PMLevyCOLPEm Resource

From: Waters, David [David.Waters@pgnmail.com]
Sent: Friday, July 20, 2012 1:01 PM
To: Habib, Donald
Cc: Rose, Dana; Kitchen, Robert; ANAND.K.SINGH@sargentlundy.com
Subject: RG 1.60 and Liquefaction Draft File Set for NRC
Attachments: 3.7 RG 1.60 FIRS 07 20 12.pdf; 2.5.2.6.3 Horizontal GMRS Final 07 17 12.pdf; 2.5.4.5.4 Backfill Nuclear Island 07 19 12.pdf; 2.5.4.8.4 Liquefaction Cyclic Stress 07 20 12.pdf; 2.5.4.8.7 Liquefaction CEUS 07 19 12.pdf; 2.5.7 New Reference 07 20 12.pdf

Don – Attached is the DRAFT FSAR additional updates to address the question re RG 1.60 spectra as discussed on the Thursday call, in preparation for the finalization of Levy COLA Revision 5 submittal. Subsection 2.5.2.6.3 is included for completeness although there has been no change from the version sent on Tuesday 7/17/2012. A revision to Subsection 2.5.7, which adds reference 2.5.4.8-203 (addressed in the revised Subsection 2.5.4.8.4) is also included for completeness.

Dave Waters
Duke Energy Nuclear Development - Licensing
PEB 1095C, 411 Fayetteville Street (PO Box 1551), Raleigh, NC 27601
919-546-7171
david.waters@pgnmail.com

Hearing Identifier: Levy_County_COL_Public
Email Number: 1161

Mail Envelope Properties (7FD614EFA33B03448166AF44364F42AE0E68B8A3A0)

Subject: RG 1.60 and Liquefaction Draft File Set for NRC
Sent Date: 7/20/2012 1:00:38 PM
Received Date: 7/20/2012 1:01:24 PM
From: Waters, David

Created By: David.Waters@pgnmail.com

Recipients:
"Rose, Dana" <Dana.Rose@pgnmail.com>
Tracking Status: None
"Kitchen, Robert" <robert.kitchen@pgnmail.com>
Tracking Status: None
"ANAND.K.SINGH@sargentlundy.com" <ANAND.K.SINGH@sargentlundy.com>
Tracking Status: None
"Habib, Donald" <Donald.Habib@nrc.gov>
Tracking Status: None

Post Office: WN000075.oak.zone1.progress-energy.com

Files	**Size**	**Date & Time**
MESSAGE	659	7/20/2012 1:01:24 PM
3.7 RG 1.60 FIRS 07 20 12.pdf	324188	
2.5.2.6.3 Horizontal GMRS Final 07 17 12.pdf	239136	
2.5.4.5.4 Backfill Nuclear Island 07 19 12.pdf	240864	
2.5.4.8.4 Liquefaction Cyclic Stress 07 20 12.pdf	324320	
2.5.4.8.7 Liquefaction CEUS 07 19 12.pdf	235229	
2.5.7 New Reference 07 20 12.pdf	255212	

Options
Priority: Standard
Return Notification: No
Reply Requested: No
Sensitivity: Normal
Expiration Date:
Recipients Received:

high frequency range (≥ 25 Hz.) except for the horizontal spectra at node 2078. At this node, the AP1000 HRHF FRS provides sufficient additional margin.

The second SSI analysis was performed using the 2D “Coarse” and “Fine” models for the BE soil profile. The SASSI Direct method was used. The 5 percent damped FRS at the six key nodes were generated. Frequency dependent Bump Factors (≥ 1.0) were calculated from the FRS as the ratio of the 2D Fine model and the 2D Coarse model FRS at the six key nodes.

The third SSI analysis was performed using the 3D 5-layer embedded model for the BE soil profile. The SASSI Direct method was used. The 5 percent damped FRS at the six key nodes were generated. The frequency dependent Bump Factors calculated from the 2D model were applied to the 3D 5-layer model FRS along the frequency spectrum to amplify the 3D 5-layer model FRS. These factored FRS are compared to the AP1000 generic and HRHF (as necessary) FRS envelops at the six key locations in Figures 3.7-220, 3.7-221, 3.7-222, 3.7-223, 3.7-224, and 3.7-225. The HRHF FRS envelope is presented for 3D nodes 2078, 2199, and 2675 to demonstrate that additional margin exists at the three nodes in the high frequency region (20-50 Hz.). As shown in the figures, the LNP site-specific factored FRS are enveloped by the AP1000 generic and HRHF FRS envelopes at each of the six nodes with sufficient margin.

