Mark T. Finley
Senior Vice President, Regulatory Affairs & Engineering

750 East Pratt Street, Suite 1400
Baltimore, Maryland 21202

UniStar
NUCLEAR ENERGY

10 CFR 50.4
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July 31, 2013

UN#13-097

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 353, Determining the Technical Adequacy of Probabilistic Risk Assessment
Results for Risk-Informed Activities

References:
1) Surinder Arora (NRC) to Paul Infanger (UniStar Nuclear Energy), "CCNPP3 - Final RAI 353 SEB2 6477," dated June 11, 2012

2) UniStar Nuclear Energy Letter UN#13-006, from Mark T. Finley to Document Control Desk, U.S. NRC, Updated RAI Closure Plan, dated January 30, 2013

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 June 11, 2012 (Reference 1). This RAI addresses the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities, as discussed in Section 19.1 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 9.

Reference 2 indicated that a response to RAI 353, Question 19.01-1 would be provided to the NRC by July 31, 2013. Enclosure 1 provides our response to RAI 353, Question 19.01-1, and includes revised COLA content. Enclosure 2 provides the COLA impact of the response to RAI 353, Question 19.01-1. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

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Enclosure 3 provides a table of changes to the CCNPP Unit 3 COLA associated with the RAI 353, Question 19.01-1 response. As identified in the Enclosure 3 Table of Changes, this response supplements previously submitted RAI 160 and 313 responses.

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

If there are any questions regarding this transmittal, please contact me at (410) 369-1907 or Mr. Wayne A. Massie at (410) 369-1910.

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

Executed on July 31, 2013

Mark T. Finley

Enclosures:
1) Response to NRC Request for Additional Information RAI 353, Question 19.01-1, Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities, Calvert Cliffs Nuclear Power Plant, Unit 3
2) Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 353, Question 19.01-1, Calvert Cliffs Nuclear Power Plant, Unit 3
3) Table of Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 353, Question 19.01-1, Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn-Willingham, NRC Environmental Project Manager, U.S. EPR COL Application
Tomeka Terry, NRC Environmental Project Manager, U.S. EPR COL Application
Amy Snyder, NRC Project Manager, U.S. EPR DC Application, (w/o enclosures)
Patricia Holahan, Acting Deputy Regional Administrator, NRC Region II, (w/o enclosures)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2,
David Lew, Deputy Regional Administrator, NRC Region I (w/o enclosures)

Enclosure 1

Response to NRC Request for Additional Information RAI 353, Question 19.01-1, Determining the Technical Adequacy of Probabilistic Risk Assessment Results for Risk-Informed Activities, Calvert Cliffs Nuclear Power Plant, Unit 3

RAI No. 353

Question 19.01-1

<u>This RAI Question is supplementary to previous RAI 313, Question 19-26</u>.

The staff has reviewed the response to RAI 313, Question 19-26 and finds it inadequate with respect to the following:

1. The response stated that the GMRS is anchored to a PGA of 0.076q which is inconsistent with a previous response to RAI 160, Question 19-19 which stated that the CCNPP Unit 3 ground motion response spectra (GMRS) peak ground acceleration is 0.084 g.
2. The PRA-based SMA performed for the EPR design certification (DC) application includes a seismic model with accidence sequences which addressed the DC portion of EPR plant. In accordance with ISG-20 (ML1004912330), the scope of COL application is to update the DC system model to incorporate the COL portion of the EPR plant and to reflect the site-specific and plant specific features. The response only made a statement "Possible effects of site soil failures on the U.S. EPR FSAR accident sequences need not be assessed." But it did not address other aspects of system model update such as plant specific design features within the scope of COL application. In addition, the COL updating cannot be completed until the DC PRA-based analysis is completed.
3. The DC fragility analysis is based on the assumed site parameters and typically does not consider soil related failure modes. Therefore, the HCLPF (high confidence low probability of failure) capacity for the EPR DC portion of Structures, Systems and Components (SSCs) was established at 0.5g PGA (Based on CSDRS shapes). It is not clear in the response whether the applicant is confirming the plant level seismic capacity at 1.67 times CSDRS HCLPF capacity or at the site-specific 1.67 times GMRS HCLPF capacity for the Calvert Cliff site.
4. The DC fragility analysis establishes the HCLPF capacity for the SSCs in system model at 1.67 times CSDRS using a generic site which does not address site-specifc soil failures. Therefore, for soil site, the applicant needs to confirm whether the DC HCLPF calculations remain valid for the site or update them using the site-specific GMRS to confirm the plant level HCLPF at 1.67 times GMRS. It is not clear from the response that the applicant has done it. In addition, AREVA has not yet completed the DC fragility analysis for the SSCs. Therefore, it may not be possible for the applicant to perform the fragility update until the DC's analysis is complete.

The staff requests that the applicant provide additional information to address the issues raised above and provide auditable references to analyses/calculations in support of the updating process. The staff requests that the applicant provide a proposed markup of FSAR changes for the staff review.

Response:

ITEM 1:

The Probabilistic Seismic Hazard Analysis (PSHA) for the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 has been updated to incorporate the recently developed 2012 Central and Eastern United States (CEUS) Seismic Source Characterization (EPRI/DOE/NRC, 2012). Both the previously reported values of the Peak Ground Acceleration (PGA) of the Ground Motion Response Spectra (GMRS), 0.076 g and 0.084 g, have been superseded by the result obtained with the use of the 2012 CEUS characterization. The updated and current value of the PGA associated to the CCNPP Unit 3 GMRS is 0.115 g. For illustration purposes, Figure 1 provides the GMRS of the CCNPP Unit 3.

The updated GMRS was provided as part of the response to RAI 345, Question 02.05.02-24[1].

