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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

June 18, 2010

UN#10-157

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI 240, Questions 02.05.04-21, 22, and 23, Stability of Subsurface Materials and Foundations

Reference:
1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 240 RGS1 4659" email dated May 14, 2010

2) UniStar Nuclear Energy Letter UN#10-163, from Greg Gibson to Document Control Desk, U.S. NRC, RAI 240, Questions 02.05.04-21, 22, and 23, Stability of Subsurface Materials and Foundations, dated June 14, 2010

The purpose of this letter is to respond to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated May 14, 2010 (Reference 1). This RAI addresses Stability of Subsurface Materials and Foundations, as discussed in Section 2.5.4 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 6.

Reference 2 anticipated that the response would be provided by June 21, 2010. The enclosure provides our response to RAI No. 240, Questions 02.05.04-21, 22, and 23, and includes revised

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COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

Our response does not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

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

Executed on June 18, 2010

Greg Gibson

Enclosure: Response to NRC Request for Additional Information RAI No. 240, Questions 02.05.04-21, 22, and 23, Stability of Subsurface Materials and Foundations, Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Loren Plisco, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

GTG/SJS/mdf

Enclosure

Response to NRC Request for Additional Information RAI No. 240,
Questions 02.05.04-21, 22, and 23, Stability of Subsurface Materials and Foundations,

Calvert Cliffs Nuclear Power Plant, Unit 3

RAI No. 240

Question 02.05.04-21

FSAR Section 2.5.4.7 states that the SSE spectra "would be expected to be modified as appropriate to develop ground motion for design considerations." Please justify why the GMRS would need modification during design and clarify when this design will occur. Please provide the results of such modifications when performed and appropriately update the FSAR. The requested information must be in sufficient detail for the evaluation of the suitability of the proposed site and adequacy of the design bases in accordance with 10 CFR 100.23.

Response

The Safe Shutdown Earthquake (SSE) spectra developed in FSAR Section 2.5.2.6 and its specific location at a free ground surface reflect the seismic hazard in terms of probabilistic seismic hazard analysis (PSHA) and geologic characteristics of the site and represent the site-specific ground motion response spectrum. These spectra are modified as appropriate to develop ground motion for design considerations. Detailed descriptions on response of site soils and rocks to dynamic loading are addressed in FSAR Section 2.5.2.

The CCNPP 3 ground motion response spectra (GMRS) is below minimum acceleration thresholds. Development of Site SSE in accordance with Appendix S of 10 CFR Part 50 requires that the horizontal component of the SSE ground motion in the free-field at the foundation level of the structures must be an appropriate response spectrum with a peak ground acceleration of at least 0.1 g. The foundation input response spectra (FIRS) for the horizontal direction in the free-field at the foundation level of the Nuclear Island Common Basemat Structures has a peak ground acceleration of 0.076 g. Therefore, a Site SSE was developed for the design and analysis of structures.

The Site SSE ground motion for CCNPP Unit 3 is the envelope of the U.S. EPR FSAR European Utility Requirements (EUR) Soft Soil spectrum anchored at 0.15 g and the horizontal RG 1.60 spectrum anchored at 0.1 g, therefore satisfying the requirements of Appendix S of 10 CFR Part 50. The Site SSE ground motion, which is specified for both horizontal and vertical directions, is presented FSAR Section 3.7.1.1.1. FSAR Section 3.7.1.1.1 and Section 2.5.4.7 were revised by UniStar in letter UN#09-519[1]. The revised Section 2.5.4.7 and Figures 3.7-1, 2 and 3 are provided below and show the relationship of the Site SSE to the GMRS.

2.5.4.7 Response Of Soil And Rock To Dynamic Loading

The spectra developed in Section 2.5.2.6 and its specific location at a free ground surface reflect the seismic hazard in terms of PSHA and geologic characteristics of the site and represent the site-specific ground motion response spectrum. These spectra are modified to develop ground motion for design considerations. Detailed descriptions on response of site soils and rocks to dynamic loading are addressed in Section 2.5.2.

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1 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Section 3.7 and response to FSAR Section 3.7 RAI sets 19, 25, 58, 63, 65, 112, 113, 139, 158, 159, 167, 168, 179, 180, 181, and 193," Letter UN#09-519, dated December 29, 2009.