3.7.2.4.1.6 Bearing Pressure and Base Shear

Based on the SSI analysis, the maximum bearing pressure on the RCC bridging mat beneath the NI basemat for the BE, UB, LB and LLB soil profiles is 20.29 ksf. The maximum bearing pressure corresponds to the BE soil profile. The LNP site specific maximum bearing pressure is enveloped by the AP1000 soft rock site maximum bearing pressure of 24 ksf for soft rock sites.

Based on the SSI analysis, the maximum base shear on the RCC bridging mat for the BE, UB, LB and LLB soil cases is 77,600 kips. The maximum base shear corresponds to the BE soil profile. The maximum 77,600 kips base shear yields a base shear to vertical load ratio of 0.12 for the NI. This ratio is enveloped by the AP1000 maximum ratio of 0.55.

3.7.2.4.1.7 Sensitivity Evaluations for Regulatory Guide 1.60 Spectra FIRS

The Regulatory Guide 1.60 Foundation Input Response Spectra (FIRS) is anchored at peak ground accelerations for the scaled site-specific FIRS in Table 2.5.2-236 (0.1g horizontal and 0.0695g vertical). The scaled site-specific FIRS was developed using the updated EPRI SOG methodology and scaled to meet 10 CRF Part 50 Appendix S requirements. Tables 3.7-203 and 3.7-204 present the 5% damped site specific FIRS, the 5% damped Regulatory Guide 1.60 FIRS, and the ratio of the Regulatory Guide FIRS and the site specific FIRS at various frequencies for horizontal and vertical spectra respectively.

Sensitivity evaluations were performed to assess whether the FRS at the six key locations using the Regulatory Guide 1.60 FIRS instead of the scaled site-specific FIRS remains bounded by the Certified Seismic Design Response Spectra (CSDRS) FRS. The sensitivity evaluations were performed using conservative simplified methodology by scaling the entire site specific FRS by the ratio of the Regulatory Guide 1.60 FIRS and the scaled site specific FIRS at the predominant response frequency at the node/direction. The predominant response frequency was determined from the peaks in the site specific FRS at each of the six nodes in the X, Y, and Z directions. The site specific FRS at the six nodes in the X, Y, and Z directions are shown in Figures 3.7-214, 3.7-215, 3.7-216, 3.7-217, 3.7-218, and 3.7-219. For this evaluation the lowest predominant response frequency is used because it will yield a larger scaling factor and is thus conservative. Table 3.7-205 presents the predominant response frequencies at the six key nodes in the X, Y, and Z directions, the ratio of the Regulatory Guide 1.60 FIRS and the scaled site specific FIRS at the predominant response frequency (scaling factor), and the minimum margin for site specific FRS with respect to the CSDRS FRS when the whole site specific FRS is scaled by the scaling factor for the predominant response frequency for the node and direction. Because the scaling factors to develop the Regulatory Guide 1.60 FRS are always smaller than the available margin with respect to the CSDRS FRS, the Regulatory Guide 1.60 FRS will be bounded by the CSDRS FRS. In addition, because the Regulatory Guide 1.60 spectra has only a small frequency content above 20 Hz. and no frequency content above 33 Hz., the Regulatory Guide 1.60 FRS peaks in the high frequency range (>20 Hz.) will be lower that that obtained by the simple scaling used, thus providing additional margin with respect to the CSDRS FRS.

As stated in Subsections 2.5.4.5.4 and 2.5.4.10.1.1, the conceptual design of the RCC bridging mat is based on a bearing pressure of 8.9 kips per square foot [ksf] for static loading and 24.0 ksf for dynamic loading. The static bearing pressure is based on DCD Tier 1 Table 5.0.1. The dynamic bearing pressure is the maximum subgrade pressure at the AP1000 basemat that results from the generic AP1000 analysis for soft rock sites. For the subsurface rock bearing capacity calculations, the RCC self weight was included as an additional bearing pressure load of 5.16 ksf. The buoyancy effects due to the hydrostatic pressure acting at the bottom of the RCC were considered in this analysis. A base shear load of 136,000 kips based on the AP1000 generic analysis was applied at the top of the RCC bridging mat. Because the AP1000 generic analyses are based on the CSDRS (0.3g Regulatory Guide 1.60 spectra enhanced in the high frequency region), the RCC design is conservative for the Regulatory Guide 1.60 FIRS.

