PMComanchePekNPEm Resource

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Subject: Partial Response to RAI #11, #22, #21, and #14
Attachments: TXNB-09035 RAIs #11, #14, #21, #22 _partial_.pdf

Luminant has submitted the responses to 8 geotech questions from the subject RAIs. If there are any questions regarding these responses, please contact me or contact Don Woodlan (254-897-6887, Donald.Woodlan@luminant.com).

Thanks,

John Conly
COLA Project Manager NuBuild
Luminant Power
(254) 897-5256

Hearing Identifier: ComanchePeak_COL_NonPublic
Email Number: 781

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Subject: Partial Response to RAI #11, #22, #21, and #14
Sent Date: 8/29/2009 2:58:55 PM
Received Date: 8/29/2009 3:17:54 PM
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Ref. # 10 CFR 52

August 28, 2009

U. S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555
ATTN: David B. Matthews, Director
Division of New Reactor Licensing

SUBJECT: COMANCHE PEAK NUCLEAR POWER PLANT, UNITS 3 AND 4
DOCKET NUMBERS 52-034 AND 52-035
PARTIAL RESPONSES TO REQUESTS FOR ADDITIONAL INFORMATION
NO. 1889, 2929, 3015, AND 3037

Dear Sir:

Luminant Generation Company LLC (Luminant) hereby submits the attached partial responses to Requests for Additional Information for the Combined License Application for Comanche Peak Nuclear Power Plant Units 3 and 4. The specific questions answered are:

RAI #1889 (CP RAI #11)	RAI #2929 (CP RAI #22)	RAI #3015 (CP RAI #21)	RAI #3037 (CP RAI #14)
02.05.02-7	02.05.04-11	02.05.01-5	02.05.01-1
02.05.02-9	02.05.04-12		
02.05.02-10			
02.05.02-21			

Should you have any questions regarding these responses, please contact Don Woodlan (254-897-6887, Donald.Woodlan@luminant.com) or me.

There are no commitments in this letter.

I state under penalty of perjury that the foregoing is true and correct. Executed on August 28, 2009.

Sincerely,

Luminant Generation Company LLC

Donald R. Woodlan for

Rafael Flores

Attachments - 1. Response to Request for Additional Information No. 1889 (CP RAI #11)
2. Response to Request for Additional Information No. 2929 (CP RAI #22)
3. Response to Request for Additional Information No. 3015 (CP RAI#21)
4. Response to Request for Additional Information No. 3037 (CP RAI #14)

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RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak Unit 3/4

Luminant Generation Company LLC

Docket No. 52-034 and 52-035

RAI NO.: 1889 (CP #11)

SRP SECTION: 02.05.02 – VIBRATORY GROUND MOTION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/01/2009

QUESTION NO.: 02.05.02-7

FSAR Subsection 2.5.2.5 states that "The average shear wave velocity of Layer C is greater than 6000 ft/sec". FSAR Figure 2.5.4-239 shows the average shear wave velocity of Layer C is less than 6000 ft/sec. Please assess the differences between the text and Figure 2.5.4-239 and provide any correction.

ANSWER:

The value in the text was incorrectly stated. The text was revised to state that the shear wave velocity of Layer C is greater than 5800 ft/sec, which is consistent with FSAR Figure 2.5.4-239.

Impact on R-COLA

FSAR Revision 0 page 2.5-110 was revised to reflect this response.

See attached changes for page 2.5-110. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

Impact on S-COLA

None.

Impact on DCD

None.

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 ~~6000~~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. RCOL2_02.05.02-7

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 Description of Site Response Analysis

CTS-00515

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.