Figure 3.7-1 — {CCNPP Unit 3 Site SSE Spectrum (0.15g PGA), 5% damping}

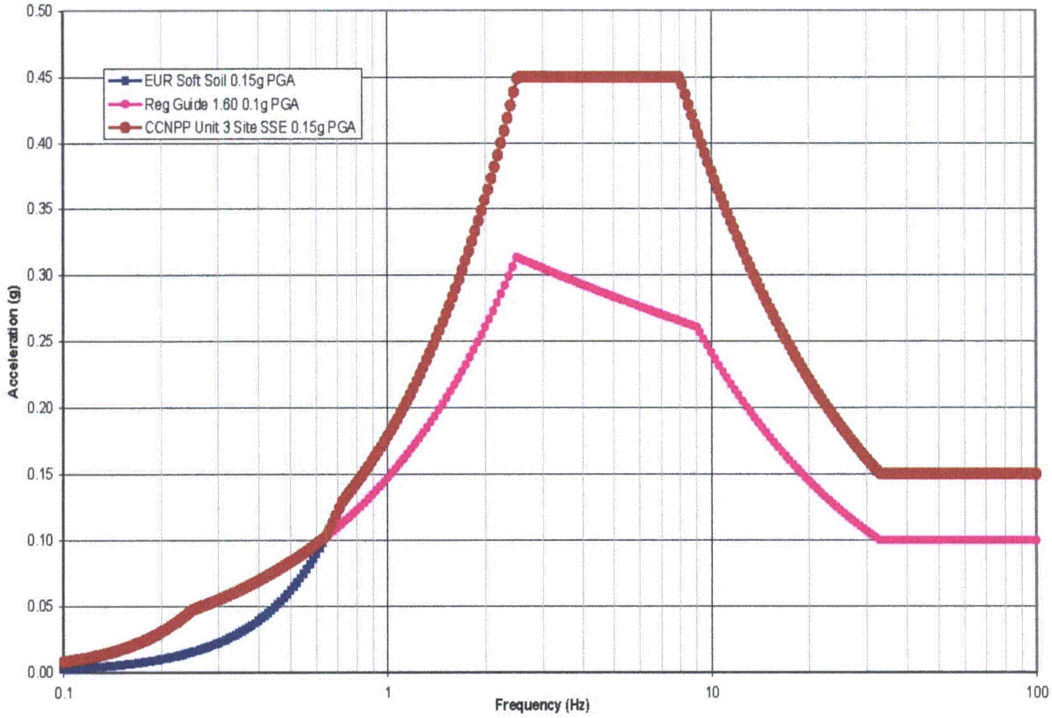


Figure 3.7-2—{CCNPP Unit 3 GMRS (Horizontal) and CCNPP Unit 3 Site SSE Spectrum, 5% damping}

