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April 1, 2010

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. 4294 AND 4297

Dear Sir:

Luminant Generation Company LLC (Luminant) submits herein the response to Request for Additional Information (RAI) No. 4294 and No. 4297 for the Combined License Application for Comanche Peak Nuclear Power Plant Units 3 and 4. RAI No. 4294 involves the use of foundation-level instrumentation for operating basis earthquake determinations and RAI No. 4297 involves installing the curved prestressing tendons in the concrete containment vessel.

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

There are no commitments in this letter.

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

Executed on April 1, 2010.

Sincerely,

Luminant Generation Company LLC

Rafael Flores

Rafael Flores

Attachments 1. Response to Request for Additional Information No. 4294 (CP RAI #146)
2. Response to Request for Additional Information No. 4297 (CP RAI #148)

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Electronic distribution w/attachments

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Attachment 1

Response to Request for Additional Information No. 4294 (CP RAI #146)

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.: 4294 (CP RAI #146)

SRP SECTION: 03.07.04 - Seismic Instrumentation

QUESTIONS for Geosciences and Geotechnical Engineering Branch 1 (RGS1)

DATE OF RAI ISSUE: 2/26/2010

QUESTION NO.: 03.07.04-2

Paragraph IV(a)4 of Appendix S of 10 CFR 50 requires that, "Suitable instrumentation must be provided so that the seismic response of nuclear power plant features important to safety can be evaluated promptly after an earthquake." Regulatory Guide (RG) 1.166 provides the guidance regarding the instrumentation and procedures to make the required evaluation.

In FSAR subsection 3.7.4.1, "Comparison with Regulatory Guide 1.12" you proposed to use foundation-level instrumentation for operating basis earthquake (OBE) determinations. The FSAR states that "it is acceptable to perform a CAV check of seismic responses measured at the R/B and PS/B foundation locations". RG 1.166 explicitly states that "The evaluation to determine whether the OBE was exceeded should be performed using data obtained from the three components of the free-field ground motion (i.e., two horizontal and one vertical)". Also, Appendix A to RG 1.166, which provides interim OBE exceedance guidelines in the case that the installed seismic instrumentation or data processing equipment is inoperable, states that "For plants at which instrumentally determined data are available only from an instrument installed on a foundation, the cumulative absolute velocity (CAV) check (see Regulatory Position 4.2 of this guide) is not applicable." Considering that the CAV value of 0.16g-sec was defined using free-field instruments, the staff is not clear based on the justification provided in the FSAR and is concerned that the plant may not be shutdown in all instances when RG 1.166 anticipated a shutdown would be performed. Please provide further clarification why foundation instrument records are appropriate for CAV checks for CPNPP's OBE determinations.

ANSWER:

10 CFR 50 Appendix S IV(a)(1)(i) states that the SSE must be characterized by free-field ground motion response spectra at the free ground surface. The certified seismic design response spectra (CSDRS) described in the DCD for the standard plant is a free-field motion and is applied at the base of the foundations. The relationship between free-field ground surface motion and the motion at the foundation bases cannot be determined without knowing site-specific information. For purposes of generic design the DCD assumes that the CSDRS and the free-field motion at the ground surface are

the same. The operating basis earthquake (OBE) is set on a site-specific basis by the COL applicant. The DCD requires each applicant to confirm that the foundation input response spectra (FIRS) are enveloped by the CSDRS, which is the input motion for design at the base of foundation.

As shown in FSAR Figure 3.7-201, the computed theoretical free-field ground surface motion is greater than the computed theoretical input motion at the foundation bases. As stated in FSAR Subsection 3.7.4.1, at the CPNPP Units 3 and 4 site the free-field ground motion and theoretically calculated motion at FIRS locations are both enveloped by the minimum earthquake motion tied to a peak SSE ground acceleration or zero period acceleration of 0.1g that is specified in 10CFR50 Appendix S for a horizontal component. Therefore, for CPNPP the minimum earthquake motion specified in Appendix S is adopted as the required FIRS input, and consequently, the free-field motion for purposes of design and shutdown consideration is one and the same as the FIRS at the foundation bases. As stated in FSAR Subsection 3.7.1.1, 10 CFR 50 Appendix S IV(2)(i)(A) is adopted for the CPNPP Units 3 and 4 site, such that OBE is set at 1/3 of the site-specific SSE. With respect to shutdown considerations, this is conservative compared to Appendix S IV(2)(i)(B), which allows OBE to be set at a greater value, and is also conservative with respect to shutdown evaluation using 1/3 of the CSDRS for the standard plant.

