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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
RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION NO. 2994

Dear Sir:

Luminant Generation Company LLC (Luminant) herein submits the response to Request for Additional Information No. 2994 for the Combined License Application for Comanche Peak Nuclear Power Plant Units 3 and 4. The affected Final Safety Analysis Report pages are included with the response.

The response to Question 03.08.04-1 in Attachment 3 contains sensitive unclassified non-safeguards information (SUNSI). The NRC is requested to withhold Attachment 3 from public disclosure under 10 CFR 2.390(d)(1). This letter is unclassified upon separation from Attachment 3. Attachment 1 provides the same response without the SUNSI.

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

The only commitment made in this letter is specified on page 3.

I state under penalty of perjury that the foregoing is true and correct.

Executed on December 10, 2009.

Sincerely,

Luminant Generation Company LLC

Donald R. Woodlan for

Rafael Flores

Attachments
1. Response to Request for Additional Information No. 2994 (CP RAI #108) Question 03.08.04-1 (Public Version)
2. Response to Request for Additional Information No. 2994 (CP RAI #108) Questions 03.08.04-2 through 03.08.04-17 (Unclassified)
3. Response to Request for Additional Information No. 2994 (CP RAI #108) Question 03.08.04-1 (Non-Public Version)

D090
NRO

cc: Stephen Monarque w/all attachments

Electronic distribution w/Attachment 1

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Regulatory Commitments in this Letter

This communication contains the following new or revised commitments which will be completed or incorporated into the CPNPP licensing basis as noted. The Commitment Number is used by Luminant for internal tracking.

Number	Commitment	Due Date/Event
6871	The response to RAI No. 3006 (CP RAI #122) Question 03.08.04-52, to be submitted no later than December 21, 2009, provides further discussion on the testing methods of the engineered backfill for CPNPP Units 3 and 4.	December 21, 2009
6881	The description of this [equivalent time travel] method will be added to FSAR Sections 3NN.2 and 3NN.3 in the response to RAI No. 3006 (CP RAI #122) Question 03.08.04-53 to be submitted to the NRC no later than December 21, 2009.	December 21, 2009

Attachment 1

Response to Request for Additional Information No. 2994 (CP RAI #108) Question 03.08.04-1 (Public Version)

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-1

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(19) in Comanche Peak Nuclear Power Plant (CPNPP) COL FSAR, Subsection 3.8.4.1.3.1, "ESWPT" (Page 3.8-5), the first paragraph states that "The ESWPT [essential service water pipe tunnel] is an underground reinforced concrete structure. Figure 3.8-203 shows the typical section of the ESWPT...The tunnel is divided into two sections by an interior concrete wall to provide separation of piping trains. Each section contains both ESWS [essential service water system] supply and return lines."

The applicant is requested to provide the following information:

(a) In CPNPP COL FSAR Figure 3.8-202, the top of concrete for ESWPT is at EL 810.25 ft, whereas in Figure 3.8-203, it is at EL 809.75 ft. Explain this discrepancy.

(b) In Figure 3.8-202, the thickness of the top slab of ESWPT is 2 ft-6 in., whereas, in Figure 3.8-203, it is 2 ft-0 in. Explain this discrepancy.

(c) In Figure 3.8-202, call out the rebar size and quantity, and indicate on the drawing which pipe is the supply line and which one is the return line.

(d) In the right cross section of Figure 3.8-202, the remark under the shear key of the base slab states that "SHEAR KEY – SEE DETAIL THIS DRAWING," but there is no detail given in the drawing. Provide this detail.

(e) In the right drawing of CP COL Figure 3.8-202, the remark at right states "UHS BASIN FOR REINF. SEE FIGURE 3.8-210." However, no rebar information is given in Figure 3.8-210. Call out rebar size and quantity in Figure 3.8-210.

ANSWER:

(a) There are three types of ESWPT segments as shown on the key site plan in FSAR Figure 3.8-201 that comprise the entire ESWPT.

Expansion joints separate the tunnel into these segments. The segments were grouped into the following three different types.

- Tunnel Segment 1, as shown in Section G in FSAR Figure 3.8-203, is representative of typical tunnel segments to the east and west of the R/B. The top of the fill concrete is EL 810.25 ft.
- Tunnel Segment 2, as shown in Sections F and F' in FSAR Figure 3.8-202, is representative of segments adjacent to the Ultimate Heat Sink (UHS) structures. A tornado missile shield extends from the top of this segment to protect openings in the UHS. The top of the fill concrete is EL 809.75 ft.
- Tunnel Segment 3, as shown in Sections H and H' in FSAR Figure 3.8-204 is representative of segments with fuel pipe access tunnels extending from the top. These are located adjacent to the PSFSVs. The top of the fill concrete is EL 810.25 ft.

A key plan which shows the locations of the three segments has been added to FSAR Figure 3.8-201. In addition, FSAR Subsections 3.8.4.1.3.1 and 3.8.5.1.3.1 have been revised to add more discussion concerning the design of the ESWPT.

Each segment was designed separately and required different thicknesses of concrete for the specified loading conditions. Segments 1 and 3 have roof slab and mat slab thicknesses of 2'-0" while Segment 2 has a roof slab and mat slab thickness of 2'-6".

All segments were designed for the same basic load conditions, but due to differing geometry the values of some of the loads (seismic, soil pressure, live loads, etc.) varied. The resulting moments and shears also varied. Thus, Segment 2 requires a thicker roof slab because this segment includes the tornado missile shield structure. This requires a thicker roof to resist additional reactions not present in the roof slabs of the other segments.

Similarly, a thicker mat slab is required in Segment 2 to resist additional moments and shears at the two large shear keys and to resist additional bearing pressures. The keys are required to resist soil dynamic and active pressures because over most of the length of this segment backfill is placed on only one side of the structure. In this segment there are unbalanced soil pressures, thus requiring shear keys to resist the lateral forces. Higher bearing pressures are placed on the mat slab as well due to overturning moments and a greater overall weight of this segment versus the other segments.

At the interface of two different segments, the interior wall, mat and slab surfaces line up evenly with the adjacent segments and any difference in top slab or bottom mat thicknesses only affects the outer dimensions of the ESWPT segments. This is the reason for the difference in elevations for the top of concrete for the ESWPT segments.

(b) Please refer to the response in part (a).

(c) The rebar shown in FSAR Figure 3.8-202 is a general depiction of reinforcement for the particular segment, and detailed information on the rebar size and quantities are not shown in the FSAR.

Detailed information regarding the rebar size and quantities will be included on the appropriate construction drawings after completion of detailed design.

Figures 3.8-202 through 3.8-205 and 3.8-210 have been revised to differentiate between the supply and return lines.

(d) Detailed information on the rebar size and quantities for the shear key are not intended to be shown in the FSAR. The reference to a shear key detail shown in FSAR Figure 3.8-202 is therefore deleted. The detailed information for the ESWPT shear key for Segment 2, showing rebar size and quantities, will be included on the appropriate construction drawings after completion of detailed design.

(e) The rebar in FSAR Figure 3.8-210 serves as a general depiction of reinforcement for the UHS basin. The design of the ESWPT reinforcement sizes, quantity, and spacing are subject to adjustment as details of the design are finalized, and are therefore not called out in the FSAR figure. Therefore, the reference to reinforcement details for the UHS shown in FSAR Figure 3.8-202 has been deleted. Detailed information regarding the rebar size and quantities will be included on the appropriate construction drawings after completion of the detailed design.

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

See attached marked-up FSAR Revision 1 pages 3.8-4, 3.8-5, 3.8-15, Figures 3.8-201, 3.8-202, 3.8-203, 3.8-204, 3.8-205, and 3.8-210.

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

None.

<u>Impact on DCD</u>

None.

The following structures are supported by the ESWPT as an integral part of the tunnel:

- Fuel/Pipe access tunnels, providing access from the PS/B to the PSFSVs are shown in Figures 3.8-204 and 3.8-212.

- Reinforced concrete air intake enclosures projecting above the ground for ESWS piping from the ESWS pump houses.

For details see Figures 3.8-202 through 3.8-205.

The modeling and analysis of the ESWPT is described in Appendix 3LL. RCOL2_03.08.04-1

The ESWPT is divided into three segments separated by expansion joints. A key plan showing the locations of the three segments is included in Figure 3.8-201. The segments are defined as follows:

- Tunnel Segment 1, as shown in Section G in FSAR Figure 3.8-203, is representative of the typical tunnel segments to the east and west of the R/B.

- Tunnel Segment 2, as shown in Section F and F' in FSAR Figure 3.8-202, is representative of segments adjacent to the Ultimate Heat Sink (UHS) structures. A tornado missile shield extends from the top of this segment to protect openings in the UHS.

- Tunnel Segment 3, as shown in Section H and H' in FSAR Figure 3.8-204 is representative of segments with fuel pipe access tunnels extending from the top. These are located adjacent to the PSFSVs.

Each segment has a somewhat different geometry and is designed separately. Segments 1 and 3 have roof slab and mat slab thicknesses of 2'-0" while Segment 2 has a roof slab and mat slab thickness of 2'-6".

All segments are designed for the same basic load conditions, but due to differing geometry the values of some of the loads (seismic, soil pressure, live loads, etc.) varied. The resulting moments and shears also varied. Thus, Segment 2 requires a thicker roof slab because this segment includes the tornado missile shield structure. This requires a thicker roof to resist additional reactions not present in the roof slabs of the other segments.

Similarly, a thicker mat slab is required in Segment 2 to resist additional moments and shears at the two large shear keys and to resist additional bearing pressures. The keys are required to resist soil dynamic and active pressures because over most of the length of this segment backfill is placed only on one side of the structure. In this segment there are unbalanced soil pressures, thus requiring shear keys to resist the lateral forces. Higher bearing pressures are placed on the mat slab as well due to overturning moments and a greater overall weight of this segment versus the other segments. RCOL2_03.08.04-1

It is intended that at the interface of two different segments, the interior wall, mat, and slab surfaces line up evenly with the adjacent segments and any difference in slab thicknesses affects only the outer dimensions of the ESWPT segments.

3.8.4.1.3.2 UHSRS

The UHSRS consists of a cooling tower enclosure; UHS ESW pump house, and UHS basin. All of them are reinforced concrete structures, described below.

UHS Basin - There are four basins for each unit and each reinforced concrete basin has one cooling tower with two cells. Each basin rests on a separate foundation, is square in shape, constructed of reinforced concrete, and separated from the adjacent basin by a minimum 4 inch expansion joint. A site-specific specification for the expansion/separation joint that provides material or system performance requirements will be prepared. Performance requirements for an elastomeric material include requirements bounding the allowable stress-strain properties, durability requirements, and specification for a material testing program. Each basin serves as a reservoir for the ESWS. An UHS ESW pump house is located at the south-west corner of each basin. Adjacent to the pump house on the east side of the basin are cooling tower enclosures supported by UHS basin walls. The ESWPT runs east-west along the south exterior wall of the UHS basin, and is separated by a minimum 4 inch expansion joint. RCOL2_03.08.04-2

Each basin is divided into two parts, as shown on Figure 3.8-206. The larger section of the basin shares the pump house and one cooling tower cell enclosure. The other cooling tower cell enclosure is in the smaller segment of the basin. A reinforced concrete wall, running east-west, separates the cooling tower enclosure basin area from rest of the basin. This wall is provided with slots to maintain the continuity of the reservoir.

See Figure 3.8-206 for general arrangement, layout, and dimensions of the UHSRS.

UHS ESW pump house - The pump house is an integral part of the UHS basin supported by UHS basin exterior and interior walls. Each pump house contains one ESW pump and one UHS transfer pump with associated auxiliaries. The pump bay (lowest portion of the pump house required for the pump suction) is deeper than the rest of the UHS basin. A reinforced concrete wall, running east-west, divides the pump house basin from rest of the UHS basin. This wall is provided with slots for flow of water. Two baffle walls (running east-west) are provided inside the pump house basin, before the pump bay. These baffle walls are provided with slots to maintain the flow of water and are staggered to prevent trajectory of postulated direct or deflected design basis tornado missiles.

The operating floor of the pump house is a reinforced concrete slab spanning east-west and supported by UHS basin exterior and interior walls. The operating

The 4 ft. depth exceeds the maximum depth of frost penetration at CPNPP.

3.8.5.1.3 Site-Specific Structures

CP COL 3.8(24) Replace the paragraph in DCD Subsection 3.8.5.1.3 with the following new subsections.

3.8.5.1.3.1 ESWPT

The ESPWT is an underground structure supported by a monolithic reinforced concrete basemat. The basemat is a 2 ft. thick concrete slab in Segments 1 and 3 as shown in Figures 3.8-203 and 3.8-204, respectively, and is 2'-6" thick adjacent to the UHSRS in Segment 2 as shown in Figure 3.8-202, with top and bottom reinforcement in each direction arranged in a rectangular grid. RCOL2_03.08.04-1

The bottom of the basemat is at elevation 791.08 ft. (elevation 790.58 ft. adjacent to the UHSRS), and is founded on structural concrete fill placed directly on limestone. The basemat has a shear key which extends into the fill concrete in the portion of ESWPT adjacent to the UHSRS as shown in Figure 3.8-202. The fill concrete at this portion also has a shear key which extends into the limestone as shown in Figure 3.8-202. Except at this portion where the fill concrete is locally reinforced, the fill concrete is generally designed as unreinforced concrete. RCOL2_03.08.04-1 RCOL2_03.08.05-1

3.8.5.1.3.2 UHSRS

The UHS basins, ESWS pump house, and the cooling towers are free-standing structures supported on a reinforced concrete basemat. Each basin, including its pump house and cooling towers, rests on a 4 ft. thick mat with top and bottom reinforcement in each direction arranged in a rectangular grid.

The bottom of the UHS basemat is at elevation 787 ft., except the pump house sump mat is at elevation 775 ft. The pump house basemat is founded directly on limestone, whereas the rest of the UHS mat is founded on structural concrete fill placed directly on limestone.