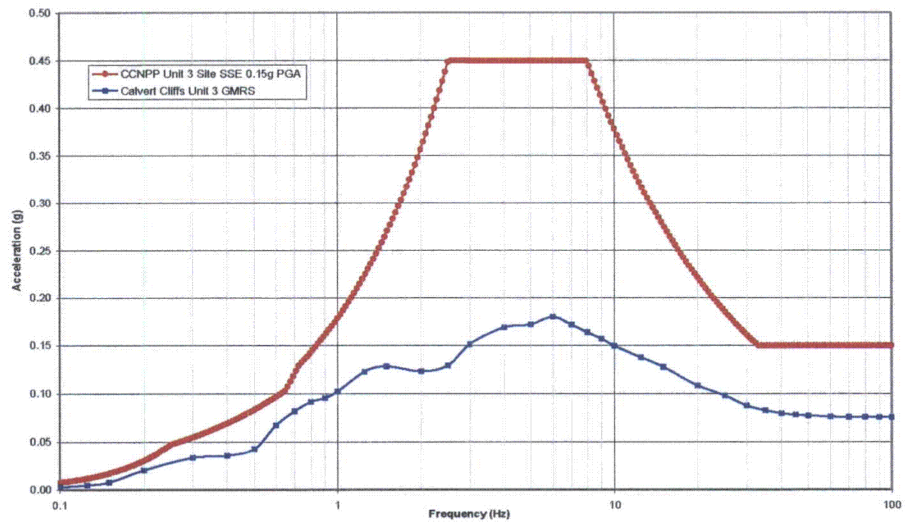
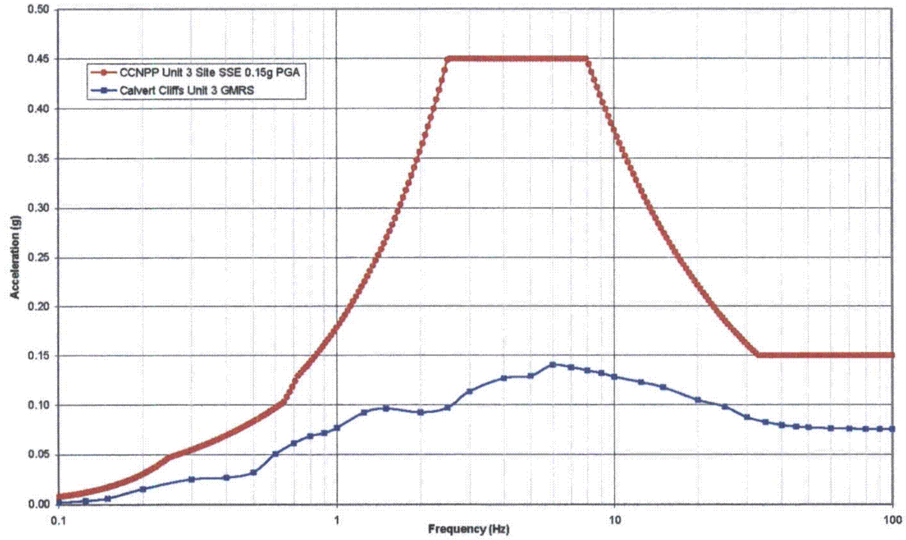


Figure 3.7-3—{CCNPP Unit 3 GMRS (Vertical) and CCNPP Unit 3 Site SSE Spectrum, 5% damping}



COLA Impact

The COLA FSAR will not be revised as a result of this response. Required COLA revisions were previously submitted in UniStar letter UN#09-519[1].

RAI No. 240

Question 02.05.04-22

FSAR Subsection 2.5.4.2.5.9 states that for soils beyond 1000 ft in depth, EPRI curves were extrapolated from the EPRI sand curves that have a range of 1000 ft in order to characterize deeper soils. Provide a basis for the extrapolation method, a figure showing examples of the shear modulus and damping curves for depths below 1000 ft, and a description of the impact of variation of the extrapolated curves on the site response analysis. Please appropriately update the FSAR with the requested information. This request is in accordance with 10 CFR 100.23.

Response

A detailed description of the Resonant Column Torsional Shear (RCTS) curve fitting process and extrapolation process is provided in the report "Reconciliation of EPRI and RCTS Results, Calvert Cliffs Nuclear Power Plant Unit 3," and is included as COLA Part 11J.

During the CCNPP Unit 3 Site Investigation, RCTS tests were performed on soils collected from the upper 400 feet of the site. RCTS tests were not performed on soils below 400 feet, since the boring depths were limited to such depth, and therefore, soils samples were not available for testing.

These deeper soils, in descending order, are the Marlboro Clay, Aquia/Brightseat Sand, Patapsco Sand, and the Patuxent/Arundel Clay.

To assess their utilization, EPRI curves initially adopted for these soils were compared with the set of curves derived from the RCTS results for the upper soils, as shown in FSAR Figure 2.5-401. The results indicate that:

- Marlboro Clay and Patuxent/Arundel Clay Curves: the EPRI curves are identical and fall nearly half-way between the RCTS-based curves for the Stratum I Sand (Curve 3) and Strata II and III soils (Curve 2) in their G/Gmax relationship and closer to Curve 3 in their damping relationship. Based on the available RCTS results, it is inconceivable for these soils at such great depths (and expected high strength) to behave as "softly" as Stratum I Sand (Curve 3) which is at relatively shallow depths and primarily non-plastic. Therefore, as a minimum, the Marlboro and Patuxent/Arundel clays are expected to behave closer to that represented by Curve 2. On this basis, Curve 2 is a reasonable representation for these soils and is used for the dynamic characterization of Marlboro Clay and Patuxent/Arundel Clay.