3.7.2.8.1 Annex Building

Add the following text to the end of DCD Subsection 3.7.2.8.1.

LNP SUP 3.7-5

In DCD Subsection 3.7.2.8.1, the maximum displacement of the roof of the Annex Building is reported as 1.6 inches for response spectra input at the base of the building that envelops the SSI spectra for the six soil profiles and also the CSDRS. The Annex Building foundation (top of mat) is at design grade. Figure 2.5.2-297 shows a comparison of the LNP scaled performance based surface response spectra (PBSRS) at the plant design grade and the CSDRS. The CSDRS envelops the LNP PBSRS by a wide margin. Thus, the LNP Annex Building roof displacement relative to its foundation is expected to be less than the 1.6 inches in the DCD for the CSDRS. The computed probable maximum relative displacement during SSE between the NI and the Annex Building foundation mat is less than 2.5 cm (1 in.) for both the scaled Performance Based Surface Response Spectra (PBSRS) or the Regulatory Guide 1.60 spectra anchored at peak ground acceleration of 0.1g applied at the foundation elevation of the Annex Building as shown in Table 3.7-206. The probable maximum relative displacement calculation included the drilled shaft supported foundation mat displacements including the drilled shaft to drilled shaft interaction effects, additional displacement due to soil column displacement, and the NI displacement at design grade. The square root of the sum of squares (SRSS) method was used to compute the probable maximum relative displacement. Thus, the LNP Annex Building roof displacement during SSE is expected to be less than 2.6 inches. As stated in DCD Subsection 3.7.2.8.1, the minimum clearance between the structural elements of the Annex Building above grade and the nuclear island (NI) is 4 inches. Figure 3.7-226 shows the conceptual design detail for the interface between the Nuclear Island (NI) and the drilled shaft supported foundation mat of the Annex Building. This design detail provides a 5.0 cm (2 in.) gap between the Annex Building foundation and the NI consistent with DCD Subsection 3.8.5.1. The top of the diaphragm wall and controlled low strength material fill between the diaphragm wall and the NI wall is at least 1.5 m (5 ft.) below the bottom of the Annex Building foundation mat as stated in Subsection 2.5.4.5.1. Engineered fill is used from the top of the controlled low strength material fill to the bottom of the Annex Building foundation as stated in Subsection 2.5.4.5.4. This interface is designed to avoid hard contact between the NI and the Annex Building foundation mat resulting from the relative displacement between the NI and the Annex Building foundation mat during the seismic event. Thus, no seismic interaction between the Annex Building and the NI is expected.

3.7.2.8.2 Radwaste Building

Add the following text to the end of DCD Subsection 3.7.2.8.2.

LNP SUP 3.7-5 The computed probable maximum relative displacement between the NI and the Radwaste Building foundation mat is less than 2.5 cm (1 in.) for both the scaled PBSRS or the Regulatory Guide 1.60 spectra anchored at peak ground acceleration of 0.1g applied at the foundation elevation of the Radwaste Building as shown in Table 3.7-206. The probable maximum relative displacement calculation included the drilled shaft supported foundation mat displacements including the drilled shaft to drilled shaft interaction effects, additional

Deleted: from a Performance Based Surface Response Spectra (

Deleted:)

Deleted: is less than 2.5 cm (1 in.)

displacement due to soil column displacement, and the NI displacement at design grade. The SRSS method was used to compute the probable maximum relative displacement. Figure 3.7-226 shows the conceptual design detail for the interface between the Nuclear Island (NI) and the drilled shaft supported foundation mat of the Radwaste Building. This design detail provides a 5.0 cm. (2 in.) gap between the Radwaste Building foundation and the NI consistent with DCD Subsection 3.8.5.1. The top of the diaphragm wall and controlled low strength material fill between the diaphragm wall and the NI wall is at least 1.5 m (5 ft.) below the bottom of the Radwaste Building foundation mat as stated in Subsection 2.5.4.5.1. Engineered fill is used from the top of the controlled low strength material fill to the bottom of the Radwaste Building foundation as stated in Subsection 2.5.4.5.4. This interface is designed to avoid hard contact between the NI and the Radwaste Building foundation mat resulting from the relative displacements during the seismic event. Thus, no seismic interaction between the Radwaste Building foundation mat and the NI is expected.