3.8.5.1.3.3 PSFSVs

PSFSVs are underground structures supported by a monolithic reinforced concrete basemat. The basemat is a 6'-6" thick concrete slab with top and bottom reinforcement in each direction arranged in a rectangular grid.

The bottom of the basemat is at elevation 782 ft., and is founded directly on limestone. Shear keys are provided which extend into the limestone as shown in Figures 3.8-213 and 3.8-214.

RCOL2_03.08.041

(SRI)

CP COL 3.8(19)

Figure 3.8-201 General Arrangement of ESWPT, UHSRS, and PSFSV

RCOL2_03.08.04-1

(SRI)

CP COL 3.8(19)
CP COL 3.8(24)

Figure 3.8-202 Typical ESWPT Sections Adjacent to UHS Basin with Cooling Water Air Intake Missile Shield Enclosure Supported by the Tunnel

RCOL2_03.08.04-1

(SRI)

CP COL 3.8(19)

Figure 3.8-203 Typical Section for ESWPT

RCOL2_03.08.04-1

(SRI)

CP COL 3.8(19)

Figure 3.8-204 Section of ESWPT at PS/B and PSFSVs Showing Fuel Pipe/Access Tunnel

RCOL2_03.08.04-1

(SRI)

CP COL 3.8(19)

Figure 3.8-205 Section of ESWPT at R/B and T/B Interface

RCOL2_03.08.04-1

(SRI)

CP COL 3.8(19)

Figure 3.8-210 Typical Section Looking West at UHS Basin and Cooling Tower Interface with ESWPT

Attachment 2

Response to Request for Additional Information No. 2994 (CP RAI #108)
Questions 03.08.04-2 through 03.08.04-17

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-2

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(19) in CPNPP COL FSAR, Subsection 3.8.4.1.3.2, the second paragraph (Page 3.8-6) states that each basin of the ultimate heat sink related structures (UHSRS) is separated from the adjacent basin by a minimum 4-inch expansion joint.

The applicant is requested to provide the following information:

(a) What is the material used for the 4-inch (minimum) expansion joints? How do the physical properties of these joints vary with aging over the 60-year life of the plant?

(b) How are the expansion joints modeled in the seismic structural analyses?

ANSWER:

A site-specific specification for the expansion/separation joint that provides performance requirements for material or system used will be prepared prior to the start of procurement. Performance requirements for an elastomeric material joint or sealer includes requirements bounding the allowable stress-strain properties, durability requirements, and specification for a material testing program.

(a) The material considered for the design of the UHSRS is ETHAFOAM 220 produced by Sealed Air Corporation. This material was considered because it has a tri-linear stress-strain curve. The initial stiff property allows placement of about a 10 ft concrete lift directly against the material while entering the second flat segment of the curve. During the seismic event, the material acts as an isolation gap since the stress-strain curve is flat beyond the strain levels induced during concrete placement. The separation joint material such as ETHAFOAM (or other approved material meeting specification requirements) is procured in accordance with the specifications, which includes the design and engineering requirements such as long term durability, material testing, and allowable stress-strain properties.

(b) The expansion joints are modeled as having complete separation from adjacent structures in seismic structural analyses. For these analyses only one structure is modeled, with the expansion joint modeled as a lack of soil or adjacent structure on the isolated sides. This is appropriate for the material considered in the joint.

FSAR Subsections 3.8.4.1.3 and 3.8.4.1.3.2 have been revised to incorporate this response.

Impact on R-COLA

See attached marked-up FSAR Revision 1 pages 3.8-3 and 3.8-5.

Impact on S-COLA

None.

Impact on DCD

None.

- Establish acceptability and compare measured lift-off values with predictions and minimum requirements.
- General visual inspection of all accessible concrete surface areas to assess the general structural condition of the containment.

3.8.4 Other Seismic Category I Structures

CP COL 3.8(15) Replace the fourth paragraph in DCD Subsection 3.8.4 with the following.

The ESWPT, UHSRS, and PSFSVs are site-specific seismic category I structures. These structures are discussed in detail in Subsection 3.8.4.1.3. No site-specific seismic category II structures are applicable at CPNPP.

3.8.4.1.3 ESWPT, UHSRS, PSFSVs, and Other Site-Specific Structures

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CP COL 3.8(19) Replace the second paragraph in DCD Subsection 3.8.4.1.3 with the following.

The ESWPT, UHSRS, and PSFSVs are designed to the site-specific SSE, and are described in detail in Subsections 3.8.4.1.3.1, 3.8.4.1.3.2, and 3.8.4.1.3.3, respectively. Figure 3.8-201 provides the general arrangement of ESWPT, UHSRS, and PSFSVs. <u>Each of these structures is separated from other structures with expansion/isolation joints as shown in various views in Figures 3.8-201 through 3.8-214. The performance specifications for the elastomeric joint or seal materials address requirements for critical characteristics such as bounding the allowable stress-strain properties, durability requirements, and associated material testing.</u> RCOL2_03.08.04-2

3.8.4.1.3.1 ESWPT

The ESWPT is an underground reinforced concrete structure. Figure 3.8-203 shows the typical section of the ESWPT. The tunnel layout is a rectangular configuration forming a closed looped structure starting at the UHS Basins and terminating at the T/B. The outside dimensions of the tunnel are shown in Figure 3.8-203. The tunnel is divided into two sections by an interior concrete wall to provide separation of piping trains. Each section contains both ESWS supply and return lines. End walls are also provided where required to maintain train separation. The top of the tunnel is approximately 12.25 ft. below grade. Access to the tunnel is provided by reinforced concrete manholes.

mat slab as well due to overturning moments and a greater overall weight of this segment versus the other segments. RCOL2_03.08.04-1

It is intended that at the interface of two different segments, the interior wall, mat, and slab surfaces line up evenly with the adjacent segments and any difference in slab thicknesses affects only the outer dimensions of the ESWPT segments.

3.8.4.1.3.2 UHSRS

The UHSRS consists of a cooling tower enclosure; UHS ESW pump house, and UHS basin. All of them are reinforced concrete structures, described below.

UHS Basin - There are four basins for each unit and each reinforced concrete basin has one cooling tower with two cells. Each basin rests on a separate foundation, is square in shape, constructed of reinforced concrete, and separated from the adjacent basin by a minimum 4 inch expansion joint. A site-specific specification for the expansion/separation joint that provides material or system performance requirements will be prepared. Performance requirements for an elastomeric material include requirements bounding the allowable stress-strain properties, durability requirements, and specification for a material testing program. Each basin serves as a reservoir for the ESWS. An UHS ESW pump house is located at the south-west corner of each basin. Adjacent to the pump house on the east side of the basin are cooling tower enclosures supported by UHS basin walls. The ESWPT runs east-west along the south exterior wall of the UHS basin, and is separated by a minimum 4 inch expansion joint. RCOL2_03.08.04-2

Each basin is divided into two parts, as shown on Figure 3.8-206. The larger section of the basin shares the pump house and one cooling tower cell enclosure. The other cooling tower cell enclosure is in the smaller segment of the basin. A reinforced concrete wall, running east-west, separates the cooling tower enclosure basin area from rest of the basin. This wall is provided with slots to maintain the continuity of the reservoir.

See Figure 3.8-206 for general arrangement, layout, and dimensions of the UHSRS.

UHS ESW pump house - The pump house is an integral part of the UHS basin supported by UHS basin exterior and interior walls. Each pump house contains one ESW pump and one UHS transfer pump with associated auxiliaries. The pump bay (lowest portion of the pump house required for the pump suction) is deeper than the rest of the UHS basin. A reinforced concrete wall, running east-west, divides the pump house basin from rest of the UHS basin. This wall is provided with slots for flow of water. Two baffle walls (running east-west) are provided inside the pump house basin, before the pump bay. These baffle walls are provided with slots to maintain the flow of water and are staggered to prevent trajectory of postulated direct or deflected design basis tornado missiles.

The operating floor of the pump house is a reinforced concrete slab spanning east-west and supported by UHS basin exterior and interior walls. The operating

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-3

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(19) in CPNPP COL FSAR, Subsection 3.8.4.1.3.2, "UHSRS," the paragraph at the top of Page 3.8-7 states that "Air intakes are located at the north and south faces of the enclosure and configured to protect the safety-related substructures and components from tornado missiles. The north side air intake is an integral part of the cooling tower enclosure, whereas the south side air intake is an integral part of the ESWPT."

The applicant is requested to:

(a) List the safety-related substructures and components that are protected from tornado missiles.

(b) Referring to Figure 3.8-210, where it shows the air intake, explain how objects are prevented from falling into and blocking the air intake.

ANSWER:

(a) FSAR Subsection 3.8.4.4.3.2, third paragraph, states "Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles." DCD Subsection 3.3.2.2.3 provides a discussion of tornado missile effects and FSAR Subsection 3.3.2.2.4 provides a discussion of combined tornado effects for site-specific seismic category I structures.

FSAR Table 3.2-201 lists the site-specific equipment and components located in the UHSRS that are protected from tornado missiles by the UHSRS.

FSAR Subsection 3.8.4.1.3.2 has been revised to incorporate an appropriate reference to the safety-related components in Table 3.2-201 that are protected from tornado missile impacts and to clarify the statement quoted in the question above.

(b) The air intake structures for the UHS are designed to protect against tornado generated missiles. The configuration of these openings does not explicitly prevent objects from falling into them. However, for each cell of a UHS, the openings on the north and south sides are 7 ft. wide by 45 ft. long as shown in FSAR Figures 3.8-206 and 3.8-210. This is a large opening and full or partial restriction of air flow to the cooling tower cell is not likely to occur.

The UHS and ESW systems support normal operations as well as transient and accident modes of operation. Any blockage of the cooling tower air intakes would be identified during periodic maintenance/inservice inspection and the debris would be removed. Any significant increase in blockage that might occur during normal operations would be detectable by a decreased efficiency in the cooling capability of the cooling tower, such as high temperature alarms, and corrected accordingly.

The UHS design provides redundancy to accommodate a postulated single failure of a cooling tower. The UHS requires a minimum of two cooling towers to maintain a safe plant shutdown condition. In addition, any unacceptable debris blockage of the air intakes associated with tornado-generated debris would be a short duration event that would be corrected during the post-DBT inspection and remediation efforts. Therefore, blockage of the air intake would not impact the UHS' ability to perform safety its functions.

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

See attached marked-up FSAR Revision 1 page 3.8-6.

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

None.

<u>Impact on DCD</u>

None.

floor supports the ESWS pump, UHS transfer pump, and motors. The roof of the pump house is a reinforced concrete slab spanning north-south and supported by reinforced concrete beams. To allow access to the ESWS pump/motor, a removable reinforced concrete cover is provided in an opening in the roof of the pump house.

Tornado missile shields are provided to protect the air intake and air outlets of the ESWS pump house HVAC system from tornado missiles. The structural design considers tornado differential pressure loads as discussed in Subsection 3.3.2.2.2. RCOL2_09.04.05-4

UHS cooling tower enclosures - Each UHS basin has one cooling tower with two cells. Each cell is enclosed by reinforced concrete structures that house the equipment required to cool the water for ESWS. The reinforced concrete wall running north-south separates the two cell enclosures. The enclosures are an integral part of the UHS basin supported by the basin interior and exterior walls on the basemat foundation. A reinforced concrete wall, running east-west, separates the cell enclosure portion of the basin from the rest of the UHS basin. An east-west wall is provided with openings at the basemat to maintain the continuity of the UHS basin. Air intakes are located at the north and south faces of the cooling tower enclosure. The missile shields at the air intakes are ~~and~~ configured to protect the safety-related substructures and components housed within the UHS structure from tornado missiles. FSAR Table 3.2-201 lists the site-specific equipment and components located in the UHSRS that are protected from tornado missiles. The north side air intake is an integral part of the cooling tower enclosure, whereas the south side air intake is an integral part of the ESWPT, and is supported by reinforced concrete piers which are supported by the ESWPT walls and basemat. RCOL2_03.08.04-3

Each cooling tower cell enclosure is equipped with a fan and associated equipment to cool the water. Equipment includes header pipe, spray nozzles, and drift eliminators with associated reinforced concrete beams supported by the exterior walls of the enclosure. The fan and motor are supported by reinforced concrete deck above the drift eliminators. A circular opening is provided in the deck for the fan, and the deck is supported by enclosure walls and a deep upside circular concrete beam around the fan opening. The fan is supported by a north-south concrete beam at the center of enclosure. For air circulation and to protect the fan and motor from tornado missiles, a circular opening is provided at the roof of the enclosure (centered on the fan) with a reinforced concrete slab and heavy steel grating between the roof and the deck.

For details see Figures 3.8-207 through 3.8-211 for the UHS basin, UHS ESW pump house and cooling tower enclosures. Details of the UHSRS seismic analysis are provided in Appendix 3KK. RCOL2_03.08.04-12

3.8.4.1.3.3 PSFSVs

The PSFSVs are underground reinforced concrete structures required to house the safety-related and non safety-related fuel oil tanks. There is one vault for each

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-4

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(19) in CPNPP COL FSAR, Subsection 3.8.4.1.3.3, "PSFSVs," the 1st paragraph (Page 3.8-7) describes the reinforced-concrete underground vaults used to house the safety-related and non-safety-related fuel oil tanks.

The applicant is requested to address whether seismic analysis has been performed with varying levels of fuel oil in the fuel oil storage tanks. Provide a description of such an analysis. If there is no analysis, provide the rationale as to why this condition is not important to the seismic analyses.

ANSWER:

FSAR Appendix MM has been revised to reflect that the three tanks were considered to be rigid and full with a total weight of 1,155 kip each. The tanks are modeled by stiff beam elements that are connected to the 6'-6" thick base slab. The base slab is placed on the limestone. Varying levels of fuel oil in the fuel oil storage tanks were not considered because:

1. The power fuel storage vaults are supposed to be kept full prior to an emergency such as an SSE, therefore, full tanks is the normal operating fuel level for the tanks.
2. The SSI analyses performed in SASSI demonstrated that the design input response spectra at the top of limestone and the in-structure spectra at the top of the base slab are nearly the same indicating that the SSI effects are not large. The SSI analyses were used to determine maximum accelerations for a range of soil conditions representing the uncertainty in soil properties.
3. Since the tanks are assumed rigid, the tank seismic inertial forces applied to the base slab were obtained by equivalent static analysis using lateral seismic accelerations of 0.25g which is more

than two times the base slab ZPA acceleration (0.12g). The design acceleration was increased from the base slab ZPA acceleration in order to estimate the potential increase in demands due to hydrodynamic effects. For the global design of the PSFSV, a lower mass in the tank with the inclusion of sloshing effects is expected to result in lower design forces and was therefore not considered.