- Aquia/Brightseat Sand and Patapsco Sand: the EPRI curves are nearly identical and follow Curve 2 closely in their G/Gmax and damping relationship. Based on the RCTS results, and given their depths, these soils are expected to behave somewhere in the region represented by Curves 1 and 2, and possibly closer to Curve 1. Given that a number of the RCTS tests on sandy soils banded closely and were represented by Curve 2, the deeper sandy soils of the Aquia/Brightseat and Patapsco are expected to produce relationships that are mimicked by Curve 2, as a minimum. On this basis, Curve 2 is a reasonable representation for these soils and is used for the dynamic characterization of Aquia/Brightseat Sand and Patapsco Sand.

The calculated maximum strains based on the initially adopted EPRI curves for soils below 1000 feet are in the 10^{-2}% to 10^{-3}% range for the 1E-4 and 1E-5 rock input motions, respectively, as shown in FSAR Figure 2.5-402. At such strain levels, the difference between the EPRI-based and RCTS-based curves are minor to insignificant, as evident in FSAR Figure 2.5-401. Therefore the potential impact of variation of the extrapolated curves on the site response analysis is negligible and is conservatively covered by the randomization of the soil column and strain dependant properties as described in Section 2.5.2.

COLA Impact

The following changes will be made in FSAR Section 2.5.4.2.5.9 to incorporate the information described above. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[2] on October 9, 2009.

2.5.4.2.5.9 Strain Dependant Properties

The strain dependant properties for the CCNPP3 project are developed by fitting generic curves to the site specific data reported by RCTS tests. EPRI curves from EPRI TR-102293 were used as generic curves (EPRI, 1993). EPRI "sand" curves were used for predominately granular soils and "clay" curves were used for predominately clay soils based on estimated PI values. The EPRI "sand" curves cover a depth range up to 1,000 ft. Since soils at the CCNPP site extend beyond 1,000 ft, similar curves were extrapolated from the EPRI curves, extending beyond the1,000-ft depth, to characterize the deeper soils. For instance, the "1,000-2,000 ft" curve was extrapolated by "off-setting" this curve by the amount shown between the "250-500 ft" and "500-1,000 ft" curves in EPRI TR-102293 (EPRI, 1993). To assess the adequacy of EPRI curves for the deeper soils, these were compared with the set of curves derived from the RCTS results for the upper soils, as shown in Figure 2.5-401. The comparison indicates that:

- Marlboro Clay and Patuxent/Arundel Clay Curves: the EPRI curves are identical and fall nearly half-way between the RCTS-based curves for the Stratum I Sand (Curve 3) and Strata II and III soils (Curve 2) in their G/Gmax relationship and closer to Curve 3 in their damping relationship. Based on the available RCTS results, it is inconceivable for these soils at such great depths (and expected high strength) to behave as "softly" as Stratum I Sand (Curve 3) which is at relatively shallow depths and primarily non-plastic. Therefore, as a minimum, the Marlboro and Patuxent/Arundel clays are expected to behave closer to that represented by Curve 2. On this basis. Curve 2 is a reasonable representation for these soils and is used for the dynamic characterization of Marlboro Clay and Patuxent/Arundel Clay.

- Aquia/Brightseat Sand and Patapsco Sand: the EPRI curves are nearly identical and follow Curve 2 closely in their G/Gmax and damping relationship. Based on the RCTS results, and given their depths, these soils are expected to behave somewhere in the region represented by Curves 1 and 2, and possibly closer to Curve 1. Given that a number of the RCTS tests on sandy soils banded closely and were represented by Curve 2, the deeper sandy soils of the Aquia/Brightseat and Patapsco are expected to produce relationships that are mimicked by Curve 2, as a minimum. On this basis,

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2 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