2.5.2.5.1.1 Generation of Synthetic Profiles

To account for the epistemic and aleatory uncertainties in the site's dynamic properties, multiple of 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 new 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.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak Unit 3/4

Luminant Generation Company LLC

Docket No. 52-034 and 52-035

RAI NO.: 1889 (CP #11)

SRP SECTION: 02.05.02 – VIBRATORY GROUND MOTION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/01/2009

QUESTION NO.: 02.05.02-9

In FSAR Subsection 2.5.2.6.1.2 you stated that "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".

a. Please further justify this conclusion.

b. Please explain why a qualitative argument is used to estimate the vertical GMRS rather than a quantitative methodology

---

ANSWER:

The justification for this conclusion was provided as an FSAR mark-up in Luminant letter TXNB-09005, dated April 2, 2009 (ML091120280). This revision was identified as CTS-00516 on FSAR pages 2.5-116 through 2.5-118, Tables 2.5.2-236 and 2.5.2-237, and Figures 2.5.2-247 through 2.5.2-252.

Impact on R-COLA

None.

Impact on S-COLA

None.

Impact on DCD

None.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak Unit 3/4

Luminant Generation Company LLC

Docket No. 52-034 and 52-035

RAI NO.: 1889 (CP #11)

SRP SECTION: 02.05.02 – VIBRATORY GROUND MOTION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/01/2009

QUESTION NO.: 02.05.02-10

In supplement to FSAR Subsection 2.5.2.6.1.2 you stated that "The Comanche Peak site is a deep, soft-rock site with shales and limestones 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 rock sites". Please provide further justification of this statement.

ANSWER:

The requested explanation of the statement in question is addressed by the section taken from FSAR Subsection 2.5.2.6.1.2 and repeated below concerning V/H ratios at CPNPP 3 and 4.

> 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 US-APWR DCD 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 DCD). 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 DCD. 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 2.5.2-233.

The 2600 fps is the shales and limestones near the surface having shear-wave velocity as determined by testing.

<u>Impact on R-COLA</u>

None.

Impact on S-COLA

None.

Impact on DCD

None.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak Unit 3/4

Luminant Generation Company LLC

Docket No. 52-034 and 52-035

RAI NO.: 1889 (CP #11)

SRP SECTION: 02.05.02 – VIBRATORY MOTION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/01/2009

QUESTION NO.: 02.05.02-21

1. The following is a list of editorial corrections that the staff has identified. Please provide an updated text that includes these corrections.

    a. FSAR Subsection 2.5.2.1.2, the southern Oklahoma aulacogen is not outlined on Figure 2.5.2-202 as suggested by the FSAR.

    b. FSAR Subsection 2.5.2.2.1.1 cites the wrong figure in the statement: "The Ouachita source zone extends from Arkansas into east Texas (Figure 2.5.2-233) and was defined to encompass the extent of the Ouachita fold belt within this region." Citation should read "2.5.2-203."

    c. FSAR Subsection 2.5.2.4.1 refers to current COLA calculations in Tables 2.5.2-208 and 2.5.2-209 as 2007, but they are labeled as 2008 in the tables.

    d. In FSAR Subsection 2.5.2.4.2.3.2, the reference to Figure 2.5-211, should be to Figure 2.5.1-211.

    e. FSAR Subsection 2.5.2.4.2.3.3 uses the term "tensile stress regime." Is the correct term "extensional stress regime"?

    f. FSAR Subsection 2.5.2.4.4 refers to the shaded cells in Table 2.5.2-220. There are no shaded cells in Table 2.5.2-220.

    g. In FSAR Subsection 2.5.2.6.2, Table 2.5.2-227, there are superscript numbers associated with particular values as if there were notes or footnotes (e.g. shallow site profile[1]). Are there corresponding notes?