ITEM 2:

In accordance with ISG-20, the need for an update of the U.S. EPR Seismic Margin Analysis (SMA) has been evaluated for the CCNPP Unit 3 site. The scope of CCNPP Unit 3 related SMA update includes the following site specific effects and plant specific components:

- Site specific effects
  - Liquefaction
  - Slope stability
  - Nuclear Island (NI) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Emergency Power Generation Building (EPGB) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Essential Service Water Building (ESWB) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Nuclear Auxiliary Building (NAB) related soil effects
    - Sliding Stability/Gap distance between the NAB and the Safeguards and Fuel Buildings of the NI
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Access Building (AB) related soil effects
    - Sliding Stability/Gap distance between the AB and the Safeguards Buildings of the NI
    - Seismic induced bearing pressures over soil dynamic bearing capacity
    - Turbine Island (TI) related soil effects (Includes Switchgear and Turbine Buildings)
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity

[1] UniStar Nuclear Energy Letter UN#12-102, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAIs 284 and 322, Vibratory Ground Motion, RAI 345, Vibratory Ground Motion dated September 27, 2012

- Plant specific Category I and II structures
  - Makeup Water Intake Structure (MWIS)
  - Turbine Island (TI)
  - Access Building (AB)

- Plant specific seismically qualified safety related components
  - Buried pipes in the vicinity of the Powerblock Area
  - Components in the MWIS

As indicated in Section 19.1.5 of the CCNPP Unit 3 Final Safety Analysis Report (FSAR), makeup to the ESWB cooling towers is not credited in the Design Certification (DC) Probabilistic Risk Assessment (PRA). Therefore an update of the U.S. EPR PRA does not require the fragility or margin analysis of the CCNPP Unit 3 MWIS, the pipeline corridor between the MWIS and the Powerblock Area, or the soil effects associated to these components.

The Category II TI houses the Station Blackout (SBO) Diesels and is designed to Category I seismic requirements. Therefore, its design will ensure that the Plant Level High Confidence Low Probability of Failure (HCLPF) of 1.67 is maintained.

The NAB and AB are also Category II structures designed to meet Category I seismic design criteria and do not house any risk significant or safety-related components. Therefore, only the effects related to soil interface (sliding and soil bearing failure) may impact the Plant Level HCLPF. The response to ITEM 4 addresses the soil effects related to the NAB and AB.

Given the low seismicity of the CCNPP Unit 3 site with respect to the U.S. EPR Certified Seismic Design Response Spectra (CSDRS) (Figure 2), the CCNNP Unit 3 SMA update approach is based on the screening of rugged site specific effects and plant specific components of the CCNPP Unit 3 site. The screening considers the GMRS, or Foundation Input Response Spectra (FIRS), as applicable, with the PGA scaled by a factor of 1.67. A successful screening of the site specific effects and plant specific components decouples such effects and components from the U.S. EPR SMA and, therefore, it is possible to develop the CCNPP Unit 3 site specific SMA update independent of the finalization of the U.S. EPR PRA.

ITEM 4 to this response provides the description of the screening approach, including quantification of stability parameters that establish the available seismic margin.

<u>ITEM 3:</u>

The HCLPF capacity for the CCNPP Unit 3 Structures, Systems and Components (SSCs) that are part of the U.S. EPR generic design is established at 1.67 times CSDRS (0.5 g PGA). The CCNPP Unit 3 meets this requirement since the site specific In Structure Response Spectra (ISRS) are bounded by the U.S. EPR ISRS, except at very low frequencies. Civil structures and their design are not impacted by the demands that correspond to very low frequencies and therefore the US EPR SMA analysis is bounding for CCNNP Unit 3.

The low frequency exceedances in regard to site specific effects and plant specific components are discussed in ITEM 4 to this response.

For site specific effects and plant specific components, according to ISG-20, the HCLPF capacity for the CCNPP Unit 3 site is established at 1.67 times the GMRS. In particular instances, such as sliding analysis of buildings, the FIRS are conservatively used instead of the GMRS. The response to ITEM 4 provides the rationale for the use of FIRS instead of the GMRS.

ITEM 4:

Site specific effects and plant specific components are demonstrated to be rugged for 1.67 times GMRS seismic demand and therefore the DC HCLPF calculations remain valid. The following paragraphs describe the screening process.

The CCNPP Unit 3 SMA update approach is based on the screening of site specific and plant specific information for the CCNPP Unit 3 site. The analyses and results shown in the following paragraphs indicate that the site specific effects and plant specific components are screened as rugged items based on the GMRS (or FIRS), with the PGA scaled by a factor of 1.67. Therefore it is possible to provide the CCNPP Unit 3 site specific SMA update independent of the timing of the U.S. EPR PRA.

In order to describe the screening of each of the site effects and plant components listed in ITEM 2, it is required to indicate that the SSE is higher than the GMRS and related FIRS. Therefore some level of margin is already built into the analysis. The Factor of Safety (FOS) that meets the required margin for the CCNPP Unit 3 site is:

$$FOS = \frac{1.67 \times PGA_{GMRS}}{PGA_{SSE}} = \frac{1.67 \times 0.115}{0.150} = \frac{0.192}{0.150} = 1.28$$

Where:

FOS	→	Factor of Safety
PGA_{GMRS}	→	Peak ground acceleration of the CCNPP Unit 3 GMRS
PGA_{SSE}	→	Peak ground acceleration of the CCNPP Unit 3 SSE

LIQUEFACTION

The details of the calculations that establish the potential for liquefaction are included in Section 2.5.4.8 of the CCNPP Unit 3 FSAR. The FOS to prevent liquefaction does not have a linear relationship to the PGA. Therefore, the 0.15 g analysis presented in Section 2.5.4.8 was supplemented with a counterpart that uses an input PGA of 0.192 g, and an earthquake magnitude of 6. Both analyses indicate that liquefaction potential is only present for very limited locations of the surface terrace sands. These sands are therefore to be removed and replaced by engineered backfill material that is not susceptible to liquefaction. Consequently, even for seismic ground motion consistent with 1.67 x PGA of the GMRS, there is no potential for liquefaction at the CCNPP Unit 3 site.

SLOPE STABILITY

For slope stability, since the CCNPP Unit 3 analysis was performed for an acceleration level of 0.15 g (FSAR Section 2.5.5) a Factor of Safety (FOS) against sliding that meets the required seismic margin for screening of effects and components is:

$$FOS = \frac{1.67 \times 0.115}{0.150} = 1.28$$

As described in FSAR Section 2.5.5, a pseudo-static analysis is used for two cases: total stress and effective stress. Several slope sections are evaluated for stability.