3.7.2.8.3 Turbine Building

Add the following text to the end of DCD Subsection 3.7.2.8.3.

LNP SUP 3.7-5 The computed probable maximum relative displacement between the NI and the Turbine Building foundation mat is less than 2.5 cm (1 in.) for both the PBSRS or the Regulatory Guide 1.60 spectra anchored at peak ground acceleration of 0.1g applied at the foundation elevation of the Turbine Building as shown in Table 3.7-206. The probable maximum relative displacement calculation included the drilled shaft supported foundation mat displacements including the drilled shaft to drilled shaft interaction effects, additional displacement due to soil column displacement, and the NI displacement at design grade. The SRSS method was used to compute the probable maximum relative displacement. Figure 3.7-226 shows the conceptual design detail for the interface between the Nuclear Island (NI) and the drilled shaft supported foundation mat of the Turbine Building. This design detail provides the 5.0 cm. (2 in.) gap between the Turbine Building foundation and the NI consistent with DCD Subsection 3.8.5.1. The top of the diaphragm wall and controlled low strength material fill between the diaphragm wall and the NI wall is at least 1.5 m (5 ft.) below the bottom of the Turbine Building foundation mat as stated in Subsection 2.5.4.5.1. Engineered fill is used from the top of the controlled low strength material fill to the bottom of the Turbine Building foundation mat as stated in Subsection 2.5.4.5.4. This interface is designed to avoid hard contact between the NI and the Turbine Building foundation mat resulting from the relative displacements during the seismic event. Thus, no seismic interaction between the Turbine Building foundation mat and the NI is expected.

Deleted: from a Performance Based Surface Response Spectra (

Deleted:)

Deleted: is less than 2.5 cm (1 in.)

3.7.2.8.4 Median Centered Adjacent Building Relative Displacements for 10^{-5} UHRS

TABLE 3.7-203

Ratio of Horizontal RG 1.60 FIRS and Site Specific (SS) FIRS

Frequency (Hz)	Site Specific FIRS (g)	RG 1.60 FIRS (g)	RG 1.60 / SS FIRS Ratio
1.00	0.108	0.147	1.36
1.50	0.156	0.206	1.32
2.00	0.176	0.261	1.48
2.50	0.196	0.313	1.60
3.00	0.214	0.305	1.43
3.50	0.230	0.298	1.30
4.00	0.245	0.293	1.20
5.00	0.273	0.284	1.04
6.00	0.276	0.276	1.00
9.00	0.265	0.261	0.98
10.00	0.263	0.241	0.92
12.00	0.260	0.211	0.81
15.00	0.253	0.179	0.71
20.00	0.231	0.145	0.63
30.00	0.183	0.107	0.59
33.00	0.175	0.100	0.57
100.00	0.100	0.100	1.00

TABLE 3.7-204

Ratio of Vertical RG 1.60 FIRS and Site Specific (SS) FIRS

Formatted Table

Frequency (Hz)	Site Specific FIRS (g)	RG 1.60 FIRS (g)	RG 1.60/ SS FIRS Ratio
1.00	0.068	0.071	1.05
2.00	0.104	0.129	1.24
3.00	0.122	0.182	1.49
3.50	0.130	0.207	1.59
4.00	0.139	0.203	1.46
5.00	0.154	0.197	1.28
6.00	0.157	0.192	1.22
7.00	0.157	0.188	1.20
9.00	0.157	0.181	1.15
10.00	0.159	0.168	1.06
15.00	0.170	0.124	0.73
18.00	0.174	0.109	0.63
20.00	0.175	0.101	0.58
33.00	0.144	0.070	0.49
100.00	0.070	0.070	1.00

LNP SUP 3.7-6

LNP SUP 3.7-3

Table 3.7-205
Predominant Frequencies, Scale Factors for Regulatory Guide 1.60 FIRS, and CSDRS FRS Margin

