	--	--
1003-off-30	731.1	130.6	Limestone	1.7	200	--	0.65	--	--	--	--
1004-25	756.0	104.0	Limestone	1.98	218	--	0.75	--	--	--	--
1004-38	684.4	175.6	Limestone	1.9	268	--	0.69	--	--	--	--
1005-09	828.0	29.0	Limestone	2.3	371	0.085	0.38	7.2×10^6	7.0×10^6	0.22	0.42
1005-12	812.2	44.8	Limestone	2.2	221	--	0.68	--	--	--	--
1009-I-05	816.7	40.5	Limestone	2.1	255	--	0.96	--	--	--	--
1012-17	755.0	89.0	Limestone	2.0	137	--	0.96	--	--	--	--
1012-22	730.0	114.0	Limestone	2.2	291	0.32	0.37	1.6×10^6	1.2×10^6	0.02	0.05
1012-28	700.0	144.0	Limestone	1.9	311	--	0.83	--	--	--	--
1012-33	660.0	184.0	Limestone	2.2	242	0.11[b]	0.50	5.0×10^6	4.7×10^6	0.00	0.00
1012-40	575.0	269.0	Sandstone	1.9	10	--	2.40	--	--	--	--
1013 @ 55.5'	808.0	55.5	Limestone	2.2	199	0.43	0.77	6.9×10^5	6.6×10^5	0.00	0.02
1013-12	805.3	58.2	Limestone	2.0	372	0.28	0.62	1.8×10^6	2.4×10^6	0.12	0.26
1013-19	771.7	91.8	Limestone	2.2	296	0.52	0.64	7.9×10^5	9.9×10^5	0.09	0.17
1013-23	805.3	110.1	Limestone	1.9	147	--	1.05	--	--	--	--
1014-17	777.5	73.7	Limestone	2.3	271	0.48	0.53	8.6×10^5	8.5×10^5	0.17	0.23
1014-21	756.9	94.3	Limestone	2.3	595	0.24	0.43	4.0×10^6	3.9×10^6	0.12	0.27
1029-09	816.6	30.3	Limestone	2.1	323	--	0.58	--	--	--	--

CP COL 2.5(1)

Table 2.5.4-218 (Sheet 2 of 4) Summary of Unconfined Compression Test Results

Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	L/D Ratio	Axial Failure Stress, σ_{af} (tsf)	Axial Failure Strain, ε_{af} (LVDT Jacket) (%)	Axial Failure Strain, ε_{apf} (Actuator) (%)	Secant Young's Modulus at 50% σ_{af} (psi)	Tangent Young's Modulus at 50% σ_{af} (psi)	Secant Poisson's Ratio at 50% σ_{af}	Tangent Poisson's Ratio at 50% σ_{af}
1029-18	771.1	75.8	Limestone	2.0	163	0.48	0.76	6.2×10^5	5.1×10^5	0.18	0.24
1030-11	826.6	32.0	Limestone	2.0	513[c]	--	0.53	--	--	--	--
1030-22	771.0	87.6	Limestone	2.1	251	0.30	0.57	1.4×10^6	1.4×10^6	--	0.01
1031-21	770.8	93.4	Limestone	2.2	323	--	1.01	--	--	--	--
1032-22	773.6	93.7	Limestone	2.3	361	0.25	0.63	3.5×10^6	1.8×10^6	0.04	0.15
1033-12	813.3	53.6	Limestone	2.1	288	0.35	1.10	1.7×10^6	1.4×10^6	0.06	0.10
1034-23	761.6	99.4	Limestone	1.9	493[c]	--	0.42	--	--	--	--
1035-07	831.5	24.6	Limestone	2.2	487[c]	--	0.44	--	--	--	--
1035-13	799.5	56.6	Shale	2.2	13	1.00	1.15	4.4×10^4	3.5×10^4	0.35	0.50
1037-05	818.2	34.6	Shale	2.3	92	0.41	0.87	4.9×10^5	2.6×10^5	0.00	0.00
1037-17	757.9	94.9	Shale	2.2	91	0.49	0.68	2.5×10^5	3.1×10^5	0.10	0.11
1038-17	781.1	62.4	Limestone	2.2	195	0.40	0.52	5.6×10^5	1.0×10^6	0.07	0.24
1038-24	745.7	97.8	Limestone	2.3	309	0.20	0.57	3.8×10^6	3.1×10^6	0.50	0.50
1041-09	807.0	38.0	Limestone	1.8	156	--	1.18	--	--	--	--
1041-17	766.8	78.2	Limestone	1.9	190	--	0.51	--	--	--	--
1041-20	752.0	93.0	Limestone	1.8	227	--	0.86	--	--	--	--
1042-19	765.6	80.9	Limestone	2.1	357	0.32	0.68	1.8×10^6	1.5×10^6	0.07	0.28
1042-21	757.5	88.9	Limestone	2.1	157	--	0.85	--	--	--	--
2000-13	763.7	80.3	Limestone	2.1	253	--	0.93	--	--	--	--

CP COL 2.5(1)