For the detailed design, the steel tank properties are specified and confirmed during the procurement process, and seismic behavior including hydrodynamic effects are performed to design tank supports, tank support attachments to the slab, and local reinforcement in the tank slab.

FSAR Subsections 3.8.4.1.3.3 and 3.8.4.4.3.3 have been revised to refer to details of the PSFSV seismic analysis in FSAR Appendix 3MM. FSAR Section 3MM.2 and Table 3MM-2 have been revised to incorporate this response.

Impact on R-COLA

See attached marked-up FSAR Revision 1 pages 3.8-7, 3.8-12, 3MM-1, 3MM-2, and 3MM-9.

Impact on S-COLA

None.

Impact on DCD

None.

PS/B. The vault contains two safety-related and one non safety-related oil tanks. Each tank is contained in a separate compartment. Compartments are separated by reinforced concrete walls. A common mat supports the tanks and the rest of the vault. The PSFSV roof slab is sloped to facilitate drainage. The highest point of the roof slab is slightly above grade. Bollards and a concrete curb are provided to prevent vehicular traffic on the roof.

Access to each vault is provided by a reinforced concrete tunnel from the applicable PS/B. Each tank compartment has a separate pipe/access tunnel, which is an integral part of the ESWPT.

For vault details see Figures 3.8-212 through 3.8-214. Details of the PSFSV seismic analysis are provided in Appendix 3MM. RCOL2_03.08.04-4

3.8.4.1.3.4 Other Site-Specific Structures

Additional seismic category I structures are not identified at this time. Other site-specific seismic category I structures, if required, are analyzed and designed in a manner similar to all other site-specific seismic category I structures. The applied loadings, including seismic loadings are discussed in FSAR and DCD Subsections 3.8.4.3. RCOL2_03.08.04-5

If required, ~~S~~site-specific seismic category I yard piping and conduits ~~are~~may be routed within reinforced concrete duct banks (solid) or reinforced concrete chases (hollow). The duct banks and chases have shallow embedments and are buried partially or wholly below grade within structurally engineered and compacted backfill that extends down to top of limestone at nominal elevation 782 ft. The duct banks and chases are designed for appropriate vehicle and equipment surcharge loads. The duct banks and pipe chases ~~are~~may be constructed in segments, which are separated from each other and other structures by expansion, or contraction joints. Expansion and contraction joints are placed in the duct banks and chases to control cracking due to thermal expansion or shrinkage. RCOL2_03.08.04-5 RCOL2_03.08.04-5

~~The expansion~~Isolation joints are utilized at the interface with other structures to accommodate all anticipated differential settlement and movement (due to seismic and other loading) at support points, penetrations, and entry points into other structures. Isolation joints are required to be sized using criteria given in Subsection 3.7.2.8. Structural adequacy is ensured at the joints by appropriately sizing the joints and by properly sealing the joints to minimize potential for water intrusion. RCOL2_03.08.04-5

For purposes of ductbank/chase structural design, geotechnical properties of the backfill, such as static deformation modulus E_S and Poisson's ratio μ, are determined based on the actual source of the backfill. The modulus of subgrade reaction (k_S) used for beam-on-elastic foundation analyses of shallow-embedded ductbank/chases depends in part on ductbank/chase width (B). The modulus of subgrade reaction is calculated using the following formula:

$$k_s = E_s / [(B)(1 - \mu^2)]$$

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5.

RCOL2_03.0 8.04-16
RCOL2_03.0 8.04-13
RCOL2_03.0 8.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above. RCOL2_03.0 8.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM. RCOL2_03.0 8.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are RCOL2_03.0 8.04-14

3MM MODEL PROPERTIES AND SEISMIC ANALYSIS RESULTS FOR PSFSVs

3MM.1 Introduction

This Appendix discusses the seismic analysis of the power source fuel storage vaults (PSFSVs). The computer program SASSI (Reference 3MM-1) serves as the platform for the soil-structure interaction (SSI) analyses. The three-dimensional (3D) finite element (FE) models used in the SASSI are condensed from FE models with finer mesh patterns initially developed using the ANSYS computer program (Reference 3MM-2). Further, the translation of the model from ANSYS to SASSI is confirmed by comparing the results from the modal analysis of the fixed base structure in ANSYS and the SASSI analysis of the model resting on a half-space with high stiffness. The close correlation between the SASSI transfer function results with the ANSYS eigenvalues results ensures the accuracy of the translation.

The SASSI 3D FE model is dynamically analyzed to obtain seismic results including SSI effects. The SASSI model results including seismic soil pressures are used as input to the ANSYS models for performing the detailed structural design including loads and load combinations in accordance with the requirements of Section 3.8. The Table 3MM-8 summarizes the analyses performed for calculating seismic demands. The SASSI analysis and results presented in this Appendix include site-specific effects such as the layering of the subgrade, embedment of the PSFSVs, flexibility of the basemat and subgrade, and scattering of the input control design motion. Due to the low seismic response at the Comanche Peak Nuclear Power Plant site and lack of high-frequency exceedances, the SASSI capability to consider incoherence of the input control motion is not implemented in the design of the PSFSVs. RCOL2_03.07.02-16

3MM.2 Model Description and Analysis Approach

The SASSI FE model for the PSFSV is shown in Figure 3MM-1. Table 3MM-1 presents the properties assigned to the structural components of the SASSI FE model. Table 3MM-2 summarizes the SASSI FE model structural component dimensions and weights. Detailed descriptions and figures of the PSFSV are contained in Section 3.8. RCOL2_03.07.02-16

The PSVSV is a simple shear wall structure with four exterior walls plus two interior shear walls. The walls must resist the out of plane flexure and shear due to transverse accelerations, soil pressures (for exterior walls) and flexure imparted on the wall from flexure in the roof slab. The roof slab resists vertical seismic demands as a continuous three span plate although there is some two-way response. Critical locations are therefore centers and edges of roof slabs and walls for flexure and bottom of walls for in-plane shear. RCOL2_03.07.02-16

Shell elements are used for the roof, interior and exterior walls, brick elements are used for the base mat, and stiff beam elements are used to represent the RCOL2_03.08.04-4 emergency power fuel oil tanks and their supports, which are connected to the basemat. The three tanks are considered to be rigid, and full with a total weight of 1155 kips each, which corresponds to the normal operating fuel level. The steel tank mass and stiffness properties, and seismic behavior including hydrodynamic effects, are considered in the design of tank supports, tank support attachments to the slab, and local reinforcement in the tank slab. Walls are modeled using gross section properties at the centerline. The tapered east wall of the vault is modeled at the centerline of the top portion of the wall. The change in thickness is modeled using the average thickness of the wall at each element layer. RCOL2_03.08.04-4 RCOL2_03.07.02-16

The materials and properties of the roof slab are changed to reflect the cracked concrete properties for out of plane bending. The cracked concrete properties are modeled for one-half of the uncracked flexural stiffness of the roof. Un-cracked properties are considered for the in-plane stiffness ~~and the mass of the roof~~ (Reference 3MM-3). Therefore, to achieve 1/2 flexural out-of-plane stiffness of the slab without reducing its in-plane stiffness or mass, the following element properties are assigned: RCOL2_03.07.02-16 RCOL2_03.07.02-16

$$t_{cracked} = (C_F)^{0.5} \cdot t$$

$$E_{cracked} = [1/(C_F)^{0.5}] \cdot E_{concrete}$$

$$\gamma_{cracked} = [1/(C_F)^{0.5}] \cdot \gamma_{concrete}$$

where:

C_F = the factor for the reduction of flexural stiffness, taken as 1/2,

$t_{cracked}$ = the effective slab thickness to account for cracking

t = the gross section thickness

$\gamma_{cracked}$ = the effective unit weight to offset the reduced stiffness and provide the same total mass

$\gamma_{concrete}$ = unit weight of concrete

$E_{cracked}$ = effective modulus to account for the reduction in thickness that keeps the same axial stiffness while reducing the flexural stiffness by C_F

$E_{concrete}$ = modulus of elasticity of concrete.

The analysis of the PSFSV produces 50 modes below 45 Hz. The natural frequencies and descriptions of the associated modal responses of the fixed-base model are presented in Table 3MM-3 for the PSFSV and these frequencies are RCOL2_03.07.02-16

Table 3MM-2

SASSI FE Model Component Dimensions and Weights[1]

FE Component	Slab Width or Wall Height (ft)	Slab or Wall Length (ft)	Slab or Wall Thickness (ft)	Weight (kips)
North Exterior Wall	40	83.5	2.5	1,330
South Exterior Wall	40	83.5	2.5	1,420
West Exterior Wall	40	75.5	2.5	1,284
East Exterior Wall	40	75.5	Varies from 4.5 at bottom to 2.5 at top	1,926
West Interior Wall	40	75.5	1.5	982
East Interior Wall	40	75.5	1.5	982
Roof Slab	83.5 (east-west)	75.5 (north-south)	2[2]	2,206
Base mat	83.5 (east-west)	75.5 (north – south)	6.5	6,462
Tanks including full fuel oil content	N/A	N/A	N/A	~~1,162~~1,155 x 3 = ~~3,486~~3,465
Total Weight	40	83.5	2.5	~~20,078~~20,057
Equivalent Weight (ksf) on Slab Area (78'x88')				2.9
Peak Dynamic Pressure[3] (ksf)				2.2

RCOL2_03.08.04-4

Notes:

1) The width and length dimensions in the table have been adjusted from actual dimensions to suit the mesh pattern used for the FE model. The adjustments are minor and do not affect the accuracy of the analysis results. Actual component dimensions are shown in Section 3.8 Figures 3.8-212, 3.8-213, and 3.8-214.
2) The actual roof slab thickness of 2 ft is adjusted to 1.414 ft in the FE model to account for its cracked properties, as discussed in Appendix Subsection 3MM.2.
3) Peak dynamic pressure at corner elements, each representing less than 1 percent of the slab area, are as high as 4.1 ksf. Average peak pressure over total slab area is 0.7 ksf.

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-5

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

CP COL 3.8(19) in CPNPP COL FSAR, Subsection 3.8.4.1.3.4, "Other Site-Specific Structures," states "Site-specific seismic category I yard piping and conduits are routed within reinforced concrete duct banks (solid) or reinforced concrete chases (hollow). The duct banks and chases have shallow embedments and are buried partially or wholly below grade within structurally engineered and compacted backfill that extends down to top of limestone at nominal elevation 782 ft. The duct banks and pipe chases are constructed in segments, which are separated from each other and other structures by expansion joints. The expansion joints accommodate all anticipated differential settlement and movement (due to seismic and other loading) at support points, penetrations, and entry points into other structures."

The description of the buried piping and conduits is fairly general and lacks specific descriptions of the details of the design. The applicant is requested to:

(a) Provide details on the concrete enclosures, including cross-section views that show steel reinforcing.

(b) Provide details on the analyses performed to assure the safety-related function of the buried piping and conduits under all loadings, including seismic loads.

(c) Provide a detailed description of the expansion joints used between the segments of the buried ducts and banks, and explain how the structural adequacy of the reinforced ducts and banks is assured at these joints.

(d) Assuming that the analyses were based on beams on elastic foundations, describe how the foundation modulus for beams on the elastic foundations was calculated or otherwise obtained.

(e) Procedures for the design of restrained underground piping of American Society of Mechanical Engineers (ASME) B31.1 provide guidance for the thermal loading of the buried ducts and banks. Address whether this guidance was used for the design of buried utilities for the CPNPP. If not, explain the rationale for not following the ASME B31.1 guidance.

(f) Describe the properties of the engineered backfill and how the reconciliation of the as-built properties with the design values is to be accomplished.

ANSWER:

Seismic category I shallow-embedded duct banks and chases are included in FSAR Chapter 3 in the anticipation that such items may be needed, but the need for these designs will be confirmed as detailed electrical, mechanical, and piping commodities design and yard layout progresses. FSAR Subsection 3.8.4.1.3.4 has been revised to clarify this.

(a) Details on theses concrete enclosures, including cross-section views will be available as the design progresses.

(b) Shallow-embedded duct banks and chases are analyzed in a manner similar to all other site-specific seismic category I structures. The applied loadings, including seismic loadings are discussed in FSAR and DCD Subsections 3.8.4.3. Shallow-embedded duct banks and chases are also designed for appropriate vehicle and equipment surcharge loads.

(c) Expansion and contraction joints are placed in the duct banks to control cracking due to thermal expansion or shrinkage.

Isolation joints are utilized at the interfaces with other structures to mitigate the effects of differential movements due to thermal or seismic loads, or differential settlement. Isolation joints are required to be sized using criteria given in DCD Subsection 3.7.2.8, which is incorporated by reference in the FSAR.

Structural adequacy is ensured at the joints by appropriately sizing the joints and by properly sealing the joints to minimize potential for water intrusion. The detailed design for expansion and isolation joints incorporates details such as expansion loops, roller supports, sleeves, or other devices that keep demands within the encased or supported systems below their capacities while accommodating the expected movement.

(d) FSAR Subsection 2.5.4.5.4.1.1 discusses backfill material. The subgrade modulus of the structural backfill for shallow-embedded ductbank/chase design is calculated based on the soil deformation modulus and Poisson's ratio, and as a function of ductbank/chase width. Geotechnical properties of the fill are determined after the backfill source is established. Based on the requirements in the FSAR Subsection 2.5.4.5.4.1.1, conservative ranges of geomechanical properties are estimated at this time for preliminary analyses discussed in (f) below. The modulus of subgrade reaction (k_s) used in beam on elastic foundation analyses is calculated for the specific width (B) of each ductbank/chase using the following formula (see reference below):

$$k_s = E_s / [(B)(1 - \mu^2)]$$

Where E_s is the soil (static) deformation modulus and μ is the Poisson's ratio. For example, given the ranges of E_s listed in answer (f) and assuming $\mu = 0.35$, the modulus of subgrade reaction for a 6-ft wide ductbank/chase ranges between 90 pci and 220 pci. The most conservative outcome resulting from this range is used in preliminary structural design.