Node / Direction	Predominant Frequency (Hz.)	Ratio RG 1.60 and Scaled FIRS	Minimum CSDRS FRS Margin
1761-X	3.0	1.43	>1.43
1761-Y	5.5	1.02	>1.02
1761-Z	5.0	1.28	>1.28
2078-X	20.0	0.63	>1.00
2078-Y	12.0	0.81	>1.00
2078-Z	20.0	0.58	>1.00
2199-X	20.0	0.63	>1.00
2199-Y	5.5	1.02	>1.02
2199-Z	20.0	0.58	>1.00
2675-X	30.0	0.59	>1.00
2675-Y	3.0	1.43	>1.43
2675-Z	6.0	1.22	>1.22
2788-X	5.0	1.04	>1.04
2788-Y	5.5	1.02	>1.02
2788-Z	18.0	0.63	>1.00
3329_X	3.5	1.30	>1.30
3329-Y	3.0	1.43	>1.43
3329-Z	7.0	1.20	>1.20

Formatted Table

Table 3.7-206
Probable Maximum Relative Displacements between the Nuclear Island (NI) and Adjacent Buildings

Adjacent Building	Probable Maximum Relative Displacement (in.)	
	Site Specific FIRS	RG 1.60 FIRS
Between NI and Annex Building	0.70	0.59
Between NI and Radwaste Building	0.77	0.64
Between NI and Turbine Building	0.40	0.35

Formatted Table

Deleted: ¶
Section Break (Next Page)

2.5.2.6.3 Horizontal GMRS

Regulatory Guide 1.208 defines the GMRS as a risk-consistent design response spectrum computed from the site-specific UHRS at a mean annual frequency of exceedance of 10^{-4} by the relationship:

$$GMRS = DF \times UHRS\ (10^{-4}) \qquad \text{Equation 2.5.2-215}$$

Parameter *DF* is the design factor specified by the expression:

$$DF = \text{Maximum}\ (1.0,\ 0.6(A_R)^{0.8}) \qquad \text{Equation 2.5.2-216}$$

In which A_R is the ratio of the UHRS ground motions for annual exceedance frequencies of 10^{-4} and 10^{-5}, specifically:

$$A_R = \frac{UHRS(10^{-5})}{UHRS(10^{-4})} \qquad \text{Equation 2.5.2-217}$$

Regulatory Guide 1.208 also specifies that when the value of A_R exceeds 4.2, the amplitude of the GMRS is to be no less than 0.45 × $SA(0.1H_D)$ that is, 45 percent of the 10^{-5} UHRS. As the 10^{-4} UHRS with CAV is 0, this second criteria is used to define the horizontal GMRS. Figure 2.5.2-294 shows the horizontal GMRS calculated as 0.45 × $SA(0.1H_D)$.

For site-specific evaluations and design (liquefaction evaluations, seismic interaction of the Auxillary Building, Turbine Building, and Radwaste Building with the Nuclear Island, and Soil Structure Interaction analysis of the Nuclear Island), scaled PBSRS and scaled FIRS described in Subsection 2.5.2.6.6 are used. The scale factor of 1.212 was used so that the FIRS has a zero period acceleration of 0.1 g as required by 10 CFR Part 50 Appendix S. To be consistent with the site-specific evaluations and design, the horizontal GMRS was also scaled by the 1.212 factor. The scaled horizontal GMRS is listed in Table 2.5.2-226 along with the 10^{-5} UHRS and is shown on Figure 2.5.2-294. The scaled horizontal GMRS represents the licensing basis for the LNP site.

Deleted: and the design of the RCC bridging mat),

2.5.4.5.4 Properties of Backfill Beneath and Adjacent to Nuclear Island

Based on a design grade elevation of 15.5 m (51 ft.) NAVD88, the elevation of each nuclear island basemat will be 3.4 m (11 ft.) NAVD88. A 15.2 cm (6 in.) mudmat will be located beneath each nuclear island basemat at elevation 3.4 m (11 ft.) NAVD88. Structural fill between the excavation bottom (elevation -7.3 m [-24 ft.] NAVD88) and the nuclear island mudmat (elevation 3.4 m [11 ft.] NAVD88) will consist of an RCC bridging mat, as shown on Figures 2.5.4.5-201B and 2.5.4.5 202B. A waterproofing membrane will be located between the RCC and the mudmat, meeting AP1000 DCD requirements of 0.55 static coefficient of friction between horizontal membrane and concrete. For buildings adjacent to the nuclear islands, the design grade will be raised to elevation 15.5 m (51 ft.) NAVD88 using engineered fill.