Table 2.5.4-218 (Sheet 3 of 4)
Summary of Unconfined Compression Test Results

Sample No.	Elevation[a] (ft)	Depth (ft)	Material Type	L/D Ratio	Axial Failure Stress, σ_{af} (tsf)	Axial Failure Strain, ε_{af} (LVDT Jacket) (%)	Axial Failure Strain, ε_{apf} (Actuator) (%)	Secant Young's Modulus at 50% σ_{af} (psi)	Tangent Young's Modulus at 50% σ_{af} (psi)	Secant Poisson's Ratio at 50% σ_{af}	Tangent Poisson's Ratio at 50% σ_{af}
2000-13A	763.7	80.3	Limestone	2.3	366	0.76	0.76	7.4×10^5	6.7×10^5	0.07	0.13
2000-17	742.1	101.9	Limestone	2.1	250	--	0.48	--	--	--	--
2000-18	737.8	106.2	Limestone	1.7	515[c]	--	0.54	--	--	--	--
2000-20	726.5	117.5	Limestone	1.9	498[c]	--	0.35	--	--	--	--
2000-26	699.0	145.0	Limestone	1.7	287	--	0.62	--	--	--	--
2001-10	772.1	65.8	Limestone	2.1	351	--	0.87	--	--	--	--
2002-31R	675.3	161.9	Limestone	2.3	295	0.16	0.32	2.5×10^6	2.6×10^6	0.02	0.07
2004-16	779.6	70.9	Limestone	2.2	467	--	0.74	--	--	--	--
2005-21	751.6	97.1	Limestone	2.2	73	--	1.73	--	--	--	--
2006-16	774.5	70.5	Limestone	2.3	376	--	0.88	--	--	--	--
2012 @ 69'	778.6	69.0	Limestone	2.1	433	0.23[b]	0.34	2.6×10^6	2.2×10^6	0.07	0.08
2012-20	778.1	69.5	Limestone	2.3	812[c]	0.15	0.23	8.0×10^6	6.9×10^6	0.05	0.15
2014 @ 43.6'	797.1	43.6	Limestone	2.3	222	0.30	0.48	1.7×10^6	8.7×10^5	0.13	0.11
2014-8	796.7	44.0	Limestone	2.2	790[c]	--	0.19	--	--	--	--
2014-16	752.4	87.0	Limestone	2.2	87	0.46[b]	0.70	2.7×10^5	3.0×10^5	0.08	0.22
2030-11	789.2	63.4	Limestone	2.2	148	--	0.65	--	--	--	--
2030-17	759.1	93.5	Limestone	2.1	118	--	0.92	--	--	--	--
2031-11	810.2	42.9	Limestone	2.1	242	0.36	0.56	1.2×10^6	1.1×10^6	0.19	0.26
2035-21	739.5	90.3	Limestone	2.1	206	--	1.04	--	--	--	--
2035-22	735.9	93.9	Limestone	2.3	268	--	0.74	--	--	--	--

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Table 2.5.4-218 (Sheet 4 of 4) Summary of Unconfined Compression Test Results

Sample No.	Elevation[(a)] (ft)	Depth (ft)	Material Type	L/D Ratio	Axial Failure Stress, σ_{af} (tsf)	Axial Failure Strain, ε_{af} (LVDT Jacket) (%)	Axial Failure Strain, ε_{apf} (Actuator) (%)	Secant Young's Modulus at 50% σ_{af} (psi)	Tangent Young's Modulus at 50% σ_{af} (psi)	Secant Poisson's Ratio at 5 0% σ_{af}	Tangent Poisson's Ratio at 50% σ_{af}
2036-13	774.6	57.8	Limestone	2.1	365	0.37	0.67	1.8×10^6	1.6×10^6	0.21	0.36
2038-13	767.1	69.9	Limestone	2.1	203	0.53	0.64	7.9×10^5	5.7×10^5	0.22	0.27
2041a-15	768.7	63.3	Limestone	1.7	83	--	0.90	--	--	--	--
2042a-15	779.1	51.9	Limestone	1.5	292	--	0.63	--	--	--	--
2042a-17	767.8	62.9	Limestone	2.3	157	--	0.85	--	--	--	--

a) Coordinate System: US State Plane 1983
Zone: Texas North Central 4202
Vertical Datum: NAVD88

b) Only one LVDT was working.

c) Sample did not fail.

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Table 2.5.4-219 (Sheet 1 of 2)
Summary of Point Load Strength Index Test Results

				Diametral				Axial				
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Failure Load (lb)	PLI (psi)	Corrected PLI I_{S50} (tsf)	Estimated Uniaxial Comp. Strength q_{ud} (tsf)	Failure Load (lb)	PLI (psi)	Corrected PLI I_{S50} (tsf)	Estimated Uniaxial Comp. Strength q_{ua} (tsf)	Strength Correlation Factors[(a)]
1000-10	813.6	37.4	Limestone	1065	188.0	14.7	144	1495	359.2	26.3	257	10
1000-17	778.4	72.6	Limestone	1121	196.9	15.5	210	1670	332.5	25.3	344	14
1000-18	772.5	78.5	Limestone	948	164.8	13.0	176	460	88.6	6.8	92	14
1000-23	748.2	102.8	Limestone	91	33.0	2.2	30	54	9.5	0.7	10	14
1000-34	686.0	165.0	Limestone	95	17.0	1.3	15	751	67.0	6.1	73	12
1000-52	457.2	393.8	Sandstone	120	22.0	1.7	35	386	90.0	6.6	136	20
1002-20	757.7	87.3	Limestone	2154	345.3	27.7	375	2128	536.7	38.8	527	14
1003-11	812.8	49.2	Limestone	1613	260.7	20.9	204	2045	430.0	32.4	317	10
1003-off-35	701.6	160.1	Limestone	1276	207.0	16.6	198	1227	220.0	17.2	206	12
1004-53	579.7	280.0	Sandstone	--	--	--	--	26	4.2	0.3	7	20
1006-10	808.2	39.8	Limestone	229	38.0	3.0	30	757	187.8	13.6	133	10
1008-17	774.3	69.7	Limestone	709	118.5	9.4	128	1611	284.8	22.3	303	14
1010-09	828.9	33.1	Limestone	--	--	--	--	2460	520.7	39.2	383	10
1010-22	826.0	35.4	Limestone	2702	471.8	37.1	363	3537	473.5	39.5	386	10
1032-15	806.5	60.5	Limestone	1627	270.1	21.5	210	3186	944.7	65.9	644	10
1033-09	828.0	39.0	Limestone	1723	285.4	22.7	222	2008	352.6	27.7	271	10
1037-14	775.7	78.3	Limestone	398	64.8	5.2	70	487	106.7	8.0	108	14
1038-13	798.7	44.3	Limestone	719	125.6	9.9	97	1457	389.4	27.8	272	10
1041-16	772.7	72.6	Limestone	417	67.7	5.4	73	330	51.0	4.1	56	14
1042-09	817.8	28.2	Limestone	1841	304.9	24.3	237	1738	325.4	25.2	246	10
1042-12	802.3	43.7	Limestone	109	17.6	1.4	14	--	--	--	--	10
2000-29	684.0	160.0	Limestone	666	107.0	8.6	102	1129	221.0	16.9	202	12
2000-31	676.1	167.9	Limestone	1488	242.0	19.2	230	1049	274.0	19.7	235	12
2000-33	663.6	180.4	Limestone	1801	289.3	23.2	277	2337	734.9	50.6	605	12
2000-34	657.8	186.2	Limestone	1798	290.9	23.3	278	1386	256.0	19.9	238	12
2001-09	779.3	58.7	Limestone	240	41.9	3.3	45	1077	269.9	19.6	265	14
2002-13	768.4	68.8	Limestone	1417	226.5	18.2	246	2058	310.5	25.2	342	14