In-structure response spectra (ISRS) are developed for the foundation bases in accordance with Subsection 3.7.2.5 of the DCD using the FIRS. The structures important to safety are founded on stiff limestone at the CPNPP Units 3 and 4 site. Therefore, only minimal amplification above the FIRS minimum earthquake motion used as input in the ISRS analysis occurs at foundation locations within the plant due to soil structure interaction.

Therefore, the seismic instrumentation provided at the R/B and PS/B foundation-levels is suitable to use for the CAV calculation check based on one third of the controlling minimum earthquake, which is equivalent to providing instrumentation in the free field as required by RG 1.166. Accordingly, plant shutdown is anticipated to be performed in all instances when shutdown would be required by RG 1.166 criteria.

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

None.

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

None.

<u>Impact on DCD</u>

None.

Attachment 2

Response to Request for Additional Information No. 4297 (CP RAI #148)

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.: 4297 (CP RAI #148)

SRP SECTION: 03.08.01 - Concrete Containment

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

DATE OF RAI ISSUE: 2/26/2010

QUESTION NO.: 03.08.01-10

This Request for Additional Information (RAI) is necessary for the staff to determine if the application meets the requirements of 10 CFR section 50.34(f) and 50.55a, and General Design Criteria (GDC) 1, 2, 4, 16, and 50.

In the applicant's response to RAI 2990 (RAI Number 106) Question 03.08.01-02, Luminant did not provide a discussion of any special measures that may be required to install curved prestressing tendons which are 500 ft long. Thus, Luminant is requested to provide a discussion addressing the measures that it may undertake to install the tendons. In addition, Luminant is requested to describe any special measures or precautions that may be necessary to avoid any delamination of the concrete in the dome of the PCCV.

Reference: Luminant response to Request for Additional Information (RAI) Number 2990 (RAI Number 106), dated November 13, 2009, Log # TXNB-09067, (ML093230705).

ANSWER:

Regarding measures taken to install the tendons, the CPNPP Units 3 and 4 are designed considering a lay out pattern to permit smooth transition to accommodate changes in direction. Minimum radius requirements are maintained around openings.

The tendon sheathing (i.e., duct) size is selected so there is ample room for tendon installation and couplers are used at the joints which have a smooth inner transition surface. Sheathing is installed and inspected to assure there are no significant offsets and indentations from their placement prior to concrete placement. In addition a device is pulled through the sheathing to verify proper clearance to assure ease of future tendon installation at a time when concrete has reached sufficient strength. The sheathing is temporarily capped to prevent the intrusion of foreign material.

Since the mid 1960's mock-ups of containment cylindrical walls have been used to determine items such as friction values and verify installation techniques. Most recently the VSL Corporation has constructed a 360-

degree mock-up with a diameter slightly larger than that planned for use at the CPNPP Units 3 and 4 containments. The mock-up also had a portion that simulated tendons deflected around an opening. The tendon length was more than 500 ft. The tendon was made from a 0.6-inch diameter strand with 55 strands per tendon. Tests were also performed for 49 strands, which are presently proposed for CPNPP Units 3 and 4. The mock-up has illustrated that large tendons can be installed and tensioned successfully.