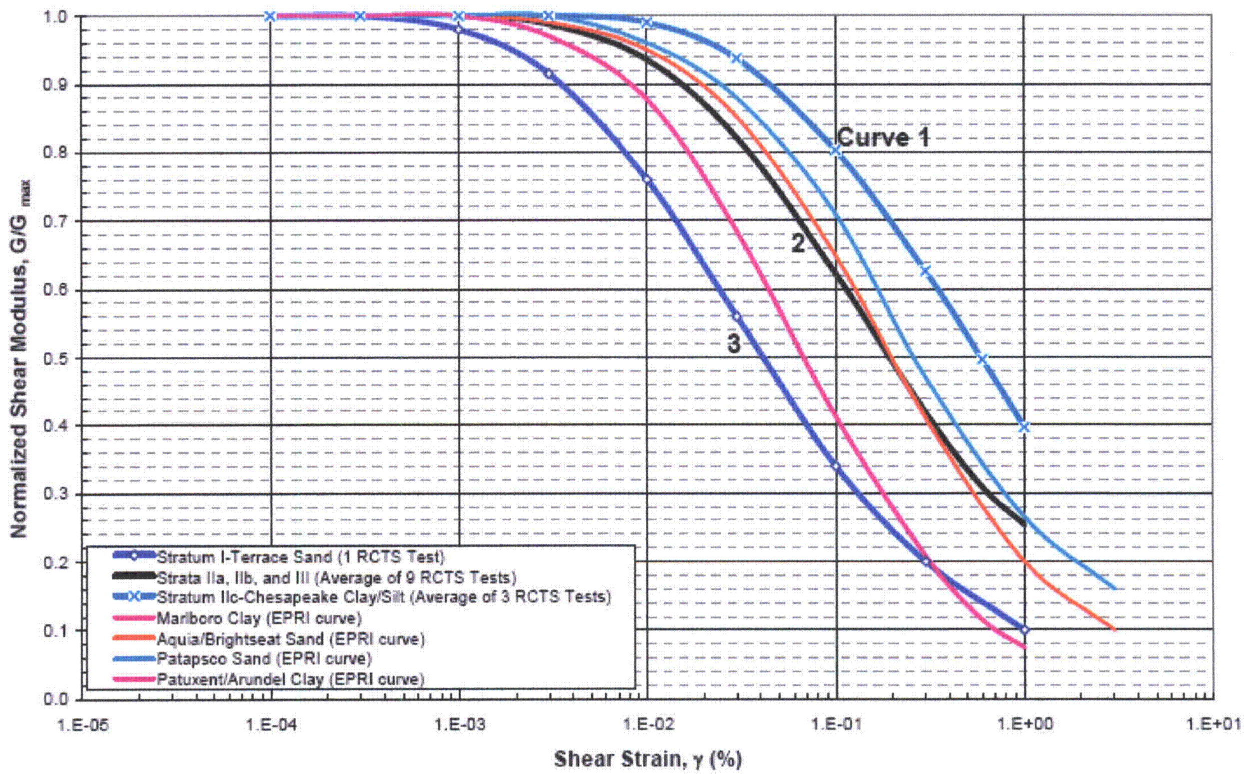
Curve 2 is a reasonable representation for these soils and is used for the dynamic characterization of Aguia/Brightseat Sand and Patapsco Sand.

The calculated maximum strains based on the initially adopted EPRI curves for soils below 1000 feet are in the 10^{-2}% to 10^{-3}% range for the 1E-4 and 1E-5 rock input motions, respectively, as shown in Figure 2.5-402. At such strain levels, the difference between the EPRI-based and RCTS-based curves are minor to insignificant as evident in Figure 2.5-401. Therefore the potential impact of variation of the extrapolated curves on the site response analysis is negligible and is conservatively covered by the randomization of the soil column and strain dependant properties as described in Section 2.5.2.

EPRI curve selection for the upper 400 ft of the site soils was based on available soil characterization data from the site investigation.