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Table 2.5.4-219 (Sheet 2 of 2)
Summary of Point Load Strength Index Test Results

				Diametral				Axial				
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Failure Load (lb)	PLI (psi)	Corrected PLI I_{S50} (tsf)	Estimated Uniaxial Comp. Strength q_{ud} (tsf)	Failure Load (lb)	PLI (psi)	Corrected PLI I_{S50} (tsf)	Estimated Uniaxial Comp. Strength q_{ua} (tsf)	Strength Correlation Factors[(a)]
2006-19	759.8	85.2	Limestone	977	156.6	12.5	170	1124	227.5	17.3	235	14
2007-15	780.1	61.0	Limestone	3364	547.5	43.7	523	2886	478.8	38.1	455	8
2007-19	756.5	84.8	Limestone	355	57.8	4.6	63	410	66.8	5.3	72	14
2030-15	768.0	84.6	Limestone	754	131.7	10.4	140	1024	172.6	13.7	186	14
2031-10	816.7	36.3	Limestone	1308	210.2	16.8	165	2685	487.4	38.0	371	10
2031-19	770.1	83.0	Limestone	1745	281.7	22.5	306	1388	228.1	18.2	247	14
2033-19	777.1	72.9	Limestone	701	220.9	15.2	206	1144	217.9	16.8	228	14
2034-20	759.4	79.6	Limestone	1121	197.6	15.5	210	2393	557.5	41.1	557	14
2034-22	750.9	88.1	Limestone	780	138.2	10.8	147	1394	391.5	27.7	375	14
2035-14	774.0	55.8	Limestone	441	72.8	5.8	79	1319	204.8	16.5	224	14
2036-14	768.5	64.5	Limestone	809	214.2	15.3	208	516	119.8	8.8	120	14
2037-15	777.1	59.4	Limestone	1646	283.6	22.4	303	2006	294.0	24.0	326	14
2037-19	757.0	79.5	Limestone	816	141.8	11.2	151	1388	190.7	15.8	215	14
2041a-09	801.5	30.5	Limestone	1301	208.1	16.7	163	2307	353.1	28.6	280	10
2041a-20	745.4	86.7	Limestone	1660	267.3	21.4	290	2282	817.3	54.7	742	14
2042a-23	738.1	92.6	Limestone	462	81.2	6.4	86	1359	213.0	17.2	233	14
			Statistical Summary (Mean ± Stdev [Count])									
			Limestone	1112±726[40]	190±118[40]	15±9.4[40]	180±109[40]	1552±809[40]	319±204[40]	24±14.3[40]	284±163[40]	12.4±1.9[41]
			Sandstone	120± - [1]	22± - [1]	1.7± - [1]	35± - [1]	206± - [2]	47± - [2]	3.5± - [2]	72± - [2]	20± - [2]

a) Strength Correlation Factor was used to correlate both, diametral and axial unconfined compressive strength.

CP COL 2.5(1)

Table 2.5.4-220
Summary of Strength Properties with Statistical Data

Material Type / Geologic Layer	Primary Lithology	Compressive Strength[a] (tsf) Mean ± Stdev [Count]						
			Point Load Index Test		CU Triaxial Compression Test		UU Triaxial Compression Test	
		Unconfined Compression Test	Axial	Diametral	Peak	Ultimate	Peak	Ultimate
Shale	Shale	75 ± 36 [4]	--	-	27 ± 20 [33]	15 ± 8 [16]	20 ± 14 [4]	16 ± 7 [3]
Limestone	Limestone	299 ± 149 [59]	285 ± 161 [40]	180 ± 108 [40]	361 ± 177 [7]	--	396 ± 140 [6]	--
Sandstone	Sandstone	10 ± -- [1]	71 ± 64 [2]	35 ± -- [1]	82 ± 26 [6]	--	50 ± -- [1]	9 ± -- [1]
Layer A	Limestone	281 ± 129 [11]	324 ± 123 [11]	168 ± 94 [11]	18 ± 6 [3]	--	298 ± 206 [5]	--
Layer B	Shale	291 ± 229 [7]	455 ± -- [1]	523 ± -- [1]	22 ± 14 [22]	14 ± 8 [14]	31 ± 10 [2]	20 ± 4 [2]
Layer C	Limestone	290 ± 153 [39]	264 ± 171 [22]	169 ± 93 [22]	337 ± 202 [7]	--	254 ± 241 [2]	7 ± -- [1]
Layer D	Shale	--	--	--	--	--	--	--
Layer E	Limestone	251 ± 69 [6]	260 ± 164 [6]	184 ± 96 [6]	94 ± 86 [4]	32 ± -- [1]	396 ± -- [1]	--
Layer F	Limestone with Shale and Sand interbeds	--	--	--	67 ± 23 [7]	17 ± -- [1]	--	--
Layer G	Sandstone	10 ± -- [1]	7 ± -- [1]	--	--	-	50 ± -- [1]	9 ± -- [1]
Layer H	Shale	-	136 ± -- [1]	35 ± -- [1]	77 ± 41 [3]	--	--	
Layer I	Sandstone	-	-	-	--	--	--	

a) The compressive strength is in terms of uniaxial stress for unconfined compression and point load index tests, and in terms of deviator stress for CU and UU tests.