ANSWER:

a. No discussion of the southern Oklahoma aulacogen is presented in Subsection 2.5.2.1.2. However, a statement regarding the spatial distribution of seismicity and the southern

Oklahoma aulacogen is made in Subsection 2.5.2.1.3. That statement refers to Figure 2.5.1-208 and Figure 2.5.2-202. The southern Oklahoma aulacogen is outlined in Figure 2.5.1-208, while only the updated seismicity is shown in Figure 2.5.2-202. Together, these figures indicate the spatial similarity in the areas of higher seismicity and the southern Oklahoma aulacogen.

b. Text has been corrected.

c. Tables 2.5.2-208 and 2.5.2-209 were revised to reflect dates of 2007 because these tables were created from data collected in 2007.

d. References to Figure 2.5-211 were changed to Figure 2.5.1-211.

e. The term “tensile stress regime” is used to describe the regional stress. “Extension” would have been used if describing strain. [Marrett and Peacock (1999), Journal of Structural Geology].

f. Table 2.5.2-220 was revised.

g. There are notes at the end of Table 2.5.2-227, Sheet 6 of 6, to explain the superscript notations.

<u>Impact on R-COLA</u>

FSAR Revision 0 pages 2.5-73, 2.5-96, 2.5-97, 2.5-286, 2.5-287 and 2.5-300, were revised to reflect this response.

See attached changes for pages 2.5-73, 2.5-96, 2.5-97, 2.5-301, 2.5-302 and 2.5-315. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

<u>Impact on S-COLA</u>

None.

<u>Impact on DCD</u>

None.

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

The Bechtel Group EST defined five source zones that contributed to hazard at CPNPP Units 1 and 2 (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 did not contribute to hazard at CPNPP Units 1 and 2 (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 contributed to hazard at CPNPP Units 1 and 2 and are used in the PSHA for CPNPP Units 3 and 4:

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-~~233~~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. RCOL2_02.05.02-21

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

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 ~~2.5-211~~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 ~~2.5-211~~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

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scenarios) to occur closer to CPNPP Units 3 and 4 than if the fault trace also included the uncertainty (Figure ~~2.5-211~~2.5.1-211). RCOL2_02.05.02-21

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.

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Table 2.5.2-208
Comparison of PGA Hazard Results

PGA comparison

Ampl. (cm/s^2)	Mean			Median			0.85 fractile		
	EPRI-SOG	~~2008~~2007	% diff	EPRI-SOG	~~2008~~2007	% diff	EPRI-SOG	~~2008~~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%

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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	~~2008~~2007	% diff	EPRI-SOG	~~2008~~2007	% diff	EPRI-SOG	~~2008~~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%

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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	7.2	7.5	6.8	7.5	5.8	7.3
R	450	710	170	620	25	380
M (r >100 km)	7.6	7.6	7.7	7.7	7.8	7.8
R (r >100 km)	800	820	850	860	860	860

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RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 2929 (CP #22)

SRP SECTION: 02.05.04 - STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/17/2009

QUESTION NO.: 02.05.04-11

NUREG-0800, Standard Review Plan (SRP), Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," establishes criteria that the NRC staff intends to use to evaluate whether an applicant meets the NRC's regulations.

Subsection 2.5.4.5.1.2 in the FSAR proposes that concrete fill will be used for foundation preparing, and further states that the fill concrete has a design compressive strength of 3,000 psi to meet the strength requirement. Please address the concrete durability, as described in American Concrete Institute (ACI) 201.2R, for fill concrete.

Erosion of porous concrete sub-foundation, as described in NRC Information Notice (IN) 97-11, and leaching of calcium hydroxide could be potential problems, since the assumed water ground table (EL. 780 ft) is very close to proposed approximate excavation bottom (about EL. 782 ft), and even could be higher than some localized excavation areas, which need to be deepened below EL. 782 ft to remove disturbed or unstable material. In addition, ground water and perched water seeping down along the sides of the structures could cause potential impact on porous concrete fill. Please explain how the differential settlement due to erosion, and loss of concrete strength due to leaching, will be addressed, and provide justification for the manner in which these potential issues will be addressed.

ANSWER:

Durability of the fill concrete will be assured by the site-specific mix design and by the particular site conditions at CPNPP. Safety-related fill concrete will conform to durability requirements given in ACI 349 Chapter 4.