(e) Thermal loadings on the encased or supported systems conform to applicable portions of ASME B31.1, considering the effect of soil cover.

(f) FSAR Subsection 2.5.4 discusses the excavation and backfill processes, including backfill material. Structural backfill is required to be granular (less than 30% fines), free of organic matters, with less than 15% by weight particles larger than 2.5 in., non-expansive and compacted at 95% (Modified Proctor Test) relative compaction. As there is a wide range of soil types that meet these requirements (from clayey or silty sands and gravels to well graded sandy gravels to gravels, the geotechnical properties of the structural backfill can only be determined after the structural backfill source is established. Based on the requirements in FSAR Subsection 2.5.4.5.4.1.1, the following (conservative) ranges of geomechanical properties are estimated at this time for preliminary analyses:

1. Dynamic properties (low strain): Shear wave velocity (Vs) and Poisson's ratio (μ) as per Table 2.5.2-227 of the FSAR. Compression wave velocity calculated as:

$$Vp = Vs\sqrt{2 \cdot \frac{1-\mu}{1-2\mu}}$$

The dynamic moduli for small strains, E and G, are calculated based on *Vp* and *Vs*, respectively. The dynamic moduli and damping coefficients of the structural backfill considered in seismic analyses is compatible with the strains induced by the design input motion as discussed in FSAR Appendix 3NN. In addition, the response to RAI No. 2897 (CP RAI #60) Question 03.07.02-2 attached to Luminant letter TXNB-09073 dated November 24, 2009 (ML093340447) provides a discussion of how strain-compatible properties of the backfill are obtained and includes a table that presents those properties.

2. Static loading properties (high strain):
   - Friction angle at failure between 33° and 40°
   - Poisson's ratio between 0.2 and 0.35
   - Deformation modulus between 800 ksf and 2000 ksf
   - Total unit weight between 115 pcf and 135 pcf
   - Sliding coefficient (vs. concrete) between 0.4 and 0.55

The preliminary design is performed for the above ranges of soil parameters, and the most conservative solutions are selected.

Project specifications and testing requirements discussed further in FSAR Subsection 2.5.4.5.4 are used to control the installed backfill properties. Testing requirements for backfill include routine pre-construction (pre-installation) mechanical and index testing to perform traditional quality control testing on physical characteristics (such as grain size distribution, maximum dry unit weight, optimum moisture content, etc), and in-situ shear wave velocity testing performed post-construction. The response to RAI No. 3006 (CP RAI #122) Question 03.08.04-52, to be submitted no later than December 21, 2009, provides further discussion on the testing methods of the engineered backfill for CPNPP Units 3 and 4. In general, the ranges of backfill geomechanical properties selected for preliminary analyses are quite large. If testing determines that backfill properties fall outside the range of the analyses, then corrective action on the backfill placement is required and/or the analyses are revised as required.

FSAR Subsection 3.8.4.1.3.4 has been revised to incorporate this response.

Reference

Bowles, Joseph E., "Foundation Analysis and Design," 4th Ed., McGraw-Hill, Inc., New York, NY

Impact on R-COLA

See attached marked-up FSAR Revision 1 pages 3.8-7 and 3.8-8.

Impact on S-COLA

None.

Impact on DCD

None.

PS/B. The vault contains two safety-related and one non safety-related oil tanks. Each tank is contained in a separate compartment. Compartments are separated by reinforced concrete walls. A common mat supports the tanks and the rest of the vault. The PSFSV roof slab is sloped to facilitate drainage. The highest point of the roof slab is slightly above grade. Bollards and a concrete curb are provided to prevent vehicular traffic on the roof.

Access to each vault is provided by a reinforced concrete tunnel from the applicable PS/B. Each tank compartment has a separate pipe/access tunnel, which is an integral part of the ESWPT.

For vault details see Figures 3.8-212 through 3.8-214. Details of the PSFSV seismic analysis are provided in Appendix 3MM. RCOL2_03.08.04-4

3.8.4.1.3.4 Other Site-Specific Structures

Additional seismic category I structures are not identified at this time. Other site-specific seismic category I structures, if required, are analyzed and designed in a manner similar to all other site-specific seismic category I structures. The applied loadings, including seismic loadings are discussed in FSAR and DCD Subsections 3.8.4.3. RCOL2_03.08.04-5

If required, ~~S~~site-specific seismic category I yard piping and conduits ~~are~~may be routed within reinforced concrete duct banks (solid) or reinforced concrete chases (hollow). The duct banks and chases have shallow embedments and are buried partially or wholly below grade within structurally engineered and compacted backfill that extends down to top of limestone at nominal elevation 782 ft. The duct banks and chases are designed for appropriate vehicle and equipment surcharge loads. The duct banks and pipe chases ~~are~~may be constructed in segments, which are separated from each other and other structures by expansion, or contraction joints. Expansion and contraction joints are placed in the duct banks and chases to control cracking due to thermal expansion or shrinkage. RCOL2_03.08.04-5 RCOL2_03.08.04-5

~~The expansion~~Isolation joints are utilized at the interface with other structures to accommodate all anticipated differential settlement and movement (due to seismic and other loading) at support points, penetrations, and entry points into other structures. Isolation joints are required to be sized using criteria given in Subsection 3.7.2.8. Structural adequacy is ensured at the joints by appropriately sizing the joints and by properly sealing the joints to minimize potential for water intrusion. RCOL2_03.08.04-5

For purposes of ductbank/chase structural design, geotechnical properties of the backfill, such as static deformation modulus E_s and Poisson's ratio μ, are determined based on the actual source of the backfill. The modulus of subgrade reaction (k_s) used for beam-on-elastic foundation analyses of shallow-embedded ductbank/chases depends in part on ductbank/chase width (B). The modulus of subgrade reaction is calculated using the following formula:

$$k_s = E_s/[(B)(1-\mu^2)]$$

The dynamic properties of the backfill considered in seismic analyses of duct banks and chases are compatible with the strains induced by the design input motion as discussed in Appendix 3NN. RCOL2_03.08.04-5

Project specifications and testing requirements discussed in Subsection 2.5.4.5.4 are used to control the installed backfill properties. Testing requirements for backfill include routine pre-construction (pre-installation) mechanical and index testing to perform traditional quality control testing on physical characteristics (such as grain size distribution, maximum dry unit weight, optimum moisture content, etc), and in-situ shear wave velocity testing performed post-construction.

3.8.4.3 Loads and Load Combinations

CP COL 3.8(20) Replace the second paragraph in DCD Subsection 3.8.4.3 with the following.

Externally generated loads from the following postulated site-specific sources are evaluated in the following subsections:

- Subsection 2.4.2.3 concludes no loads induced by floods are applicable.
- Subsection 3.5.1.6 concludes no loads from non-terrorism related aircraft crashes are applicable.
- Subsection 2.2.3.1.1 concludes no explosive hazards in proximity to the site are applicable, and
- Subsection 3.5.1.6 concludes no projectiles and missiles generated from activities of nearby military installations are applicable.
- Subsection 3.7.1.1 provides the safe-shutdown earthquake response spectra used in the site-specific seismic design. RCOL2_03.08.04-6
- Subsection 3.3.1.1 provides the site-specific design wind speed.

3.8.4.3.4.2 Roof Snow Loads and Roof Live Loads

Add the following paragraph as the last paragraph in DCD Subsection 3.8.4.3.4.2:

The extreme winter precipitation roof load considered for site-specific seismic category I buildings and structures is 37.8 psf as given in Table 2.0-1R. The roof live load used for design of site-specific seismic category I buildings and structures is 100 psf minimum. RCOL2_03.08.04-6

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-6

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(20) in CPNPP COL FSAR, Subsection 3.8.4.3, "Loads and Load Combinations" (Page 3.8-8), the applicant is requested to provide the following information:

(a) Describe the safe-shutdown earthquake (SSE) response spectra used in the design.

(b) Provide a table comparing the loads considered in CPNPP that are different from those considered in US-APWR DCD (such as rain, snow, etc).

(c) Provide information on loads due to tornado-generated missiles, or, if not considered, the rationale for not including these loads.

ANSWER:

(a) FSAR Subsection 3.7.1.1 provides the safe-shutdown earthquake response spectra used in the site-specific seismic design.

(b) The following loads are different than those considered in the DCD:

- Safe Shutdown Earthquake (E_{ss}):
  The site-specific safe-shutdown earthquake response spectra described in FSAR Subsection 3.7.1.1 are used in the design of site-specific structures such as the UHSRS, PSFSVs, and ESWPT. The US-APWR standard plant design of the R/B complex and PS/Bs is based on a site-independent safe-shutdown earthquake represented by the certified seismic design response spectra, described in DCD Subsection 3.7.1.1.

- Design Wind (W):

  The site-specific design wind speed is taken as 90 mph in accordance with FSAR Subsection 3.3.1.1. The US-APWR standard plant design is based on a design wind speed of 155 mph, described in DCD Subsection 3.3.1.1.

- Roof Snow Load:

  The site-specific extreme winter precipitation roof load is 37.8 psf as given in FSAR Table 2.0-1R. The US-APWR standard plant design described in DCD Subsection 3.8.4.3.4.2 considers an extreme winter precipitation roof load of 75 psf.

- Roof Live Load:

  The roof live load used for site-specific seismic category I structures is 100 psf minimum. The US-APWR standard plant design described in DCD Subsection 3.8.4.3.4.2 considers a roof live load of 40 psf.

For purposes of global seismic analysis, live load mass was taken as 25% of the live load, which conforms to the requirements of the DCD and SRP 3.7.2. Increases in live load mass up to 50% of the live load for individually loaded members as cited in DCD Subsection 3.8.4.3.6.2 were not used in the global seismic analyses of the PSFSVs, ESWPTs, and UHSRS. However these load increases are required to be included in the detailed design of individual members, where applicable, as detailed structural design is performed.

(c) DCD Subsection 3.5.1.4, which is incorporated by reference into the FSAR, provides the tornado-generated missile spectrum that is consistent with the missile spectrum in RG 1.76, Revision 1. FSAR Subsection 3.3.2.2.4 states that site-specific category I structures are designed for the same tornado loadings and combined tornado effects using the same methods for qualification described for standard plant SSCs.

FSAR Subsection 3.8.4.3 has been revised to incorporate this response.

Impact on R-COLA

See attached marked-up FSAR Revision 1 page 3.8-8.

Impact on S-COLA

None.

Impact on DCD

None.

The dynamic properties of the backfill considered in seismic analyses of duct banks and chases are compatible with the strains induced by the design input motion as discussed in Appendix 3NN. RCOL2_03.08.04-5

Project specifications and testing requirements discussed in Subsection 2.5.4.5.4 are used to control the installed backfill properties. Testing requirements for backfill include routine pre-construction (pre-installation) mechanical and index testing to perform traditional quality control testing on physical characteristics (such as grain size distribution, maximum dry unit weight, optimum moisture content, etc), and in-situ shear wave velocity testing performed post-construction.

3.8.4.3 Loads and Load Combinations

CP COL 3.8(20) Replace the second paragraph in DCD Subsection 3.8.4.3 with the following.

Externally generated loads from the following postulated site-specific sources are evaluated in the following subsections:

- Subsection 2.4.2.3 concludes no loads induced by floods are applicable.
- Subsection 3.5.1.6 concludes no loads from non-terrorism related aircraft crashes are applicable.
- Subsection 2.2.3.1.1 concludes no explosive hazards in proximity to the site are applicable, and
- Subsection 3.5.1.6 concludes no projectiles and missiles generated from activities of nearby military installations are applicable.
- Subsection 3.7.1.1 provides the safe-shutdown earthquake response spectra used in the site-specific seismic design. RCOL2_03.08.04-6
- Subsection 3.3.1.1 provides the site-specific design wind speed.

3.8.4.3.4.2 Roof Snow Loads and Roof Live Loads

Add the following paragraph as the last paragraph in DCD Subsection 3.8.4.3.4.2:

The extreme winter precipitation roof load considered for site-specific seismic category I buildings and structures is 37.8 psf as given in Table 2.0-1R. The roof live load used for design of site-specific seismic category I buildings and structures is 100 psf minimum. RCOL2_03.08.04-6

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-7

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.1, "ESWPT," the first paragraph (Page 3.8-8) states that "The ESWPT is designed to withstand the loads specified in Subsection 3.8.4.3."

ESWPT is an underground structure. The applicant is requested to address the issue, "Is there any surcharge pressure on the ground surface considered in the design?" If yes, specify the surcharge pressure. If not, explain why, and describe what measures or safeguards are taken to avoid any surcharge loading.

ANSWER:

A design surcharge pressure of 600 psf is applied to tunnel Segments 1 and 2 representing the tunnels running north-south and tunnels adjacent to the UHS structures, respectively. This surcharge is based on assumed cask mover and external fuel cask storage pad loads.

A design surcharge pressure of 200 psf is applied to tunnel Segment 3 representing the south tunnel segments adjacent to the PSFSV and near the power source building where plant roads are not located and where external fuel cask storage pads are not anticipated to be located.

FSAR Subsection 3.8.4.4.3.1 has been revised to incorporate this response.

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

See attached marked-up FSAR Revision 1 page 3.8-10.

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

None.