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Table 2.5.4-221 (Sheet 1 of 2) Summary of Direct Shear Test Results

					Peak	Ultimate	Ultimate	
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Effective Normal Stress (tsf)	Effective Shear Stress (tsf)	Effective Shear Stress (tsf)	Effective Cohesion (tsf)	Effective Friction Angle (degree)
1042-15b	787.1	59.4	Shale	2.2	8.3	2.6	1.40	29
				4.3	7.8	4.1		
				8.7	15.0	-		
1043-07	793.2	31.3	Shale	1.3	3.5	2.5	1.10	33
				2.6	2.7	2.3		
				5.2	4.5	4.5		
1048-14	783.3	52.5	Shale	1.7	1.4	1.2	0.80	14
				3.4	1.8	1.7		
				6.8	2.6	2.5		
1058-12	796.8	48.4	Shale	1.8	5.1	1.3	0.20	34
				3.5	2.9	2.7		
				7.0	4.9	4.9		

CP COL 2.5(1)

Table 2.5.4-221 (Sheet 2 of 2) Summary of Direct Shear Test Results

					Peak	Ultimate	Ultimate	
Sample No.	Elevation (ft)	Depth (ft)	Material Type	Effective Normal Stress (tsf)	Effective Shear Stress (tsf)	Effective Shear Stress (tsf)	Effective Cohesion (tsf)	Effective Friction Angle (degree)
2000-8	815.2	28.8	Shale	1.1	1.9	0.5	0.24	15
				2.1	0.9	0.8		
				4.2	1.3	1.3		
2001C-2	809.0	28.9	Shale	1.1	1.3	0.6	0.10	26
				2.1	1.2	1.1		
				4.2	2.6	2.1		
2051-4	794.0	16.0	Shale	1.3	1.1	0.7	0.10	22
				2.6	1.0	0.9		
				5.2	2.2	2.1		

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Table 2.5.4-222
Summary of One Dimensional Consolidation Test Results

Sample No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type (USCS)	Moisture Content (%)	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plastic Index, PI
1000-15	791.6	63.0	Shale (CL)	13.7	40	16	24
1004-12	820.0	40.0	Shale (CL)	18.4	45	17	28
1005-18	783.1	54.0	Shale (CL)	12.4	46	17	29
2003-07	793.9	47.2	Shale (CL)	13.6	46	16	30
2006-07	819.6	25.4	Shale (CL)	17.5	43	17	26

Sample No.	Effective Overburden Pressure, σ'_o (tsf)	Preconsolidation Pressure, σ'_p (tsf)	Approximate Overconsolidation Ratio, OCR	Compression Ratio, $C_{c\varepsilon}$	Recompression Ratio, $C_{r\varepsilon}$	Coefficient of Consolidation, C_v (in^2/day)
1000-15	4.8	13.0	2.7	0.065	0.025	4 to 106
1004-12	3.0	5.5	1.8	0.070	0.040	2 to 30
1005-18	5.6	21.0	3.8	0.060	0.030	2 to 93
2003-07	3.5	7.0	2.0	0.045	0.025	4 to 98
2006-07	1.9	4.5	2.4	0.065	0.035	4 to 212

Notes:
1. $C_{c\varepsilon} = C_c/(1+e_o)$, and $C_{r\varepsilon} = C_r/(1+e_o)$ where C_c and C_r are compression index and recompression index, respectively.
2. Lower and upper value of coefficient of consolidation values (C_v) correspond to higher and lower loads, respectively.

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Table 2.5.4-223(Sheet 1 of 2)
Summary of Swell Test Results

Sample No.	Sample Elevation (ft)	Sample Depth (ft)	Material Type (USCS)	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI	Moisture Content (%)	
							Initial	Final
1002-10	809.3	35.4	Shale (CH)	69	24	45	13.4	15.3
1005-17	788.2	68.8	Shale (CL)	46	19	27	17.2	19.2
1033-07	838.2	28.8	Shale (CL)	27	13	14	9.9	12.1
1037-12	784.8	68.0	Shale (CH)	56	21	35	16.3	20.1
1041-11	796.3	49.0	Shale (CH)	56	23	33	18.1	20.9
2004-06	829.8	20.2	Shale (CL)	33	15	18	12.0	12.8
2036-07	806.7	26.3	Shale (CL)	48	22	26	16.7	18.0
2036-10	790.7	42.3	Shale (CL)	45	16	29	14.4	15.8
2042-11	793.9	37.1	Shale (CH)	55	23	32	20.3	23.5

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Table 2.5.4-224 (Sheet 2 of 2)
Summary of Swell Test Results

Sample No.	Degree of Saturation (%)		Effective Overburden Pressure, σ'_v (tsf)	Inundation Pressure (ksf)	Estimated Swell Pressure (ksf)	Estimated Heave[a] (%)
	Initial	Final				
1002-10	89.6	97.4	5.1	4.0	19	1.3
1005-17	93.1	99.7	10.0	8.0	13	0.7
1033-07	72.9	88.8	4.2	4.0	4	NA
1037-12	82.0	98.4	10.0	8.0	25	2.0
1041-11	91.3	98.3	7.1	6.0	11	1.0
2004-06	92.0	99.8	3.0	2.5	11	0.5
2036-07	91.5	95.6	3.8	3.0	32	2.2
2036-10	85.4	94.8	6.1	5.0	18	1.2
2042-11	84.5	96.6	5.1	4.5	12	0.8

a) Estimated heave at inundation pressure.

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Table 2.5.4-224 (Sheet 1 of 3) Summary of Laboratory-Based Shear Wave Velocity Measurements

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Shear Wave Velocity (fps)
1000-51	475.3	375.7	Sandstone	3,129
1002-09	812.8	31.9	Limestone	6,156
1002-29	714.3	130.4	Shale	8,153
1003-off-13	815.3	46.4	Limestone	11,143
1003-off-16	799.9	62.1	Shale	3,886
1004-13	814.2	45.5	Limestone	4,523
1004-33	715.6	144.1	Shale	2,325
1004-38	684.1	175.6	Limestone	6,238
1004-46	651.0	209.0	Shale	7,975
1004-54	569.7	290.0	Sandstone	2,166
1005-09	828.0	29.0	Limestone	9,223
1005-12	812.2	44.8	Limestone	6,476
1005-20	772.5	84.5	Limestone	7,892
1007-11	801.3	42.3	Limestone	7,824
1009-I-09	797.0	60.2	Limestone	6,699
1012-11	785.0	59.0	Shale	3,858
1012-22	730.0	114.0	Limestone	12,641
1012-27	705.0	139.0	Shale	6,700
1012-28	700.0	144.0	Limestone	7,840
1012-33	660.0	184.0	Limestone	14,383
1012-37	615.0	229.0	Sandstone	2,114
1013 @ 55.5'	808.0	55.5	Limestone	5,785
1013-12	805.3	58.2	Limestone	7,661
1013-19	772.1	91.4	Limestone	6,879
1014-17	777.5	73.7	Limestone	6,643
1014-21	756.9	94.3	Limestone	8,591
1030-11	826.6	32.0	Limestone	8,988
1030-13	815.6	43.0	Limestone	6,955
1030-22	771.0	87.6	Limestone	6,264
1031-14	803.8	60.4	Limestone	6,954
1032-12	824.0	43.3	Limestone	9,522
1032-22	773.6	93.7	Limestone	7,019
1033-12	813.3	53.6	Limestone	7,226
1034-10	827.8	33.2	Limestone	6,141
1034-18	785.1	73.4	Limestone	6,711