ACI 201.2R identifies several potential issues affecting the durability of concrete, including freeze-thaw action, ice removal agents, chemical attack by sulfate, salt attack, seawater exposure, acid attack, corrosion of rebar in the concrete, and chemical reactions of aggregates. The CPNPP site is located away from the ocean and salt water bodies such that the fill concrete will not be 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. 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 fill concrete is essentially unreinforced concrete and therefore corrosion of rebar in the concrete is not an issue for this concrete. It is anticipated that some reinforcing will be present in the fill concrete where necessary at construction joints in the fill.

The foundation and fill design at CPNPP are such that the issues contained in IN 97-11 will not be applicable to fill concrete. No mortar or concrete containing high amounts of calcium aluminate cement is planned to be used in foundation or fill concrete. The fill concrete mix design will use Type II Portland cement, consistent with Subsection 3.8.4.6.1 of the US-APWR DCD, which is limited to a tricalcium aluminate content of 8% by ASTM C150. Although the CPNPP site conditions do not pose concerns of sulfate attack, this cement is classified as moderately resistant to sulfate attack. Subsection 2.5.5.2.3 states that the groundwater table was conservatively assumed to be at elevation 780 ft for purposes of modeling slope stability. However, with respect to potential groundwater effects on concrete, the maximum anticipated groundwater elevation is at elevation 760 ft, as stated in Subsections 2.4.12.5 and 2.5.4.1.7. This is well below the anticipated bottom of fill concrete. The fill concrete mix design is not anticipated to use porous concrete consisting only of coarse aggregates and cement; fine aggregates are also intended. 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. Nevertheless, perched water and precipitation run-off 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 are not anticipated to be an issue at CPNPP. Subsection 3.8.4.7 commits to periodically monitoring ground water chemistry to assure that it remains nonaggressive with respect to concrete structures.

The site-specific mix design for fill concrete has not been performed at this time. The mix design will conform to pertinent ACI and ASTM standards. Based on the above discussion, no admixtures specifically intended to reduce fill concrete porosity are anticipated to be employed in the fill concrete at this time. Fly ash and/or other pozzolanic ingredients, which tend to reduce the porosity of concrete and are common ingredients in concrete, will likely be employed in the course of mix design. See the response to Question 02.05.04-12 for further discussion of fill concrete.

Impact on R-COLA

FSAR Revision 0 pages 2.5-179 and 2.5-228 were revised to reflect this response.

See attached changes for pages 2.5-184, 2.5-185 and 2.5-243. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

Impact on S-COLA

None.

Impact on DCD

None.

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.

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. CTS-00514 RCOL2_02.05.04-12 CTS-00815

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. RCOL2_02.05.04-11

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 RCOL2_02.05.04-11 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 Subsections 2.4.12.5 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. RCOL2_02.05.04-11

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. CTS-00815

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. CTS-00514 CTS-00815

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.

2.5-433 Constantino C.J. (1996). Recommendations for Uncertainty Estimated in Shear Modulus Reduction and Hysteretic Damping Relationships. Published as an appendix in Silva, W.J., N. Abrahamson, G. Toro and C. Constantino. (1997). "Description and validation of the stochastic ground motion model." Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573. CTS-00515

2.5-434 Idriss, I.M., and Sun, J. I. (1992). SHAKE91: A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally layered Soil Deposits, Dept. of Civil and Environmental Engineering, Center for Geotechnical Modeling, Univ. of California, Davis, Calif.

2.5-435 Rathje, E.M., and M.C. Ozbey (2006). Site-Specific Validation of Random Vibration Theory-Based Seismic Site Response Analysis. Journal of Geotechnical and Geoenviromnental Engineering, ASCE, Vol 132, No.7, July.

2.5-436 Kramer, Steven L. (1996). Geotechnical Earthquake Engineering. Prentice-Hall.

2.5-437 Perloff, W.H., Baron, W. (1976), Soil Mechanics Principles and Applications, The Ronald Press Company, N.Y. CTS-00554

2.5-438 Taylor, D.W. (1948), Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York.