FIGURE 2.5-401 –

{Selection of Shear Modulus and Damping Ratios for Soils Deeper than 400 Feet}



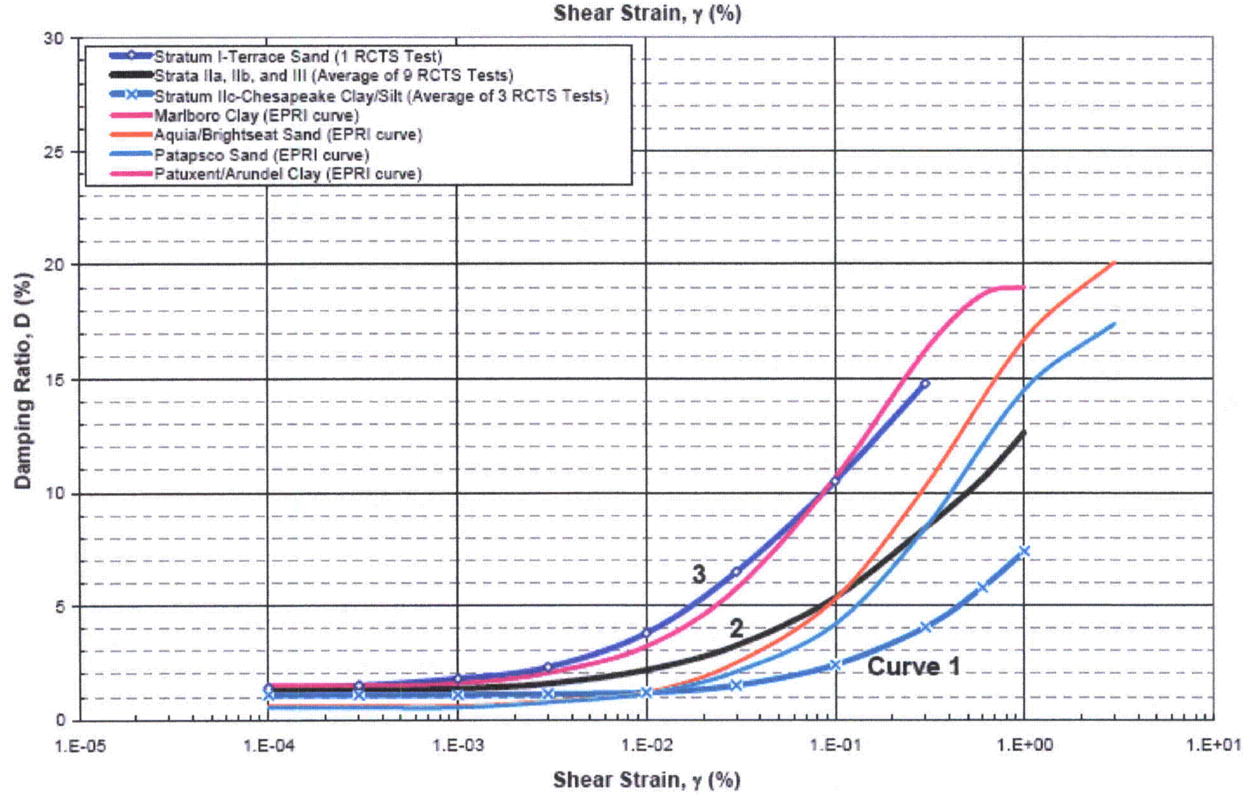
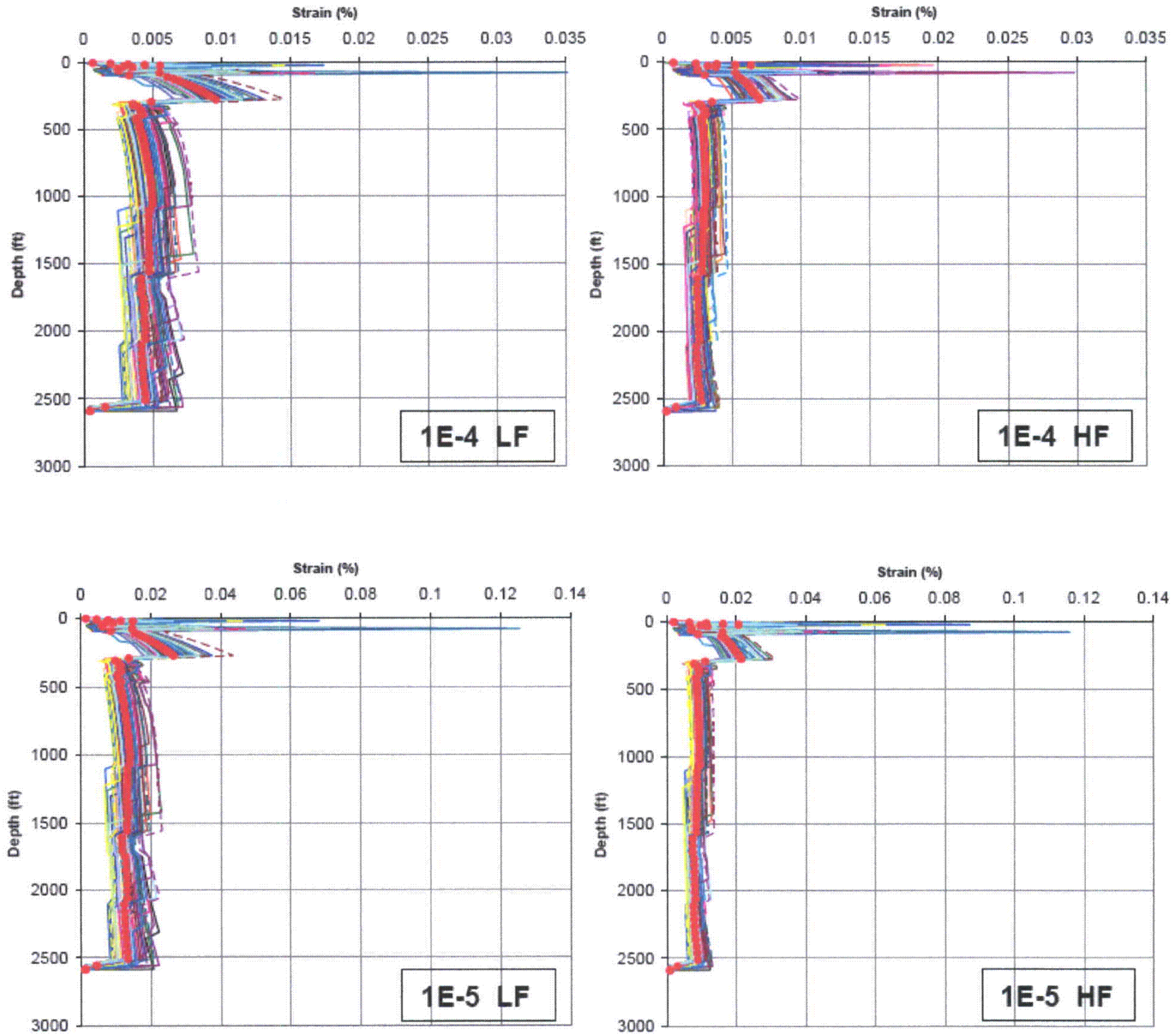