Impact on DCD

None.

directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Gravity loads on the tunnel roof include a design surcharge pressure and are resisted by one-way slab action of the roof. These loads are distributed to the outer and interior walls, transferred through the walls down to the mat slab where they are distributed, and from the bottom of the mat slab to the concrete fill over limestone bedrock. A design surcharge pressure of 600 psf is applied to tunnel segments 1 and 2 and a design surcharge pressure of 200 psf is applied to tunnel segment 3. RCOL2_03.08.04-7 RCOL2_03.08.04-7

Lateral soil pressures on outer tunnel walls are typically resisted by one-way action of the outer walls. Forces from these pressures are transferred to the roof and mat slabs. Where axial force in the roof and mat slabs transverse to the tunnel axis are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces. ~~Some~~Corner tunnel segments resist unbalanced lateral loads in part by moment frame action and in part by return walls located at an end of the segment (such as where the ESWPT changes direction). RCOL2_03.08.04-9 RCOL2_03.08.04-9

Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock.

For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a frame member that transfers loads to the interior and exterior walls. The tunnel walls, roof, and base slab act as a moment frame causing out-of-plane bending in these elements. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. The dynamic soil pressures are described in Appendix 3LL. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ RCOL2_03.08.04-11

3.8.4.4.3.2 UHSRS

The UHSRS are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the UHSRS is performed using the computer program

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-8

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.1, "ESWPT," the second paragraph (Page 3.8-9) states that "The stiffness of the subgrade springs under different sections of the ESWPT is calculated using the methodology in ASCE [American Society of Civil Engineers]-4 Section 3.3.4.2 (Reference 3.8-34), for vibration of a rectangular foundation resting on an elastic half space. Since the support below the structure (fill concrete and rock) will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads."

The applicant is requested to provide the following information:

(a) ESWPT is an underground structure. The dynamic response of an underground structure subjected to earthquake excitation is different from that of a surface supported structure. The soil springs presented in ASCE-4 Section 3.3.4.2 are the impedance functions for rigid rectangular foundations on the ground surface for surface supported structures, not for underground structures. Provide the rationale and technical basis for using these springs for ESWPT. How is the soil on the sides and on the top of ESWPT considered in the analyses?

(b) Provide the technical basis and rationale for the statement that "the support below the structure will not exhibit long-term settlement effects" is a prerequisite for using the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 for both static and seismic loads.

ANSWER:

(a) FSAR Appendix 3LL discusses the seismic analysis of the ESWPT soil structure interaction (SSI) analysis. The seismic responses for the design are calculated using a two step analysis method as defined in ASCE 4-98. Step 1 is the SSI analysis using the program SASSI and step 2 is calculating seismic demands for the design using the program ANSYS as described below. The ANSYS design analysis models for the ESWPT were placed on soil springs calculated by methods provided in ASCE 4-98 to provide localized flexibility at the base of the structure. The flexibility of the base allows for the calculation of base slab demands. The effects of embedment are included in the SASSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis. The soil adjacent to the tunnel is not included in the design model in order to transfer the total seismic load through the structure down to the base slab.

The dynamic analysis of the ESWPT was performed using two analysis models: (1) a seismic SSI analysis using the computer program SASSI to determine the dynamic response of the tunnel including the in-structure response spectra, dynamic soil loads, and peak accelerations, and (2) a design analysis model using the computer program ANSYS to calculate seismic demands from equivalent lateral soil pressures and inertial demands.

The SSI analyses of the ESWPT using the program SASSI consider soil on all sides of the tunnels including the top and bottom. Where seismic isolation joints exist, the soil is separated from the structure.

The ANSYS design analyses calculated inertial demands in one of two ways:

(1) For tunnel segments 1 and 3, equivalent static acceleration loadings were applied. The applied accelerations enveloped the acceleration values calculated in the SSI analysis.

(2) For tunnel segment 2, a response spectra analysis was performed using the design input response spectra shown to be more conservative than the SSI spectra calculated at the base slab.

Soil on the sides of the tunnels was accounted for using dynamic soil pressures calculated in the ANSYS design model using static equivalent seismic soil pressure demands. The applied soil pressures were shown to envelope the SSI soil pressures calculated in SASSI. This method prevents the soil from providing support to the tunnel.

Soil above the tunnel was accounted for in two ways:

(1) a shear force was applied at the interface of the tunnel roof and the soil above where the shear value is shown to be higher than that calculated in SASSI SSI analyses, and

(2) the density of the tunnel roof slab was increased in regions of the tunnel where a balanced soil condition does not exist. This second method accounts for an assumed load path of bringing the entire soil mass into the roof slab through shear.

The total dynamic demands were calculated as the absolute sum of the inertial and dynamic soil pressure demands from the two design analyses.

FSAR Subsection 3.8.4.4.3.1 and Appendix 3LL have been revised to incorporate this response.

(b) The structures are supported on fill concrete down to limestone. All soil layers to a depth of more than 1000' have a shear wave velocity greater than 3000 ft/sec considering best estimate soil properties. Anticipated long-term settlements for CPNPP Units 3 and 4 are further discussed and justified in the responses to RAI No. 2999 (CP RAI #115) Question 03.08.05-2 attached to Luminant letter TXNB-09067 (dated November 13, 2009) (ML093230704) and to RAI No. 2929 (CP RAI #22) Question 02.05.04-17 attached to Luminant letter TXNB-09059 (dated October 28, 2009) (ML093080096). Those RAI responses indicate that maximum and differential settlements are estimated to be less than 0.5 inch, including any long-term settlements.

The wording in FSAR Subsection 3.8.5.4.4 has been revised to clarify that the settlements are "estimated" settlements.

Impact on R-COLA

See attached marked-up FSAR Revision 1 pages 3.8-9, 3.8-16, and 3LL-4.

Impact on S-COLA

None.

Impact on DCD

None.

3.8.4.3.7.1 Operating Thermal Loads (To)

CP COL 3.8(27) Replace the second paragraph in DCD Subsection 3.8.4.3.7.1 with the following.

The UHSRS, PSFSVs, and ESWPT structures experience only small ranges of operating temperatures and loads which do not require explicit analysis. The designs of the UHSRS, PSFSVs and ESWPT accommodate normal operating thermal loads and environmental thermal gradients such as those identified in Table 3.8-201.

3.8.4.4.3 Other Seismic Category I Structures

CP COL 3.8(29) Replace the last paragraph in DCD Subsection 3.8.4.4.3 with the following.

3.8.4.4.3.1 ESWPT

The ESWPT is designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the ESWPT is performed using the computer program ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3LL.

The static analyses are performed on the ANSYS model placed on soil springs at the top of the concrete fill representing the stiffness of the support provided by the concrete fill and limestone. The stiffness of the subgrade springs under different sections of the ESWPT is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34), for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil stiffness adjacent to the tunnel is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based upon. Since the support below the structure (fill concrete and rock) will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal

RCOL2_03.08.04-8

RCOL2_03.08.04-15

3.8.5.4.4 Analyses of Settlement

CP COL 3.8(26) Replace the last sentence of the first paragraph in DCD Subsection 3.8.5.4.4 with the following.

As discussed in Section 2.5.4.10.2, maximum and differential CPNPP settlements of all the ~~major~~ seismic category I buildings and structures at the CPNPP Units 3 and 4 site, including R/B, PS/Bs, ESWPT, UHSRS, and PSFSVs are estimated to be less than ½ inch, including long-term settlements. RCOL2_03.08.04-8

3.8.5.5 Structural Acceptance Criteria

CP COL 3.8(25) Replace the second sentence of the first paragraph in DCD Subsection 3.8.5.5 with the following.

All ~~major~~ seismic category I buildings and structures at the CPNPP Units 3 and 4 site, including R/B, PS/Bs, ESWPT, UHSRS, and PSFSVs, are founded either directly on a limestone layer or structural concrete fill which is placed directly on the limestone. The ultimate bearing capacity of the limestone is 146,000 psf. Table 3.8-202 shows the actual bearing pressure during static and seismic load cases with minimum factor of safety. The allowable static bearing capacity is calculated as 1/3 of the ultimate bearing capacity. The allowable dynamic bearing capacity is calculated as 1/2 of the ultimate bearing capacity. Table 2.8-203 shows the load combinations and factors of safety against overturning, sliding and flotation for site-specific buildings and structures. RCOL2_03.08.05-4 RCOL2_03.08.05-5 RCOL2_03.08.05-3

3.8.6 Combined License Information

Replace the content of DCD Subsection 3.8.6 with the following.

3.8(1) *Deleted from the DCD.*

3.8(2) *Deleted from the DCD.*

CP COL 3.8(3) ***3.8(3)*** *Material changes for PCCV*

This COL item is addressed in Subsection 3.8.1.6.

3.8(4) *Deleted from the DCD.*

response spectra and provide confirmation of the inputs to the ANSYS design model. RCOL2_03.07.02-11

ANSYS analyses are used to calculate the structural demands of the ESWPT to seismic soil pressure and seismic inertia which are then added to all other design loads discussed in Section 3.8.

The seismic inertia demand of segment 2 are calculated using ANSYS, response spectra analyses with the site specific 5% damped design response spectra. Modal combination is performed in accordance with RG 1.91 Combination Method B. Analysis of the ESWPT produced 40 modes below 50 Hz. Table 3LL-15 lists five major structural frequencies for each direction of motion organized by mass participation. RCOL2_03.07.02-16

The seismic inertia demand of segments 1 and 3 are calculated using an equivalent static lateral load based on the enveloped peak accelerations calculated in SASSI for all soil cases. RCOL2_03.07.02-11

The seismic soil pressure demands are calculated statically in ANSYS. The seismic soil pressure demands are applied on the structural elements as equivalent static pressures. The pressures applied are of larger magnitude compared to the calculated elastic solution used in ASCE 4-98 based on J.H. Wood, 1973 and the enveloped SASSI results. Soil above the tunnel is accounted for in two ways: (1) a shear force was applied at the interface of the tunnel roof and the soil above where the shear value is shown to be higher than that calculated in SASSI SSI analyses and (2) the density of the tunnel roof slab is increased in regions of the tunnel where a balanced soil condition does not exist. This second method accounts for an assumed load path of bringing the entire soil mass into the roof slab through shear. RCOL2_03.08.04-8

Demands calculated from the response spectra and soil pressure analyses performed in ANSYS for segment 2 are combined on an absolute basis to produce the maximum demands for each direction of motion and these directions are then combined spatially by 100-40-40 percent combination rule (Eq. 13 of RG 1.92). RCOL2_03.07.02-11

Demands calculated from the equivalent static accelerations and soil pressure analyses performed in ANSYS for segments 1 and 3 are combined to produce the maximum demands in each direction. The maximum demands for each direction of motion and these directions are then combined spatially by 100-40-40 percent combination rule (Eq. 13 of RG 1.92).

To confirm the design input and results from the ANSYS model of tunnel segment 2 used for response spectra analysis, the enveloped in-structure response spectra at the base slab calculated in the SASSI analysis are compared to the input spectra. The enveloped soil pressures from SASSI are compared to the soil pressures used as input to the ANSYS model, and the plate stresses from SASSI are compared to those calculated in ANSYS. The comparisons show that the

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-9

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.1, "ESWPT," the fourth paragraph (Page 3.8-9) states that "Where axial force in the roof and mat slabs are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces."

The applicant is requested to explain the direction of the "axial force" mentioned in the above quoted sentence. Is it in the direction of the central axis of the tunnel?

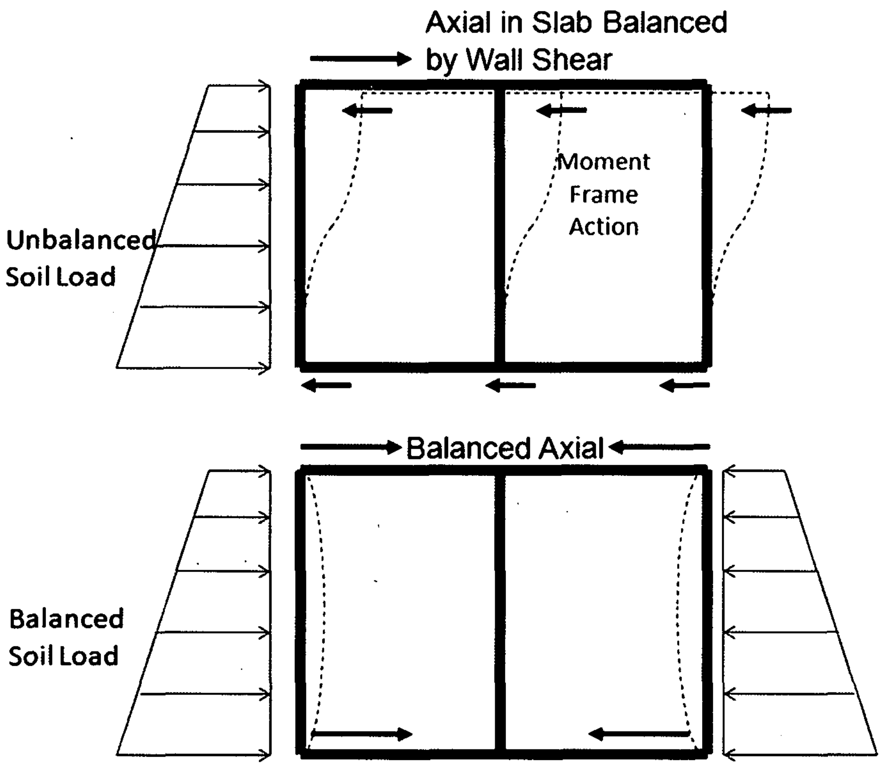
ANSWER:

The axial force in the statement quoted from Subsection 3.8.4.4.3.1 is horizontal force within the slab which is axial in the transverse direction of the tunnel due to unbalanced soil load on the exterior walls.

The response to Question 03.08.04-8 above added the following to FSAR Subsection 3.8.4.4.3.1 as shown on marked-up FSAR Revision 1 page 3.8-9:

> The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil stiffness adjacent to the tunnel is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based upon.

This statement is illustrated in the figure below showing only the horizontal, at-rest soil pressures on the sides of the tunnel. In the top figure, the axial force in the roof and mat slab is unbalanced by soil pressure on the other side. The axial force in the roof translates to out-of-plane shear in the walls, which causes the moment frame action of the tunnel. In the bottom figure, the soil pressure on both sides balances the axial force in the roof and mat slab.



FSAR Subsection 3.8.4.4.3.1 has been revised to clarify the direction of the axial forces.