A pseudo-static analysis is used to incorporate seismic forces into the slope stability analysis. Total stress conditions are representative of dynamic conditions at the site, since pore water pressures do not have time to dissipate.

The lowest FOS reported in FSAR Section 2.5.5 (for 0.15g horizontal motion) is 1.48. This exceeds the required FOS of 1.28. Therefore, it is concluded that the FOS for slope stability meets the required margin of 1.67 times the GMRS.

BUILDING RELATED SITE SOIL EFFECTS

For site soil effects that correspond to sliding analysis and dynamic bearing pressure demands of buildings, the building-specific FIRS are used instead of the GMRS. The FIRS are the seismic design basis that are used for the building analyses and are generally higher than the GMRS. Therefore, lesser margin is available for these site effects.

Table 1 provides the FIRS PGA levels for each building, the required seismic margin, and the overall required FOS, which is calculated as follows:

$$FOS = \frac{1.67 \times PGA_{FIRS}}{PGA_{SSE}}$$

Where:

FOS	→	Factor of Safety
PGA_{FIRS}	→	Peak ground acceleration of the building specific FIRS
PGA_{SSE}	→	Peak ground acceleration of the CCNPP Unit 3 SSE

The 0.15 g level SSE is used for the sliding and bearing demands analyses of the NI, the NAB, and the AB (NI/NAB/AB). For the case of the EPGB, the ESWB, and the TI, additional margin was incorporated into the analysis with the use of higher than anticipated Structure to Soil to Structure Interaction (SSSI) effects. Details of the calculation of FIRS and the incorporation of SSSI effects are described in Section 3.7.1 and 3.7.2 of the CCNPP Unit 3 FSAR. Table 1 provides the building specific seismic margins that are required for screening as well as their associated FOS target values.

The Category II Buildings that are adjacent to the NI (NAB, AB) are included since soil related sliding and bearing failures of these structures may potentially impact the stability of the adjacent Category I structures. These Category II structures are analyzed and designed to Seismic Category I design criteria.

Table 2 provides the seismic margins related to each particular site effect. The values reported in Table 2 originate from the analysis performed with the seismic design basis input.

As indicated in Table 2, the design basis FOS to prevent sliding of the AB does not meet the required margin. However, the design basis sliding analysis of the AB assumes several conservatisms that allow one to conclude that the actual FOS is higher than the reported value of 1.38.

The FOS to prevent sliding for the AB was obtained assuming that:

- Seismic forces, from all directions, act with the highest acceleration demand occurring at the same point in time and towards the same direction; a pseudo-static approach was used to incorporate seismic forces using the Zero Period Acceleration (ZPA) obtained from a dynamic Soil Structure Interaction (SSI) analysis.
- Uplift forces include buoyancy and the vertical seismic force; the vertical seismic force is assumed to act upward at all times.
- No side wall friction or adhesion of the walls in contact with the soils is considered.

As discussed below, by removing only the first conservatism listed above, the FOS will increase by more than 35%, from 1.38 to 1.89.

Inter-story forces of the AB are calculated by multiplying the story masses times the ZPA. The maximum base horizontal shear is obtained after performing a Square Root Sum of the Squares (SRSS) combination of the orthogonal ZPA related forces. The resulting base shear is conservative since it is obtained with the assumption that seismic forces act at their maximum magnitude, at the same moment in time, and toward the same direction. The SRSS combination of orthogonal forces always use the maximum ZPA related values of each direction. In order to obtain an estimate of the FOS without the incorporation of this conservatism, horizontal forces are combined at each time step and the maximum force is obtained as the maximum recorded throughout the time history. This approach is referred to as the time history (TH) approach.

Figure 3 illustrates the relationship between the ZPA and the TH approaches by comparing the total base shear. Figure 4 illustrates the relationship between the design basis FOS and the FOS that is estimated from a time history analysis.

The FOS to prevent sliding of the AB exceeds the required target FOS of 1.47 by ample margin. It is therefore concluded that the sliding stability of the AB may be screened out as a rugged component since the FOS is higher than the one required when using a seismic margin level of 1.67 x AB building specific FIRS.

PLANT SPECIFIC COMPONENTS, BURIED PIPES

The seismic design basis calculation of buried pipes is provided in CCNPP Unit 3 FSAR Section 3.0 Appendix 3E.6. The maximum Demand/Capacity ratio reported is 0.5, obtained for an acceleration level of 0.15 g. Since the design of buried pipes is likely controlled by loads other than seismic, the 0.5 ratio suggests that buried pipes have sufficient ruggedness to meet 1.67 times the 0.115g GMRS ZPA. However, since a FOS for this item was not calculated during design basis analysis, a seismic margin for buried pipes was calculated using three representative duct bank sizes analyzed for 0.192 g ZPA. The method of analysis for seismic response accounts for soil-structure interaction (SSI) effect between the duct bank and surrounding soil (the design basis calculation had conservatively ignored this aspect). The dead and soil load responses, which are based on a beam-on-elastic-foundation approach, are taken from the design basis calculation. Moment and shear demands for these loads are combined with respective values obtained for seismic loads.

The layout of duct banks, duct bank sizes, conduit sizes and conduit material are based on information taken from preliminary drawings and remain as now in CCNPP Unit 3 COLA FSAR, Revision 9.

The Seismic Category I Buried Duct Banks for CCNPP Unit 3 have been found to be adequate in design for the increased ZPA of 0.192 g. The duct banks have a minimum Seismic Margin (SM) of 0.37 g, which provides an FOS greater than 3.0 (0.37/0.115). The SM is higher than the required 0.192 g value since the design of buried pipes is heavily influenced by loads other than seismic.

LOW FREQUENCY EXCEEDANCE

Figure 2 indicates that the GMRS exceeds the CSDRS at the very low frequency range ($f \leq 0.3$ Hz). The exceedances are present at the ISRS level to a lesser degree. The seismic demand associated with the site specific effects and plant components is driven by the seismic response of civil structures. Civil structures and their design are not impacted by the demands that correspond to very low frequencies and their associated low acceleration levels. Issues such as structural design, structural sliding, and bearing demand, are controlled by higher frequencies. Other analyses, such as liquefaction and slope stability, are dependent on the PGA only. Therefore, the screening of civil/structural rugged components is not impacted by the exceedances at the very low frequency range.