Regarding the avoidance of delamination, the CPNPP Units 3 and 4 PCCVs are designed in accordance with the requirements of the ASME Section III, Division 2 Code. The provisions of ASME CC-3545 state that either the design requires a sufficient cross section to ensure that delaminations do not occur or radial reinforcing is provided if the higher code compression allowable is used. The fabrication and installation will comply with the requirements of the ASME Section III, Division 2 Code, Article CC-4400. Construction specifications are developed in accordance with ASME CC-4462 to provide the required sequence of prestressing. Proper sequencing assures that prestressing loads during construction do not exceed the construction loads and load combinations considered in the design as per ASME Table CC-3230-1 and ASME CC-3221.2. This process protects against delamination caused by improper consideration of unbalanced post-tensioning loads, which contributed to delamination in the past. The information provided in Attachment 1 below illustrates the conservatism that is now required by the ASME Section III, Division 2 Code compared to what was used many years ago.

NUREG/CR-4652, "Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants", September, 1986 (ML040230118) provides some discussion of dome delaminations that occurred in the past. In addition, findings from an original investigator at the Turkey Point site (Attachment 2) provide additional explanation as to why two past dome delaminations occurred.

For the two containments that had experienced dome delaminations in the United States in the late 1960's and early 1970's, the main reference code was ACI 318-63. This code allowed a compression stress of up to 0.60 f_{ci}' at initial prestressing [Section 2605 (a) 1. Compression of ACI 318-63]. Apparently both containments did not use a value this high, but more like 0.45 f_{ci}'. However the fact that in the dome the tendon ducts reduced the effective cross section by about 25% resulted in the stress on the net section to approach 0.45 f_{ci}' (1.0/(1.0-0.25)) = 0.60 f_{ci}'. As shown in Attachment 2 below, this value is close to the ultimate strength of a large concrete member, and when combined with radial tension, the outer layer is susceptible to delaminating and separating if no radial reinforcing is provided. This is especially true for concrete mixes used in Florida, with Oolite aggregate. The CPNPP Units 3 and 4 PCCV domes will not use Oolite aggregate and will have radial tension reinforcement designed to meet the provisions of ASME CC-3545 to avoid delamination of the concrete.

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

None.

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

None.

<u>Impact on DCD</u>

None.

<u>Attachments</u>

1. Summary of Concrete Compression Allowables and Radial Tension Reinforcing in Prestressed Concrete Containments Designed in Accordance with the ASME Section III, Division 2 Code

2. Turkey Point Unit 3 Dome Delaminations - Comments of Original Investigator

Attachment 1

Summary of Concrete Compression Allowables and Radial Tension Reinforcing in Prestressed Concrete Containments Designed in Accordance with the ASME Section III, Division 2 Code

1.0 General

After the delamination of the Turkey Point Nuclear Power Plant prestressed concrete dome, provisions were incorporated in the ASME Section III, Division 2 Code to:

1] Provide criteria that would have a have a high probability to prevent delaminations from occurring using sufficient concrete compression allowables.

2] Point out the need to use net concrete section areas in computing compression stresses.

3] Require as an option radial reinforcement that would maintain the entire concrete thickness to function as a unit, in the unlikely event that the concrete shell would form delamination planes. This option then allows a higher concrete compression allowable.

2.0 ASME Section III. Division 2 Code Provisions

The Code requirements are summarized below:

CC-3545 Radial Tension Reinforcement

For portions of prestressed containments with curvature, consideration shall be given to the radial forces. Radial reinforcement shall be provided to resist radial tensile forces from curved tendons in portions of the containment with double curvature.

The spacing of reinforcement shall not be greater than the shell thickness or 3 ft (914 mm), whichever is smaller. The radial reinforcement shall be developed on either side of the centroid of the curved tendons in accordance with CC-3530.

CC-3430 Table CC-3431-1, "ALLOWABLE COMPRESSION STRESSES FOR SERVICE LOADS," allows the following for Primary Membrane:

The normal Service Load compression allowable is 0.30 f_c' and under initial prestress loading it is 0.35 f_c' and the following notes apply:

NOTES:

1] The primary portion of this calculated stress shall not exceed the allowable stress applicable when primary stress acts alone.

2] These allowable stresses may be increased by 0.05 f_c' for sections with radial tension reinforcement.

3] At initial prestress.

CC-3432 Reinforcing Steel Stresses and Strains CC-3432.1 Bar Tension

(a) The average tensile stress shall not exceed 0.5 f_y.