Impact on R-COLA

See attached marked-up FSAR Revision 1 page 3.8-10.

Impact on S-COLA

None.

Impact on DCD

None.

directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Gravity loads on the tunnel roof include a design surcharge pressure and are resisted by one-way slab action of the roof. These loads are distributed to the outer and interior walls, transferred through the walls down to the mat slab where they are distributed, and from the bottom of the mat slab to the concrete fill over limestone bedrock. A design surcharge pressure of 600 psf is applied to tunnel segments 1 and 2 and a design surcharge pressure of 200 psf is applied to tunnel segment 3. RCOL2_03.08.04-7 RCOL2_03.08.04-7

Lateral soil pressures on outer tunnel walls are typically resisted by one-way action of the outer walls. Forces from these pressures are transferred to the roof and mat slabs. Where axial force in the roof and mat slabs transverse to the tunnel axis are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces. ~~Some~~Corner tunnel segments resist unbalanced lateral loads in part by moment frame action and in part by return walls located at an end of the segment (such as where the ESWPT changes direction). RCOL2_03.08.04-9 RCOL2_03.08.04-9

Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock.

For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a frame member that transfers loads to the interior and exterior walls. The tunnel walls, roof, and base slab act as a moment frame causing out-of-plane bending in these elements. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with K_0 = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. The dynamic soil pressures are described in Appendix 3LL. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ RCOL2_03.08.04-11

3.8.4.4.3.2 UHSRS

The UHSRS are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the UHSRS is performed using the computer program

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

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This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.1, "ESWPT," the fifth paragraph (Page 3.8-9) states that "Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock."

The friction mentioned in the above quoted paragraph includes friction between the tunnel and the concrete fill and friction between the concrete fill and the bedrock. The applicant is requested to address the issue, "What are the coefficients of friction used in these friction calculations, and what is the rationale for assuming these values?"

ANSWER:

Resistance to lateral loads is achieved by friction between the foundation basemat and the supporting subgrade. Passive soil resistance is not relied upon to resist lateral loads. Further, friction resistance acting on the side walls of embedded structures is not relied upon to resist lateral loads. Shear keys transfer lateral loads by lateral bearing on limestone and/or lateral bearing on fill concrete. Because there is no reliance on passive soil pressure, no specific displacement estimates have been made with respect to development of passive resistance of the soil. Resistance to lateral loads for site-specific structures is described further in the general discussions in FSAR Subsection 3.8.4.4.3.1 for the ESWPT.

An "ultimate" coefficient of friction is not applied in the stability design. The coefficient of friction considered is a static coefficient of friction since the structures are designed to preclude sliding. Therefore, a factor of safety is not applied to the coefficient of friction. Instead, a minimum factor of safety for sliding is applied that is consistent with the requirements of Table 3.8.5-1 of the DCD, which is incorporated by reference in the FSAR. FSAR Table 3.8.5-1 is based on the safety factor requirements contained in SRP 3.8.5 regarding stability against sliding, overturning and buoyancy. The factor of safety varies depending on each load combination shown in Table 3.8.5-1. The strength design of site-specific concrete structures (as opposed to stability requirements) is in accordance with the loads and load combination given in Table 3.8.4-3 of the DCD, which is incorporated by reference in the FSAR. Thus, the pertinent loads and load combinations of FSAR Table 3.8.4-3 are also applied to the strength design for the individual elements of structures, such as the mat foundations, below-grade walls, and shear keys, if and where needed.

A value of 0.6 is cited for the coefficient of friction between for the concrete/fill concrete interface, while 0.45 is used for the foundation concrete/limestone interface. The 0.6 factor is from ACI 349-01 Paragraph 11.7.4.3 for friction at concrete placed against hardened concrete not intentionally roughened.

The 0.45 factor is consistent with a friction angle of internal friction of approximately 24°. This is at the low end of values for various types of soil as given in Table 2-6 of Bowles, "Foundation Analysis and Design," and is therefore conservative. The use of the angle of internal friction for the friction angle between base and soil is appropriate as discussed on Bowles page 551. The 0.45 factor is also conservative because it is less than the coefficient of friction value of 0.60 for consideration at the interface of concrete and limestone, which is specified in FSAR Subsection 2.5.4.10.5 and clarified in the response to RAI No. 2929 (CP RAI #22) Question 02.05.04-18 attached to Luminant letter TXNB-09042 dated September 10, 2009 (ML092820486).

The damp-proofing used on the exterior surfaces of below-grade walls does not extend below the structure basemats and is therefore not present between the foundation and limestone, nor between fill concrete and limestone. Also, as stated above, no credit is taken for sliding resistance provided by friction acting along the side walls of structures. Therefore this coefficient of friction is not affected by the damp-proofing used at CPNPP.

FSAR Subsection 3.8.4.4.3.2 has been revised to incorporate this response.

References

ACI 349-01, "Code Requirements for Nuclear Safety Related Concrete Structures," 2001, American Concrete Institute, Farmington Hills, MI

Bowles, Joseph E., "Foundation Analysis and Design," 4th Ed., McGraw-Hill, Inc., New York, NY

Impact on R-COLA

See attached marked-up FSAR Revision 1 page 3.8-12.

Impact on S-COLA

None.

Impact on DCD

None.

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with $K_0 = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5. RCOL2_03.08.04-16 RCOL2_03.08.04-13 RCOL2_03.08.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above. RCOL2_03.08.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM. RCOL2_03.08.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are RCOL2_03.08.04-14

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-11

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.1, "ESWPT," the sixth paragraph (Page 3.8-9) states that "For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. The exterior walls are also designed for static and dynamic soil pressure in accordance with ASCE 4-98 (Reference 3.8-34)."

The applicant is requested to provide the following information:

(a) The first sentence of the above quoted paragraph discussed only the dynamic forces oriented parallel to the length of the tunnel segment. Provide a similar description for dynamic forces perpendicular to the length of the tunnel segment.

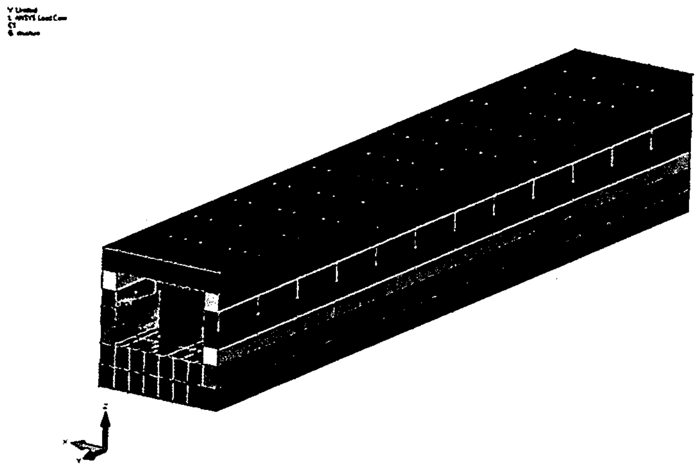
(b) Provide detailed information that shows how the static and dynamic soil pressure is calculated in accordance with ASCE 4-98.

ANSWER:

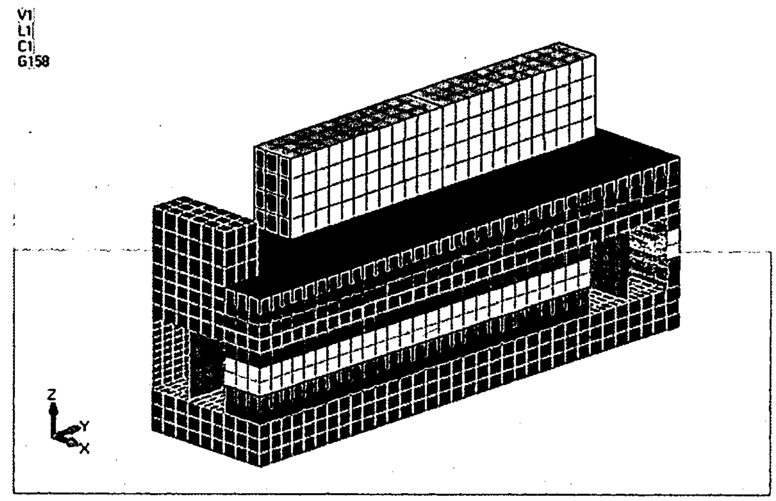
(a) For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a diaphragm that transfers loads to the interior and exterior walls. The tunnel walls and slabs behave similar to a moment frame causing out-of-plane bending in these elements. The dynamic soil and structural forces are transferred through the tunnel walls, into the mat slab, and then into the concrete fill and supporting limestone.

(b) The static soil pressures are not calculated using ASCE 4-98. The static soil pressures were calculated using at-rest pressures with $K_o = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243 of the FSAR. The design also considered the load from the overburden pressure and the soil compaction pressure.

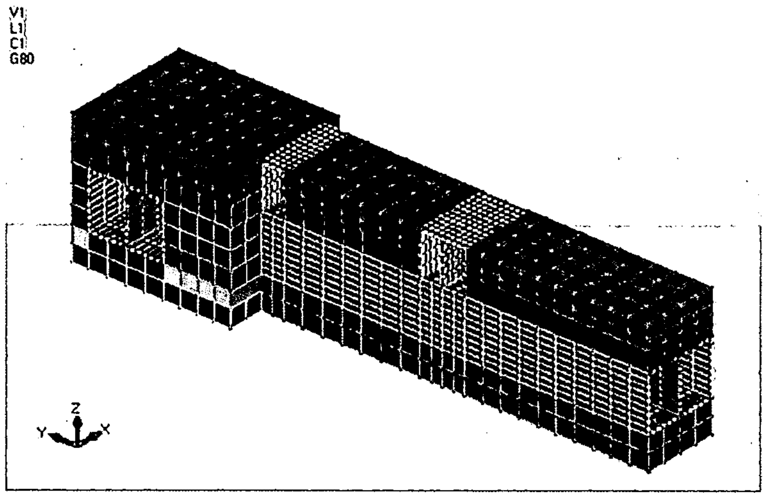
The seismic soil pressures are calculated directly in the SASSI analyses and are applied as equivalent static pressures in the ANSYS design model. The equivalent static seismic soil pressures applied in the ANSYS design model are shown to be conservative when compared to the calculated elastic solution used in ASCE 4-98 based on J.H. Wood, 1973 and the enveloped SASSI results. For calculation of the seismic soil pressures based on ASCE 4-98, the soil acceleration was taken as 1.5 times the ZPA of the spectrum at El. 782' (bottom of fill concrete) for tunnel segments 1 and 3 and 2.0 times this ZPA for tunnel segment 2. The value of C_v was taken equal to 0.99. The soil unit weight was taken as 125 pcf.



Tunnel Segment 1 including surrounding soil layers and fill beneath



Tunnel Segment 2 including surrounding soil layers and fill beneath



Tunnel Segment 3 including surrounding soil layers and fill beneath

FSAR Subsection 3.8.4.4.3.1 has been revised to incorporate this response. For clarification, the key plan of Segments 1, 2 and 3 has been added to FSAR Figure 3.8-201 in response to Question 03.08.04-1 above.

Impact on R-COLA

See attached marked-up FSAR Revision 1 page 3.8-10.

Impact on S-COLA

None.

Impact on DCD

None.

directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.0 8.04-15

Gravity loads on the tunnel roof include a design surcharge pressure and are resisted by one-way slab action of the roof. RCOL2_03.0 8.04-7 These loads are distributed to the outer and interior walls, transferred through the walls down to the mat slab where they are distributed, and from the bottom of the mat slab to the concrete fill over limestone bedrock. A design surcharge pressure of 600 psf is applied to tunnel segments 1 and 2 and a design surcharge pressure of 200 psf is applied to tunnel segment 3. RCOL2_03.0 8.04-7

Lateral soil pressures on outer tunnel walls are typically resisted by one-way action of the outer walls. Forces from these pressures are transferred to the roof and mat slabs. Where axial force in the roof and mat slabs transverse to the tunnel axis are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces. RCOL2_03.0 8.04-9 ~~Some~~Corner tunnel segments resist unbalanced lateral loads in part by moment frame action and in part by return walls located at an end of the segment (such as where the ESWPT changes direction). RCOL2_03.0 8.04-9

Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock.

For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a frame member that transfers loads to the interior and exterior walls. The tunnel walls, roof, and base slab act as a moment frame causing out-of-plane bending in these elements. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. The dynamic soil pressures are described in Appendix 3LL. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ RCOL2_03.0 8.04-11

3.8.4.4.3.2 UHSRS

The UHSRS are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the UHSRS is performed using the computer program

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-12

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.2, "UHSRS," the second paragraph (Page 3.8-9) states that "ANSYS analyses are performed on the model placed on soil springs at the bottom of the base slab, with the springs representing the stiffness of the rock subgrade. To address the sensitivity of the structural response on the subgrade stiffness, an additional set of analyses simulating a fixed base condition is performed on the model. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 393 ft."

The applicant is requested to provide the following information:

(a) The soil springs presented in ASCE-4 Section 3.3.4.2 are for the foundations resting on the surface of elastic half space. Provide technical justification for neglecting the embedment effect.

(b) Provide a rationale for only considering the best estimate properties of the soil layers for the ANSYS analysis; whereas, the SASSI analysis for UHSRS presented in Appendix 3KK considers the best estimate, the lower bound, the upper bound, and the high bound properties of the soil layers.

(c) Provide a summary of the results of the "additional set of analyses" performed.

ANSWER:

(a) FSAR Appendix 3KK discusses the seismic analysis of the UHSRS soil structure interaction (SSI) analysis. The seismic responses for the design are calculated using a two step analysis method as defined in ASCE 4-98. Step 1 is the SSI analysis using the program SASSI and step 2 is calculating the seismic demands for the design using the program ANSYS as described below.

The ANSYS design analysis models for the UHSRS were placed on soil springs calculated by methods provided in ASCE 4-98 to provide localized flexibility at the base of the structure. The flexibility of the base allows for calculation of base slab demands. The effects of embedment are included in the SSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis. The soil adjacent to the UHSRS is not included in the design model in order to transfer the total seismic loads through the structure down to the base slab.