There are no site specific effects or plant specific components that are susceptible to low frequency ground motion. The U.S. EPR SSCs that are susceptible to low frequency ground motion will meet a HCLPF of 1.67 times the GMRS or CSDRS, whichever is controlling.

The analysis of the site specific soil effects and plant specific components indicates that the DC HCLPF calculations are not affected by the CCNPP Unit 3 site effects and plant specific components. The site specific effects and plant specific components are screened out for ruggedness and do not impact the plant level HCLPF of 1.67 times the GMRS (or FIRS). Since the items are screened out, the DC fragility analysis requires no further update in regard to soil effects and civil/structural components.

TABLE 1: BUILDING SITE SPECIFIC FIRS AND ASSOCIATED SEISMIC MARGINS REQUIRED FOR SCREENING OF SLIDING AND BEARING DEMANDS

BUILDING	FIRS(1) [g]	FIRS(2) (w/SSSI) [g]	SSE(3) (w/SSSI) [g]	REQUIRED MARGIN(4) [g]	FOS_T (5)
NI/NAB/AB	0.135	0.135	0.150	0.225	1.50
EPGB	0.117	0.130	0.181	0.217	1.20
ESWB	0.130	0.144	0.181	0.240	1.33

[1] Building specific FIRS using site specific response analysis

[2] Building specific FIRS using site specific response analysis and SSSI factor(1.105, except NI)

[3] SSE Based outcrop motion at foundation level (includes higher utilized SSSI factor)

[4] Required seismic margin calculated as 1.67 x FIRS

[5] Target FOS with 0.15 g analysis calculated as (1.67 x FIRS / SSE Motion)

TABLE 2: SUMMARY OF SEISMIC DESIGN BASIS AND SCREENING TARGET FOS

CASE	SM_T (1) [g]	FOS_T (2)	FOS_D (3)	SM_{CHECK} (4)	FOS_R (5)
NI Sliding	0.224	1.47	1.79	PASS	>> 1.79
NI Bearing (a)	0.224	1.47	1.57	PASS	>> 1.57
EPGB Sliding	0.216	1.11	1.57	PASS	>> 1.57
EPGB Bearing (b)	0.216	1.11	7.10	PASS	>> 7.10
ESWB Sliding	0.240	1.23	1.99	PASS	>> 1.99
ESWB Bearing (c)	0.240	1.23	3.86	PASS	>> 3.86
NAB Sliding (d)	0.224	NOTE (d)	NOTE (d)	PASS	NOTE (d)
NAB Bearing (d)	0.224	1.47	1.49	PASS	>> 1.49
AB Sliding (e)	0.224	1.47	1.38	< 1.47	1.89
AB Bearing (e)	0.224	1.47	3.22	PASS	>> 3.22
TI Sliding (f)	0.227	NOTE (f)	NOTE (f)	PASS	NOTE (f)

(1) Target seismic margin to screen component

(2) Target Factor of Safety to screen component

(3) Factor of Safety obtained from the analysis for seismic design basis (0.15 g)

(4) 1.67 x HCLPF Check; pass if $FOS_D > FOS_T$

(5) Factor of Safety for rugged component under design basis ground motion level. >> Symbol indicates that FOS_R for component is greater than the design basis value

(a) Allowable dynamic bearing capacity (DBC) is 31.3 ksf, site demand is 23.0 ksf

(b) Allowable DBC is 51.1 ksf (Ref. 8), site demand is 7.2 ksf

(c) Allowable DBC is 41.3 ksf (Ref. 9), site demand is 10.7 ksf

(d) NI/NAB gap is greater than twice the sliding distance plus the displacement related to building rotation (30" gap to accommodate 10" of displacement); allowable DBC is 52.9 ksf (Ref. 10); demand is 35.5 ksf

(e) AB does not slide; NI/AB gap is greater than twice the displacement related to building rotation; allowable DBC is 58.0 ksf ; demand is 18.1 ksf; additional ruggedness was incorporated into FOS_R to show that sufficient margin is available

(f) No sliding; TI GAP is at least 30 ft and sliding/bearing failure does not impact Category I structures

FIGURE 1: Ground Motion Response Spectra (GMRS) of the CCNPP Unit 3

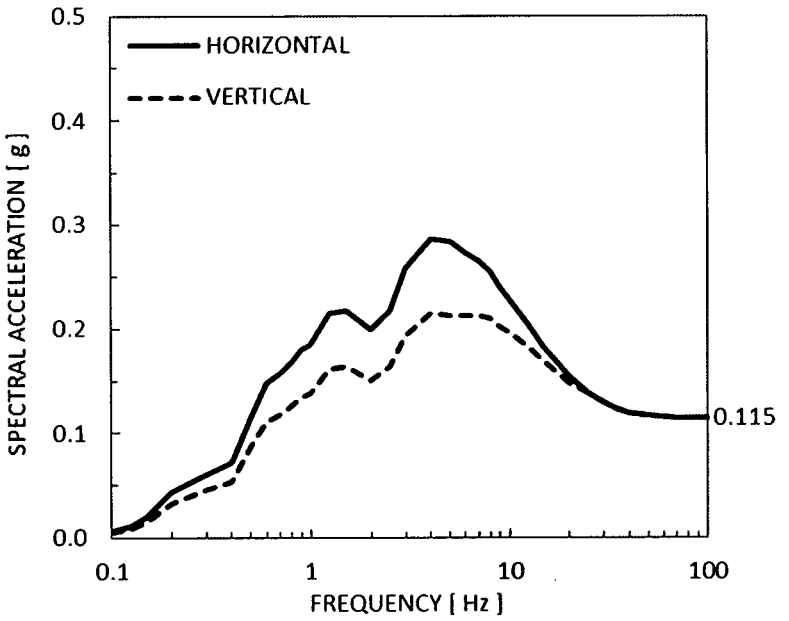


FIGURE 2: CCNPP Unit 3 GMRS (Horizontal) and U.S. EPR Certified Seismic Design Response Spectra (CSDRS)