2.5-439 Poulos, H.G., and Davis, E.H. (1974), Elastic Solutions for Soil and Rock Mechanics, Wiley and Sons, New York.

2.5-440 American Concrete Institute, ACI 349-01, Code Requirements for Nuclear Safety Related Concrete Structures. RCOL2_02.05.04-11 RCOL2_02.05.04-12

2.5-441 2.5-441United States Nuclear Regulatory Commission, Information Notice 97-11, Cement Erosion From Containment Subfoundations At Nuclear Power Plants, March 21, 1997. RCOL2_02.05.04-11

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 2929 (CP #22)

SRP SECTION: 02.05.04 - STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 07/17/2009

QUESTION NO.: 2.5.4-12

By letter dated April 2, 2009, Luminant provided a revision to FSAR Subsection 2.5.4.5.4.1.2 “Fill Concrete.” In its revision, Luminant proposed using the American Society for Testing and Material (ASTM) C94/C94M-07 “Standard Specification for Ready-Mixed Concrete,” for use of ready mixed concrete for backfill purposes. The bulk of the ASTM C94/C94M standard is a performance, or end-result, specification. ASTM C94/C94M does not prescribe a method of achieving these requirements and results, such as how to achieve the slump, the air content, the temperature, or minimum strengths. Please indicate why the ASTM C94/C94M standard, and not the standard in American Concrete Institute 349, will be used.

ANSWER:

The fill concrete placed under seismic category I foundations is safety-related and it is agreed that it is subject to the pertinent requirements of ACI 349. ACI 349 was added to the discussion in Subsection 2.5.4.5.4.1.2. Other ACI and ASTM standards applicable to concrete used at CPNPP, and relating to slump, air content, mix temperature, and minimum strength, are discussed in Subsection 3.8.4.6.1.1 of the US-APWR DCD which is incorporated by reference in Chapter 3 of the CPNPP FSAR.

<u>Impact on R-COLA</u>

FSAR Revision 0 pages 2.5-179 and 2.5-228 were revised to reflect this response.

See attached changes for pages 2.5-184 and 2.5-243. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

<u>Impact on S-COLA</u>

None.

<u>Impact on DCD</u>

None.

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.

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.

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CTS-00815

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.

RCOL2_02.05.04-11

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

RCOL2_02.05.04-11

2.5-433 Constantino C.J. (1996). Recommendations for Uncertainty Estimated in Shear Modulus Reduction and Hysteretic Damping Relationships. Published as an appendix in Silva, W.J., N. Abrahamson, G. Toro and C. Constantino. (1997). "Description and validation of the stochastic ground motion model." Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573. CTS-00515

2.5-434 Idriss, I.M., and Sun, J. I. (1992). SHAKE91: A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally layered Soil Deposits, Dept. of Civil and Environmental Engineering, Center for Geotechnical Modeling, Univ. of California, Davis, Calif.

2.5-435 Rathje, E.M., and M.C. Ozbey (2006). Site-Specific Validation of Random Vibration Theory-Based Seismic Site Response Analysis. Journal of Geotechnical and Geoenviromnental Engineering, ASCE, Vol 132, No.7, July.

2.5-436 Kramer, Steven L. (1996). Geotechnical Earthquake Engineering. Prentice-Hall.

2.5-437 Perloff, W.H., Baron, W. (1976), Soil Mechanics Principles and Applications, The Ronald Press Company, N.Y. CTS-00554

2.5-438 Taylor, D.W. (1948), Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York.

2.5-439 Poulos, H.G., and Davis, E.H. (1974), Elastic Solutions for Soil and Rock Mechanics, Wiley and Sons, New York.