FIGURE 2.5-402

{Calculated Maximum Strains Based on Initially Adopted EPRI Curves}



RAI No. 240

Question 02.05.04-23

FSAR Section 2.5.4.8.2 states that a PGA of 0.15g and that a magnitude 6.0 earthquake was used for the liquefaction analyses.

a. Please justify why a larger magnitude earthquake representing the low-frequency distant controlling earthquake, for example 7.0 at 0.10 g, was not considered for the liquefaction analysis.

b. Regarding the CPT data used in liquefaction analysis, the depths associated with sleeve and tip resistance do not match each other. Please explain this discrepancy.

Response

(Sub-question a)

The following discussion provides information related to the impact of a distant seismic event. The deaggregation results from the CCNPP Unit 3 Probabilistic Seismic Hazard Analysis (PSHA) indicate that there is contribution from distant events at low frequency. The controlling ground motion for the distant event scenario corresponds to a 6.9 magnitude earthquake. It is meaningful to examine the impact that a 6.9 magnitude earthquake, with a maximum Peak Ground Acceleration of 0.1 g, has on the factor of safety (FS) against liquefaction. The 0.1 g value, which is higher than the site specific GMRS (0.08g PGA), is used as the minimum acceleration level according to Appendix S of 10 CFR Part 50. The FS against liquefaction may be expressed as:

$$FS = \frac{CRR_{7.5}}{CSR} MSF \qquad (1)$$

Where:

$CRR_{7.5}$ is the cyclic resistance ration for magnitude 7.5 earthquakes,
CSR is the calculated cyclic stress ratio generated by earthquake shaking,
MSF is the magnitude scaling factor,