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Table 2.5.4-224 (Sheet 2 of 3)
Summary of Laboratory-Based Shear Wave Velocity Measurements

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Shear Wave Velocity (fps)
1034-21	768.9	89.6	Limestone	10,819
1035-07	831.5	24.6	Limestone	4,813
1035-17	781.7	74.4	Limestone	7,945
1035-20	767.4	88.7	Limestone	8,683
1037-05	818.2	34.6	Limestone	4,813
1037-17	757.9	94.9	Limestone	5,830
1038-10	814.4	28.6	Limestone	5,322
1038-17	781.1	62.4	Limestone	6,331
1038-24	745.7	97.8	Limestone	6,382
1041-09	807.3	38.0	Limestone	5,295
1041-17	766.8	78.2	Limestone	6,146
1041-20	752.3	93.0	Limestone	6,515
1042-15d	784.5	61.9	Shale	1,982
1042-19	765.6	80.9	Limestone	6,553
2001-09	779.3	58.7	Limestone	6,317
2001-10	772.1	65.8	Limestone	6,290
2002-22	720.2	116.8	Shale	4,886
2002-23	716.0	121.0	Limestone	7,320
2002-31	675.1	161.9	Limestone	7,767
2002-39	638.4	198.8	Shale	8,603
2002-60	538.7	298.3	Sandstone	5,766
2003-04	808.0	33.0	Limestone	14,241
2003-08	790.3	50.7	Shale	6,590
2004-14	792.2	57.8	Shale	2,081
2004-15	786.6	63.4	Shale	2,471
2004-25	735.8	114.7	Shale	5,059
2005-11	799.1	49.6	Limestone	7,287
2007-13	786.3	54.9	Limestone	7,504
2012 @ 69'	778.6	69.0	Limestone	10,281
2014 @ 43.6'	797.1	43.6	Limestone	7,517
2029-12	789.0	54.6	Shale	1,516
2030-07	806.9	45.1	Shale	6,708
2030-08	801.9	50.7	Limestone	6,241
2030-11	789.2	63.4	Limestone	5,023
2030-17	759.1	93.5	Limestone	4,890
2031-11	810.2	42.9	Limestone	5,906

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Table 2.5.4-224 (Sheet 3 of 3)
Summary of Laboratory-Based Shear Wave Velocity Measurements

Sample No.	Elevation (ft)	Depth (ft)	Material Type	Shear Wave Velocity (fps)
2033-12	814.0	36.7	Limestone	11,510
2035-21	739.5	90.3	Limestone	5,201
2036-06	809.2	23.8	Limestone	7,259
2036-13	774.6	57.8	Limestone	7,037
2037-10	799.8	36.7	Limestone	9,140
2037-23	737.2	99.3	Limestone	4,485
2038-13	767.1	69.9	Limestone	5,058
2041a-15	768.5	63.3	Limestone	4,135
2042a-09	807.1	23.6	Limestone	6,880
2042a-15	778.8	51.9	Limestone	8,495

Statistical Summary

	Material Type	Shear Wave Velocity (fps)
	Shale	2,588
Average	Limestone	7,261
	Sandstone	3,294
	Shale	1,516
Minimum	Limestone	4,135
	Sandstone	2,114
	Shale	3,886
Maximum	Limestone	14,383
	Sandstone	5,766
	Shale	858
Standard Deviation	Limestone	2,086
	Sandstone	1,484

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Table 2.5.4-225 (Sheet 1 of 5)
Summary of Individual Borings Engineering Layers' Top Elevations

		Top Elevation of Engineering Layer (feet)												
		Glen Rose Formation								Twin Mountains Formation				
Boring	Surface Elevation	Layer A	Layer B_1	Layer B_2	Layer C	Layer D	Layer E_1	Layer E_2	Layer E_3	Layer F	Layer G	Layer H	Layer I	Mineral Wells Formation
B1000	851.0	842.3	799.8	791.6	784.0	718.0	714.6	690.5	655.7	620.8	593.6	517.0	453.0	
B1001	842.4	831.1	798.3	790.5	782.0	716.8	713.0	690.1	655.2	622.0	590.9			
B1002	844.7	834.2	798.7	790.7	782.9	717.1	713.3	690.4	655.5	625.8	593.1			
B1003	861.7	851.2	799.1	791.4	783.5	717.6	714.1	691.6	656.6	625.7	596.5			
B1004	859.7	843.2	797.8	790.1	782.4	717.2	713.6	691.4	656.9	622.8	597.7			
B1005	857.0	840.0	798.4	791.0	782.0	717.0	713.4	691.2	655.6					
B1006	847.7	840.2	797.7	789.9	781.9	716.5	713.1	690.5	654.9					
B1006E	840.7	820.7												
B1007	843.6	832.1	799.6	791.7	783.8	718.2	714.3	690.1	656.7	620.9				
B1008	847.6	838.1	800.6	792.7	784.4	716.8	714.8	690.7	656.3	625.6				
B1009I	857.2	843.5	799.6	791.4	784.2	719.1	715.8	691.6	658.2					
B1010	861.4	846.4	799.2	791.4	783.2	717.4	713.4	691.4	656.2					
B1011I	854.7	842.3	797.7	790.2	782.3	717.2	713.7	691.1	656.5					
B1012	844.0	840.0	795.0	790.6	782.1	717.5	712.8	688.6	654.2	620.0	595.0	515.0	454.8	388.1
B1013	863.5	857.5	798.5	790.5	782.5	717.3	714.0	691.0						
B1014	851.2	839.7	797.8	789.4	782.3	716.2	712.7							
B1015	858.1	842.1	799.8	792.0	784.4	719.3	714.6							
B1016	865.6	847.2	799.8	791.5	783.6	717.8	716.6							
B1017	866.4	849.4	801.2	793.2	785.8	721.6	716.4							
B1018	846.4	840.0	798.0	790.3	781.0	716.7	715.3							
B1019	852.2	842.2	797.9	790.0	781.3	716.2	712.8	689.2						
B1020	861.9	843.5	798.1	790.7	782.7	717.7	716.5							
B1021	845.2	829.8	797.6	790.4	782.7	716.7	713.6							
B1022	863.5	845.0	799.0	791.0	782.9	719.3	715.8							
B1023	865.7	850.2	799.1	791.0	783.3	721.7	716.2							
B1024	861.7	844.6	800.1	792.2	784.5									
B1025	861.5	843.5	800.5	791.8	783.3	720.2	715.5							
B1026	865.1	855.6	801.3	792.9	785.7									