The following is the Design Description of the RCC. This RCC fill will serve two purposes: 1) replace the weakly cemented, undifferentiated Tertiary sediments that are present above elevation -7.3 m (-24 ft.) NAVD88, thereby, creating a uniform subsurface with increased bearing capacity; and 2) bridge conservatively postulated karst features.

The RCC bridging mat has been designed to bridge a 3-m (10-ft.) air-filled cavity located immediately beneath the RCC (elevation -7.3 m [-24 ft.] NAVD88) at any plan location for loading conditions identified in Subsection 2.5.4.10.1.1. In addition, a base shear load of 136,000 kips based on the AP1000 generic analysis was applied at the top of the RCC bridging mat. These loads are based on generic AP1000 analyses. The 1-year specified compressive strength (f'_c) of the RCC is 2500 psi. The design of the RCC bridging mat has considered a nominal tensile strength of 250 psi.

Deleted: DCD Tier 1 Table 5.0-1 and Tier 2 Table 2-1

A theoretical rock profile for the North and South Plant Units was developed using LNP site-specific rock properties and layering information. A SAP2000 Finite Element Model (FEM – linearly elastic) of the RCC, nuclear island basemat, and the subsurface rock was created using the design geometry, the rock profile beneath the RCC Bridging Mat, and the total loads applied by the nuclear island.

Also included in the FEM was the presence of theoretical cavities of different sizes and configurations. Three different cases, with cavities located at different depths, were considered:

- Case A: Cavities were located immediately below the grouted limestone, at elevation -99 ft. NAVD88 (75 ft. under the RCC).
- Case B: Cavities were located immediately below the RCC, at elevation -24 ft. NAVD88.
- Case C: Cavities were located at the top of rock layer NAV-3, which is the layer with lower Elastic Modulus for the North Reactor profile, below elevation -149 ft. NAVD88 (125 ft. under the RCC). This case was analyzed only in the North Reactor, where the lower Elastic Modulus layer is somewhat thicker than in the South Reactor profile.

Examples of the locations of these cavities are shown on Figure 2.5.4.5-204.

2.5.4.8.4 Earthquake Induced Cyclic Stress

Earthquake-induced cyclic stresses within soils considered for liquefaction analysis were computed from the site response analyses used to develop the site amplification functions for the PBSRS profiles described in Subsection 2.5.2.5. The site response analyses were performed using 60 randomized soil profiles representing each PBSRS shear wave velocity profile and 30 acceleration time histories representing each deaggregation earthquake (DE) listed in Table 2.5.2-225. In each individual site response analysis effective cyclic shear strains and iterated shear modulus were computed for each layer of the profile. The effective cyclic shear stress for each layer was then taken as the product of the effective cyclic shear strain and the iterated shear modulus. The results of the 180 analyses (60 randomized profiles times three deaggregation earthquakes) were then used to compute a weighted mean effective cyclic shear stress for each layer within each of the three PBSRS soil profiles and for the 10^{-4} and 10^{-5} exceedance level input motions. The weights used were the relative weights assigned to the DEs that are listed in Table 2.5.2-225.

The results of the site response analyses were used to produce peak ground acceleration (PGA) seismic hazard results at the finished graded elevation computed without CAV for the 10^{-4} and 10^{-5} exceedance levels. These values were used to compute a performance based PGA at the finished grade elevation using Equations 2.5.2-215 through 2.5.2-217. The resulting acceleration value is 0.118g. The corresponding PGA at the base of the excavation (-24 ft. NAVD88) is 0.071g. These values along with the site class and the value of F_a based on the International Building Code (2006) are shown in Table 2.5.4.8-201.