The dynamic analysis of the UHSRS was performed using two analysis models:

(1) a seismic SSI analysis using the computer program SASSI was used to determine the dynamic responses including the in-structure response spectra, dynamic soil loads, and peak accelerations, and

(2) a design analysis model using the computer program ANSYS to calculate seismic demands from equivalent lateral soil pressures and inertial demands.

The SSI analysis of the UHSRS using the program SASSI considers soil on all sides of the UHSRS basins except for sides with isolation joints where soil is not in contact with the UHSRS.

The design models analyzed using ANSYS considered two bounding soil cases: a fixed base analysis and a flexible base analysis. The fixed base analysis is performed with constraint conditions set to fix the nodes of the base slab, providing an upper bound of foundation stiffness. The flexible base analysis used soil springs calculated by ASCE 4-98 to support the base slab nodes. The soil embedment effect would provide a stiffness higher than the surface condition but less than the fixed base condition. The embedment condition is bounded by the analyses performed and was therefore not considered.

(b) Calculation of the soil spring stiffness used in the ANSYS analysis was performed based on an equivalent shear wave travel time through layers below the UHSRS. The stiffness was calculated by varying the thickness considered from including only the first layer beneath the base mat to a depth of twice the UHSRS base width. The depth corresponding to minimum stiffness value was used for calculation of stiffness corresponding to a lower bound spring. The structure is analyzed in both a fixed-base condition and with the springs considered to be a lower bound case and the results are enveloped to develop bounding seismic design forces.

Soil uncertainties were considered in the SSI analyses that were used to produce the in-structure response spectra and confirm the design response spectra used as input for the design analyses. The SSI analyses show that because of the stiff supporting medium, the SSI effects do not have a major impact on the design input. Fixed-base analysis alone is therefore justified as a means of analyzing the structure but analysis of the structure on springs was used to provide an additional bounding case and allow determination of foundation demands.

(c) The additional analyses performed refer to the fixed base analysis case. The results of the analyses considering a flexible support and fixed base rigid support were enveloped to determine the maximum forces for design. The forces presented in FSAR Table 3KK-5 are enveloped values of these analysis cases.

FSAR Subsection 3.8.4.1.3.2 and Subsection 3.8.4.4.3.2 have been revised to incorporate this response.

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

See attached marked-up FSAR Revision 1 pages 3.8-6 and 3.8-11.

Impact on S-COLA

None.

Impact on DCD

None.

floor supports the ESWS pump, UHS transfer pump, and motors. The roof of the pump house is a reinforced concrete slab spanning north-south and supported by reinforced concrete beams. To allow access to the ESWS pump/motor, a removable reinforced concrete cover is provided in an opening in the roof of the pump house.

Tornado missile shields are provided to protect the air intake and air outlets of the ESWS pump house HVAC system from tornado missiles. The structural design considers tornado differential pressure loads as discussed in Subsection 3.3.2.2.2. RCOL2_09.04.05-4

UHS cooling tower enclosures - Each UHS basin has one cooling tower with two cells. Each cell is enclosed by reinforced concrete structures that house the equipment required to cool the water for ESWS. The reinforced concrete wall running north-south separates the two cell enclosures. The enclosures are an integral part of the UHS basin supported by the basin interior and exterior walls on the basemat foundation. A reinforced concrete wall, running east-west, separates the cell enclosure portion of the basin from the rest of the UHS basin. An east-west wall is provided with openings at the basemat to maintain the continuity of the UHS basin. Air intakes are located at the north and south faces of the cooling tower enclosure. The missile shields at the air intakes are ~~and~~ configured to protect the safety-related substructures and components housed within the UHS structure from tornado missiles. FSAR Table 3.2-201 lists the site-specific equipment and components located in the UHSRS that are protected from tornado missiles. The north side air intake is an integral part of the cooling tower enclosure, whereas the south side air intake is an integral part of the ESWPT, and is supported by reinforced concrete piers which are supported by the ESWPT walls and basemat. RCOL2_03.08.04-3

Each cooling tower cell enclosure is equipped with a fan and associated equipment to cool the water. Equipment includes header pipe, spray nozzles, and drift eliminators with associated reinforced concrete beams supported by the exterior walls of the enclosure. The fan and motor are supported by reinforced concrete deck above the drift eliminators. A circular opening is provided in the deck for the fan, and the deck is supported by enclosure walls and a deep upside circular concrete beam around the fan opening. The fan is supported by a north-south concrete beam at the center of enclosure. For air circulation and to protect the fan and motor from tornado missiles, a circular opening is provided at the roof of the enclosure (centered on the fan) with a reinforced concrete slab and heavy steel grating between the roof and the deck.

For details see Figures 3.8-207 through 3.8-211 for the UHS basin, UHS ESW pump house and cooling tower enclosures. Details of the UHSRS seismic analysis are provided in Appendix 3KK. RCOL2_03.08.04-12

3.8.4.1.3.3 PSFSVs

The PSFSVs are underground reinforced concrete structures required to house the safety-related and non safety-related fuel oil tanks. There is one vault for each

ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3KK.

The seismic responses for the design are calculated using a two step analysis method as defined in ASCE 4-98 (Reference 3.8-34). Step 1 is the SSI analysis using the program SASSI and step 2 is calculating the seismic demands for the design using the program ANSYS as described below. RCOL2_03.08.04-12

The ANSYS design analysis models for the UHSRS were placed on soil springs calculated by methods provided in ASCE 4-98 (Reference 3.8-34) to provide localized flexibility at the base of the structure. The flexibility of the base allows for calculation of the base slab demands. The effects of embedment are included in the SSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis.

ANSYS analyses are performed based on two support conditions: (1) flexible rock subgrade by applying soil springs across all base slab nodes and (2) rigid base by applying fixed restraints across all base slab nodes. All results from these two conditions are enveloped for design. ~~on the model placed on soil springs at the bottom of the base slab, with the springs representing the stiffness of the rock subgrade. To address the sensitivity of the structural response on the subgrade stiffness, an additional set of analyses simulating a fixed base condition is performed on the model.~~ The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the UHSRS is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 393 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-12

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-13

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.2, "UHSRS," the fourth paragraph (Page 3.8-10) states that "Above grade walls loaded laterally by seismic forces ..."

The applicant is requested to provide detailed information for how the seismic forces are applied. Are these seismic forces applied dynamically or statically?

ANSWER:

ANSYS analyses were used to calculate the structural demands of the UHSRS to seismic soil pressure and seismic inertia including hydrodynamic effects. The response spectra and soil pressure cases discussed below were analyzed for two boundary conditions: (1) fixed-base and (2) on soil springs.

For seismic inertia, the ANSYS analyses used response spectra analyses using the site-specific 5% damped design response spectra. Hydrodynamic effects were included in the response spectra analysis by modeling the fluid mass impulsive component using directional masses on the walls and slab and convective components using directional masses connected to the walls using directional springs. The response spectra input was modified to address the low damping of hydrodynamic modes by using 0.5% damped spectra values in the low frequency region (< 1 Hz) where convective hydrodynamic modes exist based on SRP 3.7.3. Modal combination was performed using RG 1.92 Combination Method B. Further explanation is provided in the response in RAI No. 2883 (CP RAI #64) Question 03.07.03-2 attached to Luminant letter TXNB-09060 dated October 30, 2009 (ML093090163).

For seismic soil pressure cases, analyzed statically in ANSYS, seismic soil pressure demands were applied to the structural elements as equivalent static pressures. Where the pressure represents the peak seismic soil pressures shown to be conservative when compared to the calculated elastic solution used in ASCE 4-98 based on J.H. Wood, 1973 and the enveloped SASSI results.

Demands calculated from the response spectra and soil pressure analyses performed in ANSYS were combined on an absolute basis to produce the maximum demands for each direction of motion and these directions were then combined spatially by 100-40-40 percent combination rule (Eq. 13 of RG 1.92).

A comparison of the SASSI generated site-specific in-structure response spectra at the base slab to the ANSYS input spectra shows that the input used for the ANSYS response spectra analyses is conservative. A comparison of the SASSI generated soil pressures with the soil pressures used for the seismic soil pressure analyses performed in ANSYS demonstrates that the applied loading is conservative.

FSAR Subsection 3.8.4.4.3.2 has been revised to incorporate this response.

For clarification, FSAR pages 3KK-5 and 3KK-6 were previously revised and Table 3KK-8 was added to incorporate the response to RAI No. 2897 (CP RAI #60) Question 03.07.02-11 attached to Luminant letter TXNB-09073 dated November 24, 2009 (ML093340447).

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

See attached marked-up FSAR Draft Revision 1 page 3.8-12.

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

None.

<u>Impact on DCD</u>

None.

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with $K_0 = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5.

RCOL2_03.08.04-16

RCOL2_03.08.04-13

RCOL2_03.08.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above. RCOL2_03.08.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM. RCOL2_03.08.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are RCOL2_03.08.04-14

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-14

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29), CPNPP COL FSAR, Subsection 3.8.4.4.3.3, "PSFSVs," the second paragraph (Page 3.8-10) states that "The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 215 ft."

The applicant is requested to provide the following information:

(a) Soil springs in ASCE-4 Section 3.3.4.2 are for surface foundations. Provide the technical justification for using these springs for the PSFSVs, which have an embedment depth of 40 ft. The equivalent radius of the PSFSV foundation is about 45 ft. The depth-to-equivalent-radius ratio is about 0.9. According to ASCE4-98, the maximum depth-to-equivalent-radius is 0.3 for neglecting the effect of embedment.

(b) Provide a rationale for only considering the best estimate properties of the soil layers for the ANSYS analysis; whereas, the SASSI analysis for PSFSVs presented in Appendix 3MM considers the best estimate, the lower bound, the upper bound, and the high bound properties of the soil layers.

ANSWER:

(a) FSAR Appendix 3MM discusses the seismic analysis of the PSFSV soil structure interaction (SSI) analysis. The seismic responses for design are calculated using a two step analysis method as defined in ASCE 4-98. Step 1 is the SSI analysis using the program SASSI and step 2 is calculating seismic demands for design using the program ANSYS as described below.

The ANSYS design analysis models for the PSFSVs were placed on soil springs calculated by methods provided in ASCE 4-98 to provide localized flexibility at the base of the structure. The flexibility of the base allows for calculation of base slab demands. The effects of embedment are included in the SSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis. The soil adjacent to the PSFSV is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. The design forces for use in the ANSYS model are based on the SASSI analyses, which include effects of embedment. The soil springs are included in the ANSYS model to simulate relative stiffness values in various parts of the foundation to maximize the seismic demands in the foundation. For this purpose it was deemed acceptable to use the soil springs as determined by the procedures of ASCE 4-98.

The dynamic analysis of the PSFSV was therefore performed using two analysis models:

(1) a seismic SSI analysis using the computer program SASSI to determine the dynamic response of the structure including the in-structure response spectra, dynamic soil loads, and peak accelerations, which included the effects of embedment, and

(2) a design analysis model using the computer program ANSYS to calculate seismic demands from equivalent lateral soil pressures and inertial demands from the SASSI analyses.

The SSI analyses of the PSFSV using the program SASSI included soil on the two sides of the vault where soil exists. On the two sides where seismic isolation joints exist the soil is separated from the structure. Analyses considered a range of soil properties representing uncertainty in the soil properties.

The ANSYS design analyses calculated inertial demands in using equivalent static acceleration loadings. The applied accelerations enveloped the acceleration values calculated in the SSI analysis. The forces applied are not dependent on foundation stiffness, and therefore analysis performed on springs was used to better represent the load distribution without reducing the design demand.

Soil on the sides of the vault was accounted for using dynamic soil pressures calculated in the ANSYS design model using static equivalent seismic soil pressure demands. The applied soil pressures were shown to envelope the SSI soil pressures calculated in SASSI. This method prevents the soil from providing support.

The total dynamic demands were calculated as the sum of the inertial and dynamic soil pressure demands from the two design analyses.

Using the above methods the embedment effect is accounted for in the SSI analyses that calculate the seismic loads, and the embedment effect is not required for the design analyses that calculate the structural demands that result from the seismic loads.

(b) Calculation of the soil spring stiffness used in the analysis was performed based on an equivalent shear wave travel time through layers below the PSFSV described in the response to Question 03.08.04-15 below. The stiffness was calculated by varying the thickness considered from including only the 1st layer beneath the base mat to a depth of twice the PSFSV base width. The depth corresponding to minimum stiffness value was used for calculation of soil stiffness to represent a lower bound stiffness. This was considered adequate for the design since an equivalent static load is applied and therefore the total load is not dependant on the foundation stiffness. The lateral load path of the PSFSV is in shear through the walls, and the softer soil support places more load in the shear walls and produces higher demands in the base slab than would occur with a stiffer support.

FSAR Subsection 3.8.4.4.3.3 has been revised to incorporate this response.

Impact on R-COLA

See attached marked-up FSAR Draft Revision 1 pages 3.8-12 and 3.8-13.

Impact on S-COLA

None.

Impact on DCD

None.

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with $K_o = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5.

RCOL2_03.08.04-16

RCOL2_03.08.04-13

RCOL2_03.08.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above.

RCOL2_03.08.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM.

RCOL2_03.08.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are

RCOL2_03.08.04-14

included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the PSFSVs is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 215 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-14

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Vertical loads present on the roof of the PSFSVs are carried by the perimeter and interior walls. The roof acts as a two-way slab based on its aspect ratio with a single span in the north-south direction and a 3-span continuous slab with two-way action in the east-west direction. The vertical wall loads are transmitted to the mat slab and into the bedrock. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with $K_o = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. Application of the dynamic soil pressure is described in Appendix 3MM. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ The exterior walls are designed with and without the roof slab for lateral static soil pressure, and with the roof slab for all other loading including seismic. Walls loaded laterally by earth pressure act as two-way plate members, spreading load to the mat slab and perpendicular shear walls. For seismic load cases, the shear walls are designed to resist 100% of the applied lateral load. The shear walls transmit load to the foundation mat along their length. The load in the foundation mat is then transferred to the bedrock via friction and shear keys. RCOL2_03.08.04-16 RCOL2_03.08.04-16

3.8.4.6.1.1 Concrete

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-15

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsections 3.8.4.4.3.1, 3.8.4.4.3.2, and 3.8.4.4.3.3 (Pages 3.8-8 through 3.8-10), the applicant states that the static analyses of ESWPT, UHSRS, and PSFSV are performed on the ANSYS models placed on the soil springs calculated using the methodology in ASCE-4 Section 3.3.4.2.