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Table 2.5.4-225 (Sheet 2 of 5)
Summary of Individual Borings Engineering Layers' Top Elevations

		Top Elevation of Engineering Layer (feet)												
		Glen Rose Formation						Twin Mountains Formation						
Boring	Surface Elevation	Layer A	Layer B_1	Layer B_2	Layer C	Layer D	Layer E_1	Layer E_2	Layer E_3	Layer F	Layer G	Layer H	Layer I	Mineral Wells Formation
B1027	863.6	848.6	801.7	793.6	786.0	720.2	716.5							
B1028	842.1	829.6	799.6	789.6	784.8									
B1029	846.9	840.9	797.4	788.2	781.9	716.0	712.7							
B1030	858.6	841.1	798.6	790.7	782.9	717.8	714.0							
B1031	864.2	845.7	800.7	792.5	784.7									
B1032	867.3	852.6	800.4	792.6	784.3									
B1033	866.9	857.1	798.7											
B1034	858.5	852.9	801.7	794.1	786.0	720.5	719.1							
B1035	856.1	852.3	801.2	793.4	785.8									
B1036	858.7	847.6	800.4	793.3	784.8									
B1037	852.8	838.4	799.4	791.5	784.0	718.1	714.8							
B1038	843.5	829.7	798.8	790.8	782.8	717.7	714.4							
B1039	847.1	837.1	798.2	790.1	783.2									
B1040	847.0	838.0	798.3	791.4	781.0									
B1041	845.3	834.3	797.9	790.1	780.3	717.1	713.6							
B1042	846.4	835.4	798.4	790.9	781.1	717.7	714.1							
B1043	824.5	819.3	799.8	791.9	784.1									
B1044	838.2	817.7	798.9	790.8	782.7									
B1045	838.2	818.7	799.1	791.4	783.6									
B1046	837.3	787.3												
B1047	831.1	823.4	798.2	790.3	780.5	716.9	713.7							
B1048	835.8	830.8	798.2	790.3	782.5									
B1049	840.4	818.4	799.2	791.1	783.8									
B1050	838.5	801.0	800.7	790.6	782.7	717.1	713.5							
B1051	842.7	829.7	799.0	792.7	782.7									
B1052	842.5	828.5	797.5	790.5	781.9									
B1053	841.4	829.4	795.9	788.9	778.7									
B1054	839.7	806.0	798.9	790.9	783.1									

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Table 2.5.4-225 (Sheet 3 of 5)
Summary of Individual Borings Engineering Layers' Top Elevations

		Top Elevation of Engineering Layer (feet)												
		Glen Rose Formation								Twin Mountains Formation				
Boring	Surface Elevation	Layer A	Layer B_1	Layer B_2	Layer C	Layer D	Layer E_1	Layer E_2	Layer E_3	Layer F	Layer G	Layer H	Layer I	Mineral Wells Formation
B1055	854.8	840.8												
B1056	843.0	830.5	798.7											
B1057	861.1	846.2												
B1058	845.2	840.5	797.8											
B1059	866.2	849.1												
B1060	867.0	852.5												
B1061	852.8	845.3												
B1062	848.4	839.1	799.4											
B1063	837.2	807.2	800.9	793.2	785.0									
B1064	839.1	830.0	799.1	791.2	783.2									
B1065														
B2000	844.0	826.8	796.0	787.8	779.4	715.2	711.5	688.5	654.6	621.4	590.9	506.5	446.0	
B2001	837.9	827.9	796.1	788.3	780.9	716.9	712.9	689.5	655.2	622.6	587.9			
B2002	837.2	825.7	795.0	786.7	779.4	716.3	710.7	688.7	654.7	618.6	584.5			
B2003Off	841.1	828.1	796.1	788.6	781.0	716.8	713.2	689.8	655.2	621.2	590.3	512.7	451.1	
B2004	850.5	840.5	797.6	790.0	782.1	716.9	713.1	690.0	655.7	623.0	597.5			
B2005	848.7	842.7	797.3	789.5	781.5	716.5	715.0	690.2	654.8	623.6				
B2006	845.0	826.5	798.0	789.5	781.1	716.4	712.7	690.0	654.5	622.7				
B2007	841.2	827.2	796.0	788.1	780.7	716.5	712.7	689.4	655.2	621.6				
B2008	840.6	836.9	795.0	787.6	780.1	715.9	711.2	689.5	654.8	621.3				
B2009I	843.0	829.4	795.2	787.3	779.8	716.2	714.7	691.6	655.4					
B2010	846.4	837.4	794.7	787.6	780.2	715.5	713.8	691.4	653.7					
B2011I	847.1	832.1	795.7	790.1	781.8	717.3	713.2	691.1	656.8					
B2012	847.2	838.7	797.7	788.6	781.4	717.2	712.8							
B2013	847.0	843.5	797.5	789.6	782.0	717.0	713.3							
B2014	839.4	830.9	795.4	787.3	779.7	714.7	712.3							
B2015	846.7	839.4	797.5	789.5	782.3	717.1	712.7	690.3						
B2016	841.0	832.5	795.9	788.1	779.2	715.4	710.9							

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Table 2.5.4-225 (Sheet 4 of 5)
Summary of Individual Borings Engineering Layers' Top Elevations