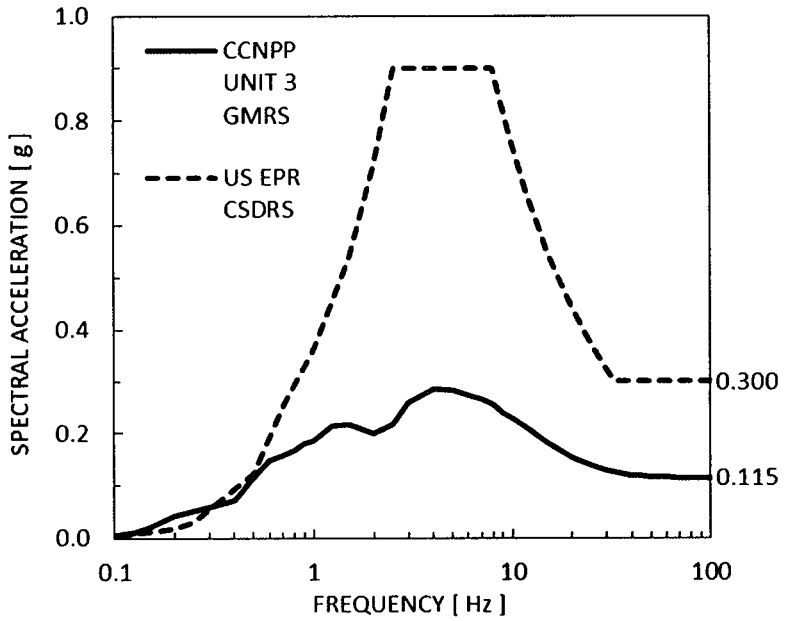


FIGURE 3: AB base shear obtained with the ZPA and TH approaches

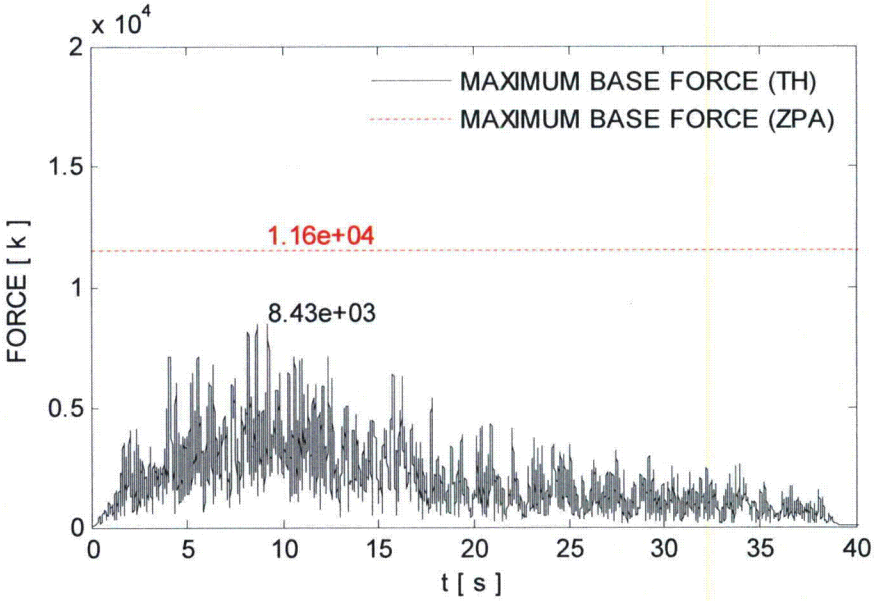
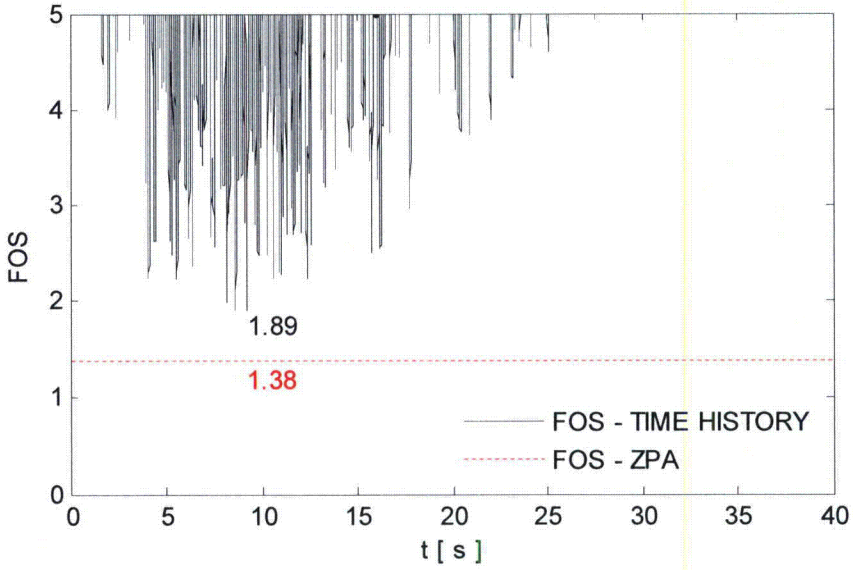


FIGURE 4: AB sliding FOS obtained with the ZPA and TH approaches



COLA Impact

The CCNPP Unit 3 COLA Part 2, FSAR, Sections 19.1.5.1.1.2 and 19.1.5.1.1.5 have been updated, and Tables 19.1-2 and 19.1-3 have been added as indicated in Enclosure 2.

Enclosure 2

Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 353, Question 19.01-1, Calvert Cliffs Nuclear Power Plant, Unit 3

19.1.5.1.1.2 Seismic Hazard Input

{Section 3.7 discusses the GMRS. The GMRS for CCNPP Unit 3 is shown in ~~Figure 3.7-3 and Figure 3.7-5~~ Figure 2.5-81. The PRA-based seismic margin assessment follows the guidance in SECY 93-087 and demonstrates that there is a minimum seismic margin of 1.67 times the GMRS for CCNPP Unit 3. ~~The 1.67 times the GMRS is referred to as seismic margin earthquake (SME) in the combined license.~~}

...