For earthquake magnitude less than 7.5, the MSF is obtained by averaging the upper-bound MSF by Andrus and Stokoe, and the lower-bound MSF by Idriss. For earthquake magnitude greater than 7.5, the recommended MSF by Idriss is used. The value of the MSF is as follows:

MAGNITUDE	MSF
5.5	2.5
6.0	1.93
6.5	1.52
7.0	1.22
7.5	1
8.0	0.84
8.5	0.72

The CSR can be expressed as:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d \qquad (2)$$

Where:

- a_{max} - peak horizontal acceleration at the ground surface in terms of g
- g - the acceleration due to gravity
- $\sigma_{vo}, \sigma'_{vo}$ - total and effective vertical overburden stresses, respectively, and
- r_d - stress reduction coefficient

For specific field conditions, location and depth, the CSR is proportional to the maximum acceleration (as all other parameters in Equation 2 are constant for site specific locations), and $CRR_{7.5}$ is constant. Therefore from Equation 1:

$$FS \propto \frac{MSF}{a_{max}} \text{ and } FS = k\frac{MSF}{a_{max}} \qquad (3)$$

Where k is an arbitrary constant that depends on depth, site conditions, location and other geotechnical parameters. The value of the FS, as a function of MSF is then:

MAGNITUDE	a_{max} [g]	FS
6.0	0.15	$k\frac{1.93}{0.15} = 12.86\,k$
6.9	0.10	$k\frac{1.46}{0.1} = 14.60\,k$
7.0	0.10	$k\frac{1.22}{0.1} = 12.20k$
7.0	0.084	$k\frac{1.22}{0.084} = 14.50\,k$

The previous results indicate that for a magnitude earthquake of 6.9 the FS would be enhanced by about 14%, when compared to the design basis analysis (M = 6.0, a_{max} = 0.15 g). For a maximum acceleration of 0.10 g and a magnitude earthquake of 7.0, which is higher than the deaggregated controlling ground motion, the change in FS is negligible.

<u>(Sub-question b)</u>

Reported depths were obtained from digitization of CPT tests results. Different depth locations in the resistance profiles were selected during the digitization process. The use of different data point depths has no impact in the development and conclusions of the liquefaction analysis.

COLA Impact

The following changes will be made in FSAR Section 2.5.4.8.2 to incorporate the information described above. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[2] on October 9, 2009 as modified by UNE Letter UN#09-519[1].

2.5.4.8.2 Soil and Seismic Conditions For CCNPP Unit 3 Liquefaction Analysis

Preliminary assessments of liquefaction for the CCNPP Unit 3 soils were based on observations and conclusions contained within CCNPP Units 1 and 2 UFSAR (BGE, 1982). The site soils that were investigated for the design and construction of CCNPP Units 1 and 2 did not possess the potential to liquefy. Given the relative uniformity in geologic conditions between existing and planned units, the soils at CCNPP Unit 3 were preliminarily assessed as not being potentially liquefiable for similar ground motions, and were further evaluated for confirmation, as will be described later in this subsection. Based on this assessment, it was determined that aerial photography as outlined in Regulatory Guide 1.198 (USNRC, 2003c) would not add additional information to the planning and conduct of the subsurface investigation; therefore, was not conducted.

A common stratigraphy was adopted for the purpose of establishing soil boundaries for liquefaction evaluation. The adopted stratigraphy was that shown generically in Figure 2.5-106 and also by the velocity profiles shown in Figure 2.5-167 and Figure 2.5-169. Only soils in the upper 400 ft of the site were evaluated for liquefaction, based on available results from the CCNPP Unit 3 subsurface investigation. Soils below a depth of 400 ft are considered geologically old and sufficiently consolidated. These soils are not expected to liquefy, as will be further discussed in Section 2.5.4.8.4.

The liquefaction analysis was performed using a peak ground acceleration (PGA) of 0.15 g from the Site Safe Shutdown Earthquake (SSE) developed in Section 3.7.1. <u>A sensitivity calculation was developed to study the impact that a distant, higher magnitude event, with lower acceleration would have in the Factor of Safety against liquefaction. The controlling distant event with magnitude 6.9 was used along with a maximum ground acceleration of 0.1g. The sensitivity analysis indicates that the Factor of Safety against liquefaction is about 14% larger for such scenario.</u>