		Top Elevation of Engineering Layer (feet)												
		Glen Rose Formation						Twin Mountains Formation						
Boring	Surface Elevation	Layer A	Layer B_1	Layer B_2	Layer C	Layer D	Layer E_1	Layer E_2	Layer E_3	Layer F	Layer G	Layer H	Layer I	Mineral Wells Formation
B2017	845.4	829.4	798.0	789.8	782.4	716.8	712.8							
B2018Off	848.7	834.6	798.7	790.8	782.7	717.6	712.8	690.9	655.6					
B2019	852.6	843.9	798.5	790.6	782.9	717.6	712.8							
B2020	832.3	821.7	796.6	788.5	781.1	716.0	711.9	689.7						
B2021	849.6	840.9	798.6	790.1	782.8	719.2	714.6							
B2022	852.0	846.5	799.4	791.2	784.1	718.2	713.6							
B2023	843.6	834.1	795.8	787.9	780.2									
B2024	838.2	828.8	795.9	788.8	781.1	717.0	712.9	689.9						
B2025	840.1	831.6	795.4	788.3	780.2									
B2026	833.5	823.5	796.4	787.6	780.0	715.8	712.2	688.5						
B2027	827.0	813.0	794.9	787.0	779.6									
B2028	823.7				776.5									
B2029	843.6	828.1	796.1	788.2	780.6	716.7	715.5							
B2030	852.6	835.0	798.5	790.9	783.0	717.5	713.3							
B2031	853.1	842.4	798.4	790.7	783.0									
B2032	853.0	838.0	799.0	791.0	783.3									
B2033	850.7	835.2	798.2	790.2	782.8									
B2034	839.1	825.1	795.2	787.4	779.8	715.4	710.9							
B2035	829.8	813.6	794.3	786.4	778.7									
B2036	832.4	825.4	795.4	787.4	779.9									
B2037	836.5	827.0	793.7	785.9	777.9	713.3	709.7							
B2038	837.0	826.8	796.9	788.4	781.2	716.7	714.8	690.0						
B2039	833.5	826.0	795.2	786.6	777.7									
B2040	835.1	827.1	794.7	786.7	779.4									
B2041	831.8	823.7	795.2	787.4	779.8	715.0	711.2	688.3						
B2042	830.8	817.0	796.2	788.2	778.8									
B2043	851.7	844.7												
B2044	843.5	826.7	798.8	790.9	782.2									

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Table 2.5.4-225 (Sheet 5 of 5)
Summary of Individual Borings Engineering Layers' Top Elevations

		Top Elevation of Engineering Layer (feet)												
		Glen Rose Formation								Twin Mountains Formation				
Boring	Surface Elevation	Layer A	Layer B_1	Layer B_2	Layer C	Layer D	Layer E_1	Layer E_2	Layer E_3	Layer F	Layer G	Layer H	Layer I	Mineral Wells Formation
B2045	832.3	828.6	798.7	791.0	783.0									
B2046	828.1	817.6	797.4	789.1	781.4									
B2047	793.7				780.4									
B2048	844.2	825.8	798.9	791.1	783.3	717.1	713.7							
B2049	839.7	828.7	799.8	790.7	782.9									
B2050	827.9	812.9	797.4	789.4	781.6									
B2051	810.0	804.3	796.5	788.6	780.9	715.9	710.6							
B2052	846.6	839.1	798.9	791.0	783.1									
B2053	839.1	829.6	798.0	789.6	782.1									
B2054	834.5	829.5	797.5	789.7	782.0									
B2055	815.3	809.9	796.9	788.5	780.9									
B2056	852.3	847.3												
B2057	831.7	824.2	796.8	788.9	781.7									
B2058	853.7	843.7												
B2059	831.7	826.7	795.9	788.2	783.1									
B2060	853.7	843.7												
B2061	851.2	842.2												
B2062	831.3	824.3	794.7	787.0	780.3									
B2063	828.2	812.1	794.7	786.0	779.1									
B2064														

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Table 2.5.4-226
Summary of Rock Low Strain Properties and Settlement Best Estimate Modulus Profile

Engineering Layers	Primary Lithology	Mean V_s (fps)	Mean V_p (fps)	Poisson's Ratio	Assigned Total Unit Weight (pcf)	Low Strain Shear Modulus, G_{max} (tsf)	Low Strain Young's Modulus, E_{max} (tsf)	Strain Reduction Factor	BE Profile (Strain Adjusted E_{max}) (tsf)
A	Limestone	3,548	8,788	0.40	145	28,343	79,361	0.98	77,774
B1	Shale	2,609	6,736	0.41	135	14,269	40,239	0.55	22,131
B2	Shale with Limestone interbeds	2,716	7,640	0.43	135	15,463	44,226	0.55	24,324
C	Limestone	5,685	11,324	0.33	155	77,787	206,913	0.98	202,775
D	Shale	3,019	8,312	0.42	135	19,106	54,2612	0.55	29,844
E1	Limestone	4,943	10,486	0.36	155	58,807	159,954	0.98	156,755
E2	Limestone	6,880	13,164	0.31	155	113,926	298,486	0.98	292,516
E3	Limestone	4,042	9,255	0.38	150	38,054	105,029	0.98	102,928
F	Limestone with Shales and Sand interbeds	3,061	7,927	0.41	130	18,914	53,338	0.68	36,270
G	Sandstone	3,290	7,593	0.38	135	22,690	62,625	0.88	55,110
H	Shale	3,429	8,188	0.39	140	25,561	71,060	0.83	58,979
I	Sandstone	3,092	7,686	0.40	145	21,526	60,273	0.92	55,451
MW	Shales with Sandstone and Limestone interbeds	5,546	10,627	0.32	150	71,642	189,134	1.00	189,134

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Table 2.5.4-227
Summary of Rock Mass Properties and Settlement Lower Bound Modulus Profile

