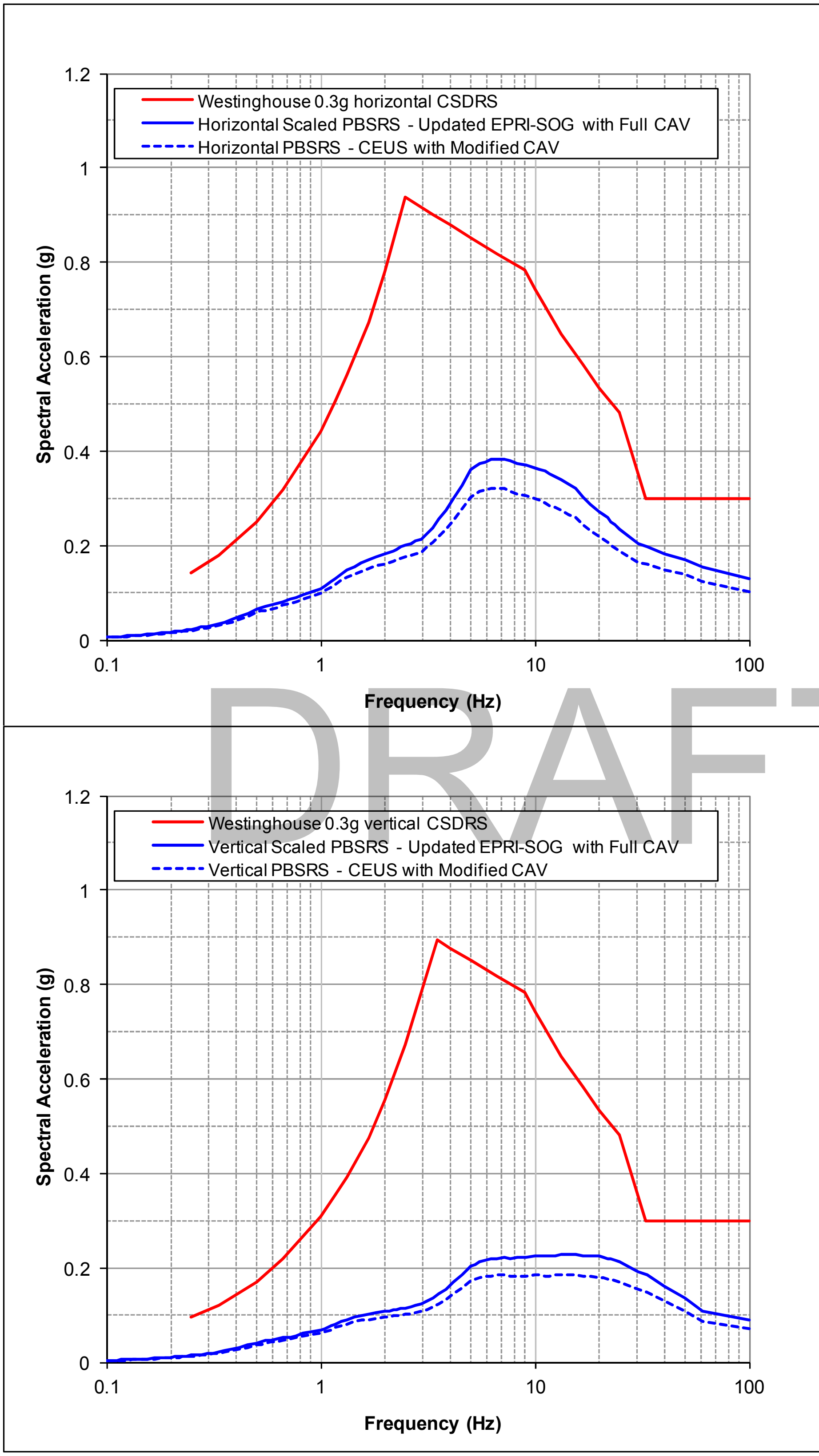

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Figure 2.5.2-357 Comparison of PBSRS based on updated EPRI-SOG and CEUS SSC models

