

PMLevyCOLPEm Resource

From: Kitchen, Robert [robert.kitchen@pgnmail.com]
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To: Habib, Donald
Cc: Rose, Dana; Waters, David
Subject: RG 1.60 and Liquefaction Draft File Set for NRC
Attachments: 3.7 RG 1.60 FIRS Final 07 17 12.pdf; 2.5.2.6.3 Horizontal GMRS Final 07 17 12.pdf; 2.5.4.5.4 Backfill Nuclear Island Final 07 17 12.pdf; 2.5.4.8.4 Liquefaction Stress Final 07 17 12.pdf; 2.5.4.8.7 Liquefaction CEUS Final 07 17 12.pdf

Importance: High

Don – Attached is the DRAFT FSAR updates to address the question re RG 1.60 spectra as discussed on Monday call in preparation for the public call on Thursday.

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3.7 RG 1.60 FIRS Final 07 17 12.pdf		288138
2.5.2.6.3 Horizontal GMRS Final 07 17 12.pdf		239136
2.5.4.5.4 Backfill Nuclear Island Final 07 17 12.pdf		228466
2.5.4.8.4 Liquefaction Stress Final 07 17 12.pdf		234086
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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 envelopes 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 FRS

Sensitivity evaluations were performed to assess whether the Floor Response Spectra (FRS) at the six key locations using the Regulatory Guide 1.60 Foundation Input Response Spectra (FIRS) instead of the scaled site-specific FIRS remains bounded by the CSDRS FRS. The Regulatory Guide 1.60 FIRS is anchored at peak ground accelerations in Table 2.5.2-236 (0.1g horizontal and 0.0695g vertical) for the scaled site-specific FIRS. The scaled site-specific FIRS was developed using the updated EPRI SOG methodology and scaled to meet 10 CFR Part 50 Appendix S requirements. 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

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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 3.7-203 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 than 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. During detailed design of the RCC bridging mat site specific loads may be used. However, if site specific loads are used they shall consider RCC bridging mat loadings considering both the scaled updated EPRI SOG FIRS (Table 2.5.2-336) and the Regulatory Guide 1.60 FIRS anchored to the scaled updated EPRI SOG FIRS peak ground accelerations (0.1g horizontal and 0.0695g vertical).

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

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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](#). 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](#). 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

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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. 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.

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3.7.2.8.4 Median Centered Adjacent Building Relative Displacements for 10⁻⁵ UHRS

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Table 3.7-203
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	1.48	>1.48
1761-Y	5.5	1.02	>1.02
1761-Z	5.0	1.27	>1.27
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

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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:

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

Parameter DF is the design factor specified by the expression:

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

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

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

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 $0.45 \times SA(0.1H_D)$ 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 $0.45 \times SA(0.1H_D)$.

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.

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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 [conceptually](#) 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. During detailed design of the RCC bridging mat site specific loads may be used. However, if site specific loads are used they shall consider RCC bridging mat loadings considering both the scaled updated EPRI SOG FIRS and the Regulatory Guide 1.60 FIRS discussed in Subsection 3.7.2.4.1.7.](#) 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.

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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.

2.5.4.8.4 Earthquake Induced Cyclic Stress

Earthquake-induced cyclic stresses within soils considered for liquefaction analysis were 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, the site class and F_a based on the International Building Code (2006), and the horizontal peak ground surface acceleration for the North and the South reactor are shown on Table 2.5.4.8-201.

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LNP COL 2.5-9

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	0.07	C	1.2	0.118
South Reactor	0.07	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 with no CAV or scaling.
g = gravity acceleration

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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. To evaluate the 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.

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To evaluate the High Capacity, Low Probability of Failure (HCLPF) 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. Thus, 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. 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.