19.1.5.1.1.5 HCLPF Sequence Assessment

~~No departures or supplements.~~

{The HCLPF capacity for the CCNPP Unit 3 Structures, Systems and Components (SSCs) that are part of the U.S. EPR generic design is established at 1.67 times CSDRS (0.5 g PGA). The CCNPP Unit 3 meets this requirement since the site specific In Structure Response Spectra (ISRS) are bounded by the U.S. EPR ISRS, except at very low frequencies.

The DC Plant SSCs that are susceptible to low frequency ground motion will meet a HCLPF of 1.67 the times GMRS or CSDRS, whichever is controlling.}

...

19.1.5.1.2.4 Key Assumptions and Insights

The U.S. EPR FSAR includes the following COL Item in Section 19.1.5.1.2.4:

> A COL applicant that references the U.S. EPR design certification will confirm that the U.S. EPR PRA-based seismic margin assessment is bounding for their specific site, and will update it to include site-specific SSC and soil effects (including sliding, overturning, liquefaction, and slope failure).

This COL Item is addressed as follows:

The PRA-based seismic margins assessment performed for the U.S. EPR FSAR is based on the assumption that the U.S. EPR is designed using the EUR-based certified seismic design response spectra (CSDRS) anchored to a peak ground acceleration (PGA) of 0.3g for selected generic soil profiles. The seismic margins assessment for the U.S. EPR FSAR used CSDRS times 1.67 to define the targeted seismic margin. The seismic margins assessment for the U.S. EPR FSAR remains valid if it can be demonstrated that the U.S. EPR FSAR seismic design parameters bound those for the site-specific seismic characteristics, including the ground motion response spectra (GMRS) and site-specific soil profiles.

~~{A comparison of the CCNPP Unit 3 GMRS versus the CSDRS is provided in Section 3.7.1 and demonstrates that the GMRS anchored to a PGA of 0.076g is much lower than that of the CSDRS, and when the spectra are considered in combination with the site-specific soil characteristics, it is concluded that the seismic demands for CCNPP Unit 3 are much lower than that used for the U.S. EPR FSAR. Therefore, the U.S. EPR FSAR bounds site-specific seismic characteristics and they do not have a significant impact on the CCNPP Unit 3 PRA results and insights.~~

~~Based on the structure seismic stability analyses, the allowable bearing capacities, and the soil failure analyses performed for the 0.15g site SSE, it is concluded that the CCNPP Unit 3 site conditions can withstand a ground motion equal 1.67 x GMRS (0.13g PGA) without inducing soil failures (including sliding, overturning, liquefaction and slope instability).~~

~~Therefore, the plant level high confidence low probability of failure (HCLPF) capacity meets the 1.67 x GMRS criterion.}~~

{For site specific effects and plant specific components, the HCLPF capacity for the CCNPP Unit 3 site is established at 1.67 times the GMRS. In particular instances such as sliding analysis of buildings, the Foundation Input Response Spectra (FIRS) are conservatively used instead of the GMRS.

In accordance with ISG-20, the need for an update of the U.S. EPR Seismic Margin Analysis (SMA) has been evaluated for the CCNPP Unit 3 site. The scope of CCNPP Unit 3 related SMA update includes the following site specific effects and plant specific components:

- Site specific effects
  - Liquefaction
  - Slope stability
  - Nuclear Island (NI) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Emergency Power Generation Building (EPGB) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Essential Service Water Building (ESWB) related soil effects
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Nuclear Auxiliary Building (NAB) related soil effects
    - Sliding Stability/Gap distance between the NAB and the Safeguards and Fuel Buildings of the NI
    - Seismic induced bearing pressures over soil dynamic bearing capacity
  - Access Building (AB) related soil effects
    - Sliding Stability/Gap distance between the AB and the Safeguards Buildings of the NI
    - Seismic induced bearing pressures over soil dynamic bearing capacity
    - Turbine Island (TI) related soil effects (Includes Switchgear and Turbine Buildings)
    - Sliding Stability
    - Seismic induced bearing pressures over soil dynamic bearing capacity

- Plant specific Category I and Category II structures
  - Makeup Water Intake Structure (MWIS)
  - Turbine Island (TI)
  - Access Building (AB)

- Plant specific seismically qualified safety related components
  - Buried pipes in the vicinity of the Powerblock Area
  - Components in the MWIS

As indicated in Section 19.1.5 of the CCNPP Unit 3 Final Safety Analysis Report (FSAR), makeup to the ESWB cooling towers is not credited in the Design Certification (DC) Probabilistic Risk Assessment (PRA). Therefore, an update of the U.S. EPR PRA does not require the fragility or margin analysis of the CCNPP Unit 3 MWIS, the pipeline corridor between the MWIS and the Powerblock Area, or the soil effects associated to these components.

The Category II TI houses the Station Blackout (SBO) Diesels and is designed to Category I seismic requirements. Therefore, its design will ensure that the Plant Level High Confidence Low Probability of Failure (HCLPF) of 1.67 is maintained.

The NAB and AB are also Category II structures designed to meet Category I seismic design criteria and do not house any risk significant or safety-related components. Therefore, only the effects related to soil interface (sliding and soil bearing failure) may impact the Plant Level HCLPF. The soil effects related to the NAB and AB are addressed further in this section.

Given the low seismicity of the CCNPP Unit 3 site with respect to the U.S. EPR CSDRS, the SMA update approach is based on the screening of rugged site specific effects and plant specific components of the CCNPP Unit 3 site. The screening considers the GMRS, or FIRS, as applicable, with the PGA scaled by a factor of 1.67.

Site specific effects and plant specific components are demonstrated to be rugged for 1.67 times GMRS seismic demand.

The CCNPP Unit 3 SMA update approach is based on the screening of site specific and plant specific information for the CCNPP Unit 3 site. The analyses and results shown in the following paragraphs indicate that the site specific effects and plant specific components are screened as rugged items based on the GMRS (or FIRS), with the PGA scaled by a factor of 1.67.