The applicant is requested to provide technical details that show how the soil springs for horizontal, rocking, vertical, and torsion motions were calculated, and how they are connected to the ANSYS finite element models.

ANSWER:

The generalized structure-foundation spring stiffnesses for a rectangular base mat (ASCE 4-98) are:

Horizontal = $$k_x = 2(1+\nu_o)G_o\beta_x\sqrt{BL}$$

Vertical = $$k_y = \frac{G_o}{1-\nu_o}\beta_z\sqrt{BL}$$

Rocking = $$k_\Psi = \frac{G_o}{1-\nu_o}\beta_\Psi BL^2$$

Where B = dimension of the basemat perpendicular to the direction of horizontal excitation, L= dimension of the basemat in the direction of horizontal excitation, G_o is the equivalent shear modulus of the half-space, ν_o is the equivalent Poisson's ratio of the half-space, and β_x, β_z, and β_Ψ are coefficients to be found in Figure 3.3-3 of ASCE-4.

The resulting equation for the equivalent shear modulus is: $G_o = \rho_o V_o^2$

Where ρ_o is the equivalent density of the effective half space and V_o is the equivalent shear wave velocity.

The equivalent shear wave velocity is then found by using the equivalent travel time method:

$$V_o = \frac{\sum H_i}{\sum \frac{H_i}{V_i}}$$

The description of this method will be added to FSAR Sections 3NN.2 and 3NN.3 in the response to RAI No. 3006 (CP RAI #122) Question 03.08.04-53 to be submitted to the NRC no later than December 21, 2009.

The equivalent density by a weighted average of the density of the individual soil layers with respect to soil layer thickness is:

$$\rho_o = \frac{\sum \rho_i H_i}{\sum H_i}$$

The equivalent Poisson's ratio by a weighted average of Poisson's ratio of the individual soil layers with respect to soil layer thickness is:

$$\nu_o = \frac{\sum \nu_i H_i}{\sum H_i}$$

Three individual, uncoupled uni-directional spring elements are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions is equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the spring stiffness that match ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness was not considered since significant torsional response was not expected (or observed) in any of the structures.

To determine the equivalent soil properties of the half space, ASCE 4-98 states that soil layers below a depth of three times the largest foundation width need not be considered. To account for some uncertainty in the correct depth to perform this calculation, the depth that calculates the minimum stiffness was used.

FSAR Subsections 3.8.4.4.3.1, 3.8.4.4.3.2 and 3.8.4.4.3.3 have been revised to incorporate this response.

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

See attached marked-up FSAR Revision 1 pages 3.8-9, 3.8-10, 3.8-11, and 3.8-13.

Impact on S-COLA

None.

Impact on DCD

None.

3.8.4.3.7.1 Operating Thermal Loads (To)

CP COL 3.8(27) Replace the second paragraph in DCD Subsection 3.8.4.3.7.1 with the following.

The UHSRS, PSFSVs, and ESWPT structures experience only small ranges of operating temperatures and loads which do not require explicit analysis. The designs of the UHSRS, PSFSVs and ESWPT accommodate normal operating thermal loads and environmental thermal gradients such as those identified in Table 3.8-201.

3.8.4.4.3 Other Seismic Category I Structures

CP COL 3.8(29) Replace the last paragraph in DCD Subsection 3.8.4.4.3 with the following.

3.8.4.4.3.1 ESWPT

The ESWPT is designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the ESWPT is performed using the computer program ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3LL.

The static analyses are performed on the ANSYS model placed on soil springs at the top of the concrete fill representing the stiffness of the support provided by the concrete fill and limestone. The stiffness of the subgrade springs under different sections of the ESWPT is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34), for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil stiffness adjacent to the tunnel is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based upon. Since the support below the structure (fill concrete and rock) will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal

RCOL2_03.08.04-8

RCOL2_03.08.04-15

directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Gravity loads on the tunnel roof include a design surcharge pressure and are resisted by one-way slab action of the roof. These loads are distributed to the outer and interior walls, transferred through the walls down to the mat slab where they are distributed, and from the bottom of the mat slab to the concrete fill over limestone bedrock. A design surcharge pressure of 600 psf is applied to tunnel segments 1 and 2 and a design surcharge pressure of 200 psf is applied to tunnel segment 3. RCOL2_03.08.04-7 RCOL2_03.08.04-7

Lateral soil pressures on outer tunnel walls are typically resisted by one-way action of the outer walls. Forces from these pressures are transferred to the roof and mat slabs. Where axial force in the roof and mat slabs transverse to the tunnel axis are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces. ~~Some~~Corner tunnel segments resist unbalanced lateral loads in part by moment frame action and in part by return walls located at an end of the segment (such as where the ESWPT changes direction). RCOL2_03.08.04-9 RCOL2_03.08.04-9

Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock.

For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a frame member that transfers loads to the interior and exterior walls. The tunnel walls, roof, and base slab act as a moment frame causing out-of-plane bending in these elements. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. The dynamic soil pressures are described in Appendix 3LL. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ RCOL2_03.08.04-11

3.8.4.4.3.2 UHSRS

The UHSRS are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the UHSRS is performed using the computer program

ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3KK.

The seismic responses for the design are calculated using a two step analysis method as defined in ASCE 4-98 (Reference 3.8-34). Step 1 is the SSI analysis using the program SASSI and step 2 is calculating the seismic demands for the design using the program ANSYS as described below. RCOL2_03.08.04-12

The ANSYS design analysis models for the UHSRS were placed on soil springs calculated by methods provided in ASCE 4-98 (Reference 3.8-34) to provide localized flexibility at the base of the structure. The flexibility of the base allows for calculation of the base slab demands. The effects of embedment are included in the SSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis.

ANSYS analyses are performed based on two support conditions: (1) flexible rock subgrade by applying soil springs across all base slab nodes and (2) rigid base by applying fixed restraints across all base slab nodes. All results from these two conditions are enveloped for design.~~ on the model placed on soil springs at the bottom of the base slab, with the springs representing the stiffness of the rock subgrade. To address the sensitivity of the structural response on the subgrade stiffness, an additional set of analyses simulating a fixed base condition is performed on the model.~~ The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the UHSRS is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 393 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-12

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the PSFSVs is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 215 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-14

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Vertical loads present on the roof of the PSFSVs are carried by the perimeter and interior walls. The roof acts as a two-way slab based on its aspect ratio with a single span in the north-south direction and a 3-span continuous slab with two-way action in the east-west direction. The vertical wall loads are transmitted to the mat slab and into the bedrock. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with $K_0 = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. Application of the dynamic soil pressure is described in Appendix 3MM. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ The exterior walls are designed with and without the roof slab for lateral static soil pressure, and with the roof slab for all other loading including seismic. Walls loaded laterally by earth pressure act as two-way plate members, spreading load to the mat slab and perpendicular shear walls. For seismic load cases, the shear walls are designed to resist 100% of the applied lateral load. The shear walls transmit load to the foundation mat along their length. The load in the foundation mat is then transferred to the bedrock via friction and shear keys. RCOL2_03.08.04-16 RCOL2_03.08.04-16

3.8.4.6.1.1 Concrete

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

Comanche Peak, Units 3 and 4

Luminant Generation Company LLC

Docket Nos. 52-034 and 52-035

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SRP SECTION: 03.08.04 - Other Seismic Category I Structures

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DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-16

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(29) in CPNPP COL FSAR, Subsection 3.8.4.4.3.3, "PSFSVs," the third paragraph (Page 3.8-11) states that "The roof acts as a two-way slab with a single span in the north-south direction and a 3-span continuous slab with two-way action in the east-west direction. The vertical wall loads are transmitted to the mat slab and into the bedrock. The exterior walls are also designed for static and dynamic soil pressure in accordance with ASCE 4-98 (Reference 3.8-34)."

The applicant is requested to provide the following information:

(a) CPNPP COL FSAR Figure 3.8-213 shows that the roof of the PSFSV is steel decking with concrete slabs supported on steel I beams. Steel deck is a one-way structure. Justify the above quoted paragraph which states that "The roof acts as a two-way slab".

(b) How are the static and dynamic soil pressures calculated in accordance with ASCE 4-98?

ANSWER:

(a) The steel beams and decking are intended to be used as in-place formwork and are designed to support the wet concrete during construction. The total slab is 2'-4 1/2" thick with 3"-steel decking used as stay-in-place formwork. The steel decking is flexible in relation to the concrete slab. The concrete portion of the slab has two layers of steel reinforcement and was designed to support all other loads without the assistance of the steel beams or decking. Load distribution was calculated based on finite element analysis distribution of loads for the concrete slab alone, which acts as a two-way slab based on the aspect ratio of the slab.

(b) The static soil pressures were calculated using at-rest pressures with $K_o = 0.47$. The design also considered the load from the overburden pressure and the soil compaction pressure. Static demands are based on at-rest soil pressures, and are not related to ASCE 4. The seismic soil pressures are calculated directly in the SASSI analyses and are applied as equivalent static pressures in the ANSYS design model. The equivalent static seismic soil pressures applied in the ANSYS design model are shown to be conservative when compared to the calculated elastic solution used in ASCE 4-98 based on J.H. Wood, 1973 and the enveloped SASSI results.

Subsections 3.8.4.4.3.2 and 3.8.4.4.3.3 of the FSAR have been revised to incorporate this response.

Impact on R-COLA

See attached marked-up FSAR Revision 1 pages 3.8-12 and 3.8-13.

Impact on S-COLA

None.

Impact on DCD

None.

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5. RCOL2_03.08.04-16 RCOL2_03.08.04-13 RCOL2_03.08.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above. RCOL2_03.08.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM. RCOL2_03.08.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are RCOL2_03.08.04-14

included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the PSFSVs is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 215 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-14

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures. RCOL2_03.08.04-15

Vertical loads present on the roof of the PSFSVs are carried by the perimeter and interior walls. The roof acts as a two-way slab based on its aspect ratio with a single span in the north-south direction and a 3-span continuous slab with two-way action in the east-west direction. The vertical wall loads are transmitted to the mat slab and into the bedrock. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with $K_0 = 0.47$. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. Application of the dynamic soil pressure is described in Appendix 3MM. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ The exterior walls are designed with and without the roof slab for lateral static soil pressure, and with the roof slab for all other loading including seismic. Walls loaded laterally by earth pressure act as two-way plate members, spreading load to the mat slab and perpendicular shear walls. For seismic load cases, the shear walls are designed to resist 100% of the applied lateral load. The shear walls transmit load to the foundation mat along their length. The load in the foundation mat is then transferred to the bedrock via friction and shear keys. RCOL2_03.08.04-16 RCOL2_03.08.04-16

3.8.4.6.1.1 Concrete

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.: 2994 (CP RAI #108)

SRP SECTION: 03.08.04 - Other Seismic Category I Structures

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

DATE OF RAI ISSUE: 10/2/2009

QUESTION NO.: 03.08.04-17

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR 50.55a, and General Design Criteria (GDC) 1, 2, 4, and 5.

In CP COL 3.8(28) in CPNPP COL FSAR, Subsection 3.8.4.6.1.1, "Concrete" (Page 3.8-11), the applicant is requested to specify the strength of concrete fill.

ANSWER:

FSAR Subsection 3.7.1.3 states that the fill concrete has a design compressive strength of 3,000 psi.

FSAR Subsection 3.8.4.6.1.1 has been revised to specify the strength of the concrete fill as 3,000 psi.

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

See attached marked-up FSAR Revision 1 page 3.8-14.

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

None.

<u>Impact on DCD</u>

None.

CP COL 3.8(28) Replace the third sentence of the first paragraph in DCD Subsection 3.8.4.6.1.1 with the following.

For ESWPT, UHSRS, and PSFSVs concrete compressive strength, f'_c = 5,000 psi is utilized. The compressive strength, f'_c, of the concrete fill under the ESWPT, UHSRS, and PSFSVs is 3,000 psi. RCOL2_03.08.04-17

3.8.4.7 Testing and Inservice Inspection Requirements

CP COL 3.8(22) Replace the second through last paragraph of Subsection 3.8.4.7 with the following.

A site-specific program for monitoring and maintenance of seismic category I structures is performed in accordance with the requirements of NUMARC 93-01 (Reference 3.8-28) and 10 CFR 50.65 (Reference 3.8-29) as detailed in RG 1.160 (Reference 3.8-30). Monitoring of seismic Category I structures includes base settlements and differential displacements.

Prior to completion of construction, site-specific programs are developed in accordance with RG 1.127 (Reference 3.8-47) for ISI of seismic category I water control structures, including the UHSRS and any associated safety and performance instrumentation.

The site-specific programs address in particular ISI of critical areas to assure plant safety through appropriate levels of monitoring and maintenance. Any special design provisions (such as providing sufficient physical access or providing alternative means for identification of conditions in inaccessible areas that can lead to degradation) to accommodate ISI are also required to be addressed in the ISI program.

Because the CPNPP site exhibits nonaggressive ground water/soil (i.e., pH greater than 5.5, chlorides less than 500 ppm, and sulfates less than 1,500 ppm), the program for ISI of inaccessible, below-grade concrete walls and foundations of seismic category I structures~~the UHSRS~~ is less stringent than would be applied for sites with aggressive ground water/soil. The program is required to include requirements for (1) examination of the exposed portions of the below-grade concrete, when excavated for any reason, for signs of degradation; and (2) conducting periodic site monitoring of ground water chemistry, to confirm that the ground water remains nonaggressive. RCOL2_03.08.01-5

3.8.5.1 Description of the Foundations

CP COL 3.8(23) Replace the second sentence of the second paragraph in DCD Subsection 3.8.5.1 with the following.