(b) The calculated average stress for the reinforcing in zones which have predicted concrete tension due to prestressing loads shall not exceed 0.5 f_y. The maximum load considered need not exceed the initial force at tendon anchoring. The values given in (a) above may be increased by 33.33% when the following loads are combined with other loads in the load combination:

(I) the temporary loads from prestressing which will reduce at completion of prestressing

CC-3541 (3) This section requires that "prior to bonding of tendons, areas of open ducts shall be deducted".

Note: Here the intent is to have the net section used specially for "unbonded/greased" type tendons.

Attachment 2

TURKEY POINT UNIT 3 DOME DELAMINATIONS – COMMENTS OF ORIGINAL INVESTIGATOR

1.0 General Description

At completion of prestressing on the Turkey Point Unit 3 Dome, it was noticed that there was some concrete spalling out at a radial location of about 30 to 40 ft from the apex. Concrete was removed to determine the extent of the spalled or what became known as the delaminated concrete, the delamination (cracking/separation essentially parallel to the surface). At the apex the depth was about 12 in from the outer surface. The dome thickness was 39 in. An extensive investigation was performed to determine the cause. The concrete strength based on 6" by 12" cylinders was 5000psi. However, the question was, what is the true strength of this large shell with inplane biaxial compression combined with a radial tension in the outer portions of the dome thickness with sustained loads?

2.0 Investigation

A. Due to the fact that 5 layers of tendons (with 4" diameter ducts) were used, what was the effective reduction of cross section, since these were not grouted and used a corrosion inhibiting filler? This reduction was determined to be 25 percent based on testing models of the actual removed concrete.

B. Based on reviewing test results of large cylinders (24" by 48") compared to normal (6" by 12") it was concluded that a large concrete member would have a strength of only about 80 percent of a typical concrete cylinder.

C. The tests performed on both normal and large specimens had end restraint in the form of friction which leads to increased strength and an apparent shear type failure surface that is a 45 degree failure surface.

D. To investigate the effect of end restraint which would not exist in long members or shells, tests were performed on cylinders with Teflon wedge shaped layers on the top and bottom to minimize friction. These tests showed another strength reduction of about 20 percent. Also the 45 degree failure surface did not exist. The cylinders split in the vertical direction apparently due to a Poissons' strain failure effect. This was interesting since that is what appeared to happen in the actual structure being investigated. In addition some biaxial tests were performed and the splitting phenomenon was also observed.

E. Another question had to do with sustained loading compared to the short term normal cylinder tests. Specimens were tested and found to fail at about 90 percent of the short term normal test values.

F. Direct tension tests were done on both delaminated concrete and 6" by 12" cylinders from the jobsite storage. These were actual tension tests and not splitting tensile tests. These tests indicated that the concrete tensile strength at this plant was about 6 percent of the 6" by 12" compressive cylinder tests. Concrete from other jobsites indicated that the tensile strength was typically 10 percent. Now 6 percent of 5000 psi is 300 psi; however, applying the same logic relative to strength reduction from the missing area due to ducts and other effects, to the tension strength it is most likely about 180 psi.

3.0 Final Conclusions as to the Delamination Causes

A. Combining all the above items that affected just the compression strength, it was concluded that a large compression member that did not have end restraint with a sustained loading and 25 percent of the area missing would have a compressive strength reduction ratio of, (0.75)*(0.8)*(0.8)*(0.9) = 0.432. Therefore what was assumed as a 5000psi concrete from cylinder tests is effectively (0.432)*5000 = 2160 psi.

B. For a straight line interaction between compression and tension and considering that the applied initial prestress would be about 100 psi then the tension around 12" below the outer surface is about [12/39]*100 = 31 psi and the compression that would act with this value, for failure, would be [(180-31)/180]*2160 = 1788 psi.

C. The calculated membrane stresses in the outer portion of the dome were about 1750 psi and with temperature gradient even higher. Therefore, it was concluded that a condition occurred where the dome was stressed to its ultimate cracking capacity in the outer portion. Below about 12" in depth from the surface the concrete was in triaxial compression and had a very high strength.