In order to describe the screening of each of the site effects and plant components listed above, it is required to indicate that the SSE is higher than the GMRS and related FIRS and therefore some level of margin is already built into the analysis. Therefore, the Factor of Safety (FOS) that meets the required margin for the CCNPP Unit 3 site is:

$$FOS = \frac{1.67 \times PGA_{GMRS}}{PGA_{SSE}} = \frac{1.67 \times 0.115}{0.150} = \frac{0.192}{0.150} = 1.28$$

Where:

FOS → Factor of Safety

PGA_{GMRS} → Peak ground acceleration of the CCNPP Unit 3 GMRS

PGA_{SSE} → Peak ground acceleration of the CCNPP Unit 3 SSE

LIQUEFACTION

The details of the calculations that establish the potential for liquefaction are included in Section 2.5.4.8 of the CCNPP Unit 3 FSAR. The FOS to prevent liquefaction does not have a linear relationship to the PGA. Therefore, the 0.15 g analysis presented in Section 2.5.4.8 was supplemented with a counterpart that uses an input PGA of 0.192 g, and an earthquake magnitude of 6. Both analyses indicate that liquefaction potential is only present for very limited locations of the surface terrace sands. These sands are therefore to be removed and replaced by engineered backfill material that is not susceptible to liquefaction. Consequently, even for seismic ground motion consistent with 1.67 x PGA of the GMRS, there is no potential for liquefaction at the CCNPP Unit 3 site.

SLOPE STABILITY

For slope stability, since the CCNPP Unit 3 analysis was performed for an acceleration level of 0.15 g (FSAR Section 2.5.5) a Factor of Safety (FOS) against sliding that meets the required seismic margin for screening of effects and components is:

$$FOS = \frac{1.67 \times 0.115}{0.150} = 1.28$$

As described in FSAR Section 2.5.5, a pseudo-static analysis is used for two cases: total stress and effective stress. Several slope sections are evaluated for stability.

A pseudo-static analysis is used to incorporate seismic forces into the slope stability analysis. Total stress conditions are representative of dynamic conditions at the site, since pore water pressures do not have time to dissipate.

The lowest FOS reported in FSAR Section 2.5.5 (for 0.15g horizontal motion) is 1.48. This exceeds the required FOS of 1.28. Therefore, it is concluded that the FOS for slope stability meets the required margin of 1.67 times the GMRS.

BUILDING RELATED SITE SOIL EFFECTS

For site soil effects that correspond to sliding analysis and dynamic bearing pressure demands of buildings, the building-specific FIRS are used instead of the GMRS. The FIRS are the seismic design basis that are used for the building analyses and are generally higher than the GMRS. Therefore, lesser margin is available for these site effects.

Table 19.1-2 provides the FIRS PGA levels for each building, the required seismic margin, and the overall required FOS, which is calculated as follows:

$$FOS = \frac{1.67 \times PGA_{FIRS}}{PGA_{SSE}}$$

Where:

FOS → Factor of Safety

PGA_{FIRS} → Peak ground acceleration of the building specific FIRS

PGA_{SSE} → Peak ground acceleration of the CCNPP Unit 3 SSE

The 0.15 g level SSE is used for the sliding and bearing demands analyses of the NI, the NAB, and the AB (NI/NAB/AB). For the case of the EPGB, the ESWB, and the TI, additional margin was incorporated into the analysis with the use of higher than anticipated Structure to Soil to Structure Interaction (SSSI) effects. Details of the calculation of FIRS and the incorporation of SSSI effects are described in Section 3.7.1 and 3.7.2 of the CCNPP Unit 3 FSAR. Table 19.1-2 provides the building specific seismic margins that are required for screening as well as their associated FOS target values.

The Category II Buildings that are close to the NI (NAB, AB) are included since soil related sliding and bearing failures of these structures may potentially impact the stability of the adjacent Category I structures. These Category II structures are analyzed and designed to Seismic Category I design criteria.

Table 19.1-3 provides the seismic margins related to each particular site effect. The values reported in Table 19.1-3 originate from the analysis performed with the seismic design basis input.

As indicated in Table 19.1-3, the design basis FOS to prevent sliding of the AB does not meet the required margin. However, the design basis sliding analysis of the AB assumed several conservatisms that allow one to conclude that the actual FOS is higher than the reported value of 1.38.

The FOS to prevent sliding for the AB was obtained assuming that:

- Seismic forces, from all directions, act with the highest acceleration demand occurring at the same point in time and towards the same direction; a pseudo-static approach was used to incorporate seismic forces using the Zero Period Acceleration (ZPA) obtained from a dynamic Soil Structure Interaction (SSI) analysis.
- Uplift forces include buoyancy and the vertical seismic force; the vertical seismic force is assumed to act upward at all times.
- No side wall friction or adhesion of the walls in contact with the soils is considered.

As discussed below, by removing only the first conservatism listed above, the FOS will increase by more than 35%, from 1.38 to 1.89.

Inter-story forces of the AB are calculated by multiplying the story masses times the ZPA. The maximum base horizontal shear is obtained after performing a Square Root Sum of the Squares

(SRSS) combination of the orthogonal ZPA related forces. The resulting base shear is conservative since it is obtained with the assumption that seismic forces act at their maximum magnitude, at the same moment in time, and toward the same direction. The SRSS combination of orthogonal forces always use the maximum ZPA related values of each direction. In order to obtain an estimate of the FOS without the incorporation of this conservatism, horizontal forces are combined at each time step and the maximum force is obtained as the maximum recorded throughout the time history. The use of the time history approach indicates that the FOS to prevent sliding of the AB exceeds the required target FOS of 1.47 by ample margin. It is therefore concluded that the sliding stability of the AB may be screened out as a rugged component since the FOS is higher than the one required when using a seismic margin level of 1.67 x AB building specific FIRS.

PLANT SPECIFIC COMPONENTS, BURIED PIPES

The seismic design basis calculation of buried pipes is provided in CCNPP Unit 3 FSAR Section 3.0 Appendix 3E.6. The maximum Demand/Capacity ratio reported is 0.5, obtained for an acceleration level of 0.15 g. Since the design of buried pipes is likely controlled by loads other than seismic, the 0.5 ratio suggests that buried pipes have sufficient ruggedness to meet 1.67 times the 0.115g GMRS ZPA. However, since a FOS for this item was not calculated during design basis analysis, a seismic margin for buried pipes was calculated using three representative duct bank sizes analyzed for 0.192 g ZPA. The method of analysis for seismic response accounts for soil-structure interaction (SSI) effect between the duct bank and surrounding soil (the design basis calculation had conservatively ignored this aspect). The dead and soil load responses, which are based on a beam-on-elastic-foundation approach, are taken from the design basis calculation. Moment and shear demands for these loads are combined with respective values obtained for seismic loads.