Engineering Layers	Primary Lithology	Assigned Total Unit Weight (pcf)	RMR	GSI	Rock Mass Cohesion (tsf)	Rock Mass Friction Angle (degrees)	LB Profile Rock Mass Young's Modulus, Erm (tsf)
A	Limestone	145	76	71	3.1 – 4.2	35 - 45	45,754
B1	Shale	135	63	58	3.1 – 4.2	35 - 45	5,825
B2	Shale with Limestone interbeds	135	63	58	3.1 – 4.2	35 - 45	5,825
C	Limestone	155	79	74	3.1 – 4.2	35 - 45	69,606
D	Shale	135	63	58	3.1 – 4.2	35 - 45	5,825
E1	Limestone	155	79	74	3.1 – 4.2	35 - 45	69,606
E2	Limestone	155	79	74	3.1 – 4.2	35 - 45	69,606
E3	Limestone	150	79	74	3.1 – 4.2	35 - 45	69,606
F	Limestone with Shales and Sand interbeds	130	75	70	3.1 – 4.2	35 - 45	22,377
G	Sandstone	135	84	79	> 4.2	> 45	45,933
H	Shale	140	83	78	> 4.2	> 45	37,587
I	Sandstone	145	83	78	> 4.2	> 45	45,105
MW	Shales with Sandstone and Limestone interbeds	150	89	84	> 4.2	> 45	100,149

Table 2.5.4-228
Summary of Ultimate Bearing Capacities

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Ultimate Bearing Capacity (ksf)		
		E-W	N-S		General Shear	Local Shear	Compression
R/B	I	213	309	783	354	348	146
T/B	II	186	315	795	342	339	146
A/B	II	133	239	785	338	335	146
EPS/B	I	115	69	785	343	340	146
WPS/B	I	115	69	785	343	340	146
PSFSV	I	85	78	782	365	362	146
UHS	I	131	131	787	369	365	146

Table 2.5.4-229
Summary of Settlement Estimates Based on “BE” Profile

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Foundation Static Load (ksf)	Settlement Estimate for Center (in)	
		E-W	N-S			Non-Layered Method	Layered Method
R/B	I	213	309	783	11.3	0.12	0.20
T/B	II	186	315	795	5.9	0.07	0.11
A/B	II	133	239	785	6.8	0.09	0.14
EPS/B	I	115	69	785	4.3	0.07	0.10
WPS/B	I	115	69	785	4.3	0.08	0.12
PSFSV	I	85	78	782	5.4	0.06	0.09
UHS	I	131	131	787	3.6	0.05	0.06

Table 2.5.4-230
Summary of Settlement Estimates Based on “LB” Profile

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Foundation Static Load (ksf)	Settlement Estimate for Center (in)	
		E-W	N-S			Non-Layered Method	Layered Method
R/B	I	213	309	783	11.3	0.30	0.37
T/B	II	186	315	795	5.9	0.19	0.20
A/B	II	133	239	785	6.8	0.23	0.26
EPS/B	I	115	69	785	4.3	0.18	0.18
WPS/B	I	115	69	785	4.3	0.20	0.21
PSFSV	I	85	78	782	5.4	0.17	0.16
UHS	I	131	131	787	3.6	0.14	0.12

Table 2.5.4-231
Summary of Rebound Estimates Based on “BE” Profile

Structure	Category	Foundation Size (ft)		Excavation Depth (ft)	Rebound Estimates for Center (in)	
		E-W	N-S		Non-Layered Method	Layered Method
R/B	I	213	309	40-50	0.07	0.12
T/B	II	186	315	40-50	0.06	0.10
A/B	II	133	239	40-50	0.07	0.10
EPS/B	I	115	69	40-50	0.06	0.08
WPS/B	I	115	69	40-50	0.06	0.10
PSFSV	I	85	78	40-50	0.05	0.08
UHS	I	131	131	40-50	0.05	0.07

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Table 2.5.5-201
Permanent Slopes Within CPNPP Units 3 and 4 Vicinity

Slope Location	Adjacent Seismic Category I Structure	Slope Type	Constructed Condition	Slope Orientation Relative to Yard Grade	Minimum Distance to Slope Crest/Toe	Slope Height[(a)]	Maximum Slope Inclination (Horizontal:Vertical)
North-Northwest of Unit 4	R/B UHS	Fill	Engineered Fill, Residual Soil & Native Rock	Descending	360 ft to R/B 100 ft to UHS	40 ft	2:1
West of Unit 4	R/B WPS/B PSFSV	Cut	Residual Soil & Native Rock	Ascending	330 ft to R/B 210 ft to WPS/B 170 ft to PSFSV	35 ft	2:1
Southwest of Unit 4	R/B PSFSV	Cut	Residual Soil & Native Rock	Ascending	420 ft to R/B 210 ft to PSFSV	50 ft	2.5:1
South of Unit 4	R/B PSFSV	Cut	Residual Soil & Native Rock	Ascending	540 ft to R/B 420 ft to PSFSV	45 ft	3:1
Northeast of Unit 3	R/B UHS	Fill	Engineered Fill, Residual Soil & Native Rock	Descending	310 ft to R/B 30 ft to UHS	45 ft	2:1
East-Northeast of Unit 3	R/B EPS/B	Cut	Residual Soil & Native Rock	Descending	230 ft to R/B 130 ft to EPS/B	15 ft	3:1
South of Unit 3	R/B PSFSV	Cut	Residual Soil & Native Rock	Ascending	630 ft to R/B 500 ft to PSFSV	15 ft	3:1

a) Slope heights are with respect to yard grade elevation of 822 ft for ascending slopes and with respect to Squaw Creek Lake elevation level of 775 ft for descending slopes.

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Table 2.5.5-202
Summary of Material Parameters for Stability Analysis

Material	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Residual Soil	110	25	200
Undocumented Fill	110	25	200
Compacted Fill	125	32	200
Shale (Layer B)	135	Non-linear (Lower Bound Envelope, see Figure 2.5.4-235)	
Limestone (Layer A)	145	Non-linear (Layer A Lower Bound Envelope, see Figure 2.5.4-237)	
Limestone (Layer C)	155	Non-linear (Layer C Lower Bound Envelope, see Figure 2.5.4-237)	

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Table 2.5.5-203
Summary of Stability Analyses

Cases	Cross Section	Static Slope Stability Factor of Safety	Pseudo-static Slope Stability Factor of Safety
Permanent	D-D’	2.80	1.96
Permanent	E-E’	2.06	1.66
Permanent	E1-E1’	1.93	1.47
Permanent	F-F’	2.14	1.56