2.5-440 American Concrete Institute, ACI 349-01, Code Requirements for Nuclear Safety Related Concrete Structures. RCOL2_02.05.04-11 RCOL2_02.05.04-12

2.5-441 2.5-441United States Nuclear Regulatory Commission, Information Notice 97-11, Cement Erosion From Containment Subfoundations At Nuclear Power Plants, March 21, 1997. RCOL2_02.05.04-11

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak Unit 3/4

Luminant Generation Company LLC

Docket No. 52-034 and 52-035

RAI NO.: 3015 (CP #21)

SRP SECTION: 02.05.01 – BASIC GEOLOGIC AND SEISMIC INFORMATION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

DATE OF RAI ISSUE: 07/15/2009

QUESTION NO.: 02.05.01-5

NUREG-0800, Standard Review Plan (SRP), Chapter 2.5.1, 'Basic Geologic and Seismic Information,' establishes criteria that the NRC staff intends to use to evaluate whether an applicant meets the NRC's regulations.

FSAR Subsection 2.5.1.1.3.1 describes a circular gravity anomaly in the southeastern portion of the site region associated with the Llano Uplift. However, FSAR Figure 2.5.1-205 shows the Llano Uplift in the southwestern portion of the site region. Please clarify if the circular gravity anomaly is associated with the Llano Uplift in the southwestern portion of the site region, or with another feature in the southeastern portion of the site region.

ANSWER:

The location of the Llano Uplift and its geophysical anomaly are incorrectly stated in the FSAR Subsection 2.5.1.1.3.1. The Llano Uplift and its geophysical anomaly occur in southwestern portions of the site region. FSAR Subsection 2.5.1.1.3.1 was modified to correctly state the location of the Llano Uplift.

<u>Impact on R-COLA</u>

FSAR Revision 0 page 2.5-10 was revised to reflect this response.

See attached changes for page 2.5-10. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

<u>Impact on S-COLA</u>

None.

<u>Impact on DCD</u>

None.

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 south~~eastern~~western 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. RCOL2_02.05.01-5

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

RAI NO.: 3037 (CP RAI #14)

SRP Section: 02.05.01 – BASIC GEOLOGIC AND SEISMIC INFORMATION

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

DATE OF RAI ISSUE: 07/13/2009

QUESTION NO.: 02.05.01-1

NUREG-0800, Standard Review Plan (SRP), Chapter 2.5.1, 'Basic Geologic and Seismic Information,' establishes criteria that the NRC staff intends to use to evaluate whether an applicant meets the NRC's regulations.

FSAR Subsections 2.5.1.1.3.1 and 2.5.1.1.3.2 discuss regional gravity and aeromagnetic data for the CPNPP site. In the FSAR, you conclude that no features (other than the Meers) "are recognizable in the regional aeromagnetic field that would indicate a capable tectonic source within the site region." Prior to that, you stated that the aeromagnetic signature associated with the Meers fault is related to "Late Paleozoic thrusting and not an expression of the recent kinematic history of the fault." Please explain the limits to using gravity and aeromagnetic data for identifying capable tectonic structures in the CPNPP site region and the likelihood that Cenozoic deformation features would provide a recognizable signature in the data.

ANSWER:

This statement was not intended to convey that the potential field methods, in and of themselves, are able to determine fault capability or Cenozoic movement. It was intended to point out that no other features similar to the Meers fault were present. This statement was removed from FSAR Subsections 2.5.1.1.3.1 and 2.5.1.1.3.2.

<u>Impact on R-COLA</u>

FSAR Revision 0 pages 2.5-11 and 2.5-12 were revised to reflect this response.

See attached changes for pages 2.5-11 and 2.5-12. Because of the text additions and deletions, the page numbers on the mark-up FSAR pages may differ from the page numbers in FSAR Revision 0.

<u>Impact on S-COLA</u>

None.

Impact on DCD

None.

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. ~~The gravity data give no indication of capable seismic sources within the site region.~~ RCOL2_02.05.01-1

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). ~~No other features are recognizable in the regional aeromagnetic field that would indicate a capable tectonic source within the site region.~~ RCOL2_02.05.01-1

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