The Seismic Category I Buried Duct Banks for CCNPP Unit 3 have been found to be adequate in design for the increased ZPA of 0.192 g. The duct banks have a minimum Seismic Margin (SM) of 0.37 g, which provides an FOS greater than 3.0 (0.37/0.115). The SM is higher than the required 0.192 g value since the design of buried pipes is heavily influenced by loads other than seismic.

SCREENING OF PLANT SPECIFIC COMPONENTS AND SITE SPECIFIC EFFECTS

The analysis of the site specific soil effects and plant specific components indicates that the DC HCLPF calculations are not affected by the CCNPP Unit 3 site effects and plant specific components. The site specific effects and plant specific components are screened out for ruggedness and do not impact the plant level HCLPF of 1.67 times the GMRS (or FIRS).}

TABLE 19.1-2: BUILDING SITE SPECIFIC FIRS AND ASSOCIATED SEISMIC MARGINS REQUIRED FOR SCREENING OF SLIDING AND BEARING DEMANDS

BUILDING	FIRS[1] [g]	FIRS[2] (w/SSSI) [g]	SSE[3] (w/SSSI) [g]	REQUIRED MARGIN[4] [g]	FOS_T [5]
NI/NAB/AB	0.135	0.135	0.150	0.225	1.50
EPGB	0.117	0.130	0.181	0.217	1.20
ESWB	0.130	0.144	0.181	0.240	1.33

[1] Building specific FIRS using site specific response analysis

[2] Building specific FIRS using site specific response analysis and SSSI factor(1.105, except NI)

[3] SSE Based outcrop motion at foundation level (includes higher utilized SSSI factor)

[4] Required seismic margin calculated as 1.67 x FIRS

[5] Target FOS with 0.15 g analysis calculated as (1.67 x FIRS / SSE Motion)

TABLE 19.1-3: SUMMARY OF SEISMIC DESIGN BASIS AND SCREENING TARGET FOS

CASE	SM_T (1) [g]	FOS_T (2)	FOS_D (3)	SM_{CHECK} (4)	FOS_R (5)
NI Sliding	0.224	1.47	1.79	PASS	>> 1.79
NI Bearing (a)	0.224	1.47	1.57	PASS	>> 1.57
EPGB Sliding	0.216	1.11	1.57	PASS	>> 1.57
EPGB Bearing (b)	0.216	1.11	7.10	PASS	>> 7.10
ESWB Sliding	0.240	1.23	1.99	PASS	>> 1.99
ESWB Bearing (c)	0.240	1.23	3.86	PASS	>> 3.86
NAB Sliding (d)	0.224	NOTE (d)	NOTE (d)	PASS	NOTE (d)
NAB Bearing (d)	0.224	1.47	1.49	PASS	>> 1.49
AB Sliding (e)	0.224	1.47	1.38	< 1.47	1.89
AB Bearing (e)	0.224	1.47	3.22	PASS	>> 3.22
TI Sliding (f)	0.227	NOTE (f)	NOTE (f)	PASS	NOTE (f)

(1) Target seismic margin to screen component

(2) Target Factor of Safety to screen component

(3) Factor of Safety obtained from the analysis for seismic design basis (0.15 g)

(4) 1.67 x HCLPF Check; pass if $FOS_D > FOS_T$

(5) Factor of Safety for rugged component under design basis ground motion level. >> Symbol indicates that FOS_R for component is greater than the design basis value

(a) Allowable dynamic bearing capacity (DBC) is 31.3 ksf, site demand is 23.0 ksf

(b) Allowable DBC is 51.1 ksf (Ref. 8), site demand is 7.2 ksf

(c) Allowable DBC is 41.3 ksf (Ref. 9), site demand is 10.7 ksf

(d) NI/NAB gap is greater than twice the sliding distance plus the displacement related to building rotation (30" gap to accommodate 10" of displacement); allowable DBC is 52.9 ksf (Ref. 10); demand is 35.5 ksf

(e) AB does not slide; NI/AB gap is greater than twice the displacement related to building rotation; allowable DBC is 58.0 ksf ; demand is 18.1 ksf; additional ruggedness was incorporated into FOS_R to show that sufficient margin is available

(f) No sliding; TI GAP is at least 30 ft and sliding/bearing failure does not impact Category I structures

Enclosure 3

Table of Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 353, Question 19.01-1, Calvert Cliffs Nuclear Power Plant, Unit 3

Table of Changes to CCNPP Unit 3 COLA

Associated with the Response to RAI No. 353

Change ID #	Subsection	Type of Change	Description of Change
Part 2 – FSAR			
CC3-13-0120	19.1.5.1.1.2 19.1.5.1.1.5 19.1.5.1.2.4 Table 19.1-2 Table 19.1-3	Incorporated COLA markups associated with the response to RAI 353, Question 19.1-1 (this response).	Supplemental information is provided explaining the relationship between HCLPF and GMRS as part of the RAI 353, Question 19.1-1 response.
CC3-11-0147	19.1.5.1.2.4 19.1.5.4.7	Incorporated COLA markups associated with the response to RAI 313, Question 19-26[2].	The response to RAI 313 Question 19-26 involved modifying the COL Item and response in section 19.1.5.1.2.4.
UN#09-494	19.1 (No COLA Changes – response only)	Provided the response to RAI 160, Question 19-19[3].	The response to RAI 160 Question 19-19 provided an explanation of the relationship between HCLPF and GMRS.

[2] UniStar Nuclear Energy Letter UN#11-241, from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 313, Probabilistic Risk Assessment and Severe Accident Evaluation, dated September 14, 2011

[3] UniStar Nuclear Energy Letter UN#09-494, from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 160, Probabilistic Risk Assessment and Severe Accident Evaluation, dated December 11, 2009