The development of the cyclic shear stress complies with the guidance in Regulatory Position 3.3.2 of Regulatory Guide 1.198 because an ensemble of time histories was used that represent the earthquakes contributing to the hazard at the LNP site. The development of the ensemble of time histories is described in Subsection 2.5.2.5.2. The time histories used to represent the DE were taken from NUREG/CR-6728 (Reference 2.5.2-263). The weighted mean magnitude for the earthquake time histories representing the high frequency (HF) 10^{-4} and 10^{-5} DEs are 6.8 and 6.1, respectively. Thus, these time histories also satisfy the acceptance criteria in SRP Section 2.5.2 in that weighted mean magnitudes for the ensembles of time histories exceed magnitude 6. The associated number of equivalent cycles of loading was estimated using the relationship between earthquake magnitude and number of loading cycles provided in Reference 2.5.4.8-203. The m_b magnitudes listed in Table 2.5.2-225 for the HF DEs were converted to moment magnitudes using the relationships given in Subsection 2.5.2.4.2.3 and the resulting average moment magnitude was used to estimate the number of cycles for each DE using Figure 12 in Reference 2.5.4.8-203. The resulting weighted mean values are 9.4 cycles and 6.5 cycles for the HF 10^{-4} and 10^{-5} hazard levels, respectively.

Deleted: the mean of the cyclic stress for each soil layer from the randomized set of soil profiles used to develop the PBSRS using the SHAKE program. The rock peak ground acceleration,

Deleted: , and the horizontal peak ground surface acceleration for the North and the South reactor

Deleted: o

Deleted: estimated based on the seismic ground motions, specifically horizontal ground accelerations versus time as identified on Table 2.5.4.8-201. These ground motions are based on the SHAKE analyses used to develop the GMRS, including the soil profile randomization procedure. The ground motions were scaled up to 0.10 g.

Table 2.5.4.8-201
Summary of Peak Ground Acceleration Used for Liquefaction Analysis

Structure	Rock Peak Ground Acceleration (g)	Site Class	F_a	a_{max} (g)
North Reactor	0.071	C	1.2	0.118
South Reactor	0.071	C	1.2	0.118

Notes:

Site Class and F_a were estimated based on International Building Code (IBC) (2006).

a_{max} = Horizontal peak acceleration at ground surface for the PBSRS profile with no CAV or scaling
g = gravity acceleration

Deleted: A minimum value of 0.1 g was used, per 10 CFR 50.

2.5.4.8.7 Liquefaction Potential Evaluations for CEUS SSC

The soils under the Nuclear Island will be excavated and backfilled with RCC; therefore, no liquefaction potential exists under the Nuclear Island foundation. For design basis evaluations of liquefaction potential of soils under the adjacent Annex, Turbine and Radwaste Buildings, earthquake-induced cyclic stresses in the soil column were based on ground motions computed for the PBSRS profile using the updated EPRI-SOG model. The associated PGA at the finished grade elevation is 0.118g (Table 2.5.4.8-201) and is based on the surface hazard curves computed without CAV. The PGA at the finished grade elevation computed without CAV using the CEUS SSC model is 0.091g. As the computed equivalent cyclic shear stresses are proportional to the PGA at the finished grade, the equivalent cyclic shear stresses based on the CEUS SSC model would be lower than those computed based on the updated EPRI-SOG model. Therefore, the liquefaction evaluations based on the updated EPRI-SOG LNP ground motions bound those from the CEUS SSC ground motions.

Deleted: To evaluate the

Deleted: consistent with the finished grade scaled PBSRS. As shown in Figure 2.5.2-357, the CEUS SSC PBSRS is enveloped by the updated EPRI-SOG scaled PBSRS.

For site specific seismic margins evaluation presented in Subsection 19.55.6.3 liquefaction potential of soils under the adjacent Annex, Turbine and Radwaste Buildings, earthquake-induced cyclic stresses in the soil column, based on ground motions consistent with the updated EPRI-SOG finished grade 10^{-5} UHRS, were used. As shown in Figures 3.7-228 and 3.7-229, 1.67*GMRS and 1.67*PBSRS developed using the CEUS SSC methodology and modified CAV filter are enveloped by the updated EPRI-SOG finished grade 10^{-5} UHRS. Furthermore, the PGA for the 10^{-5} PBSRS profile surface motions computed without CAV using the CEUS SSC model are lower than those computed using the updated EPRI-SOG model. Thus, the High Confidence Low Probability of Failure (HCLPF) capacity for no liquefaction potential of soil under the Annex, Turbine, and Radwaste Buildings exceeds the 1.67*GMRS goal for the plant level HCLPF for the CEUS SSC ground motions.

Deleted: To evaluate the High Capacity, Low Probability of Failure (HCLPF)

New Reference added to Subsection 2.5.7

2.5.4.8-203 Seed H.B., Idriss I.M., Makdisi F., & Banerjee N., Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses, Report No EERC 75-29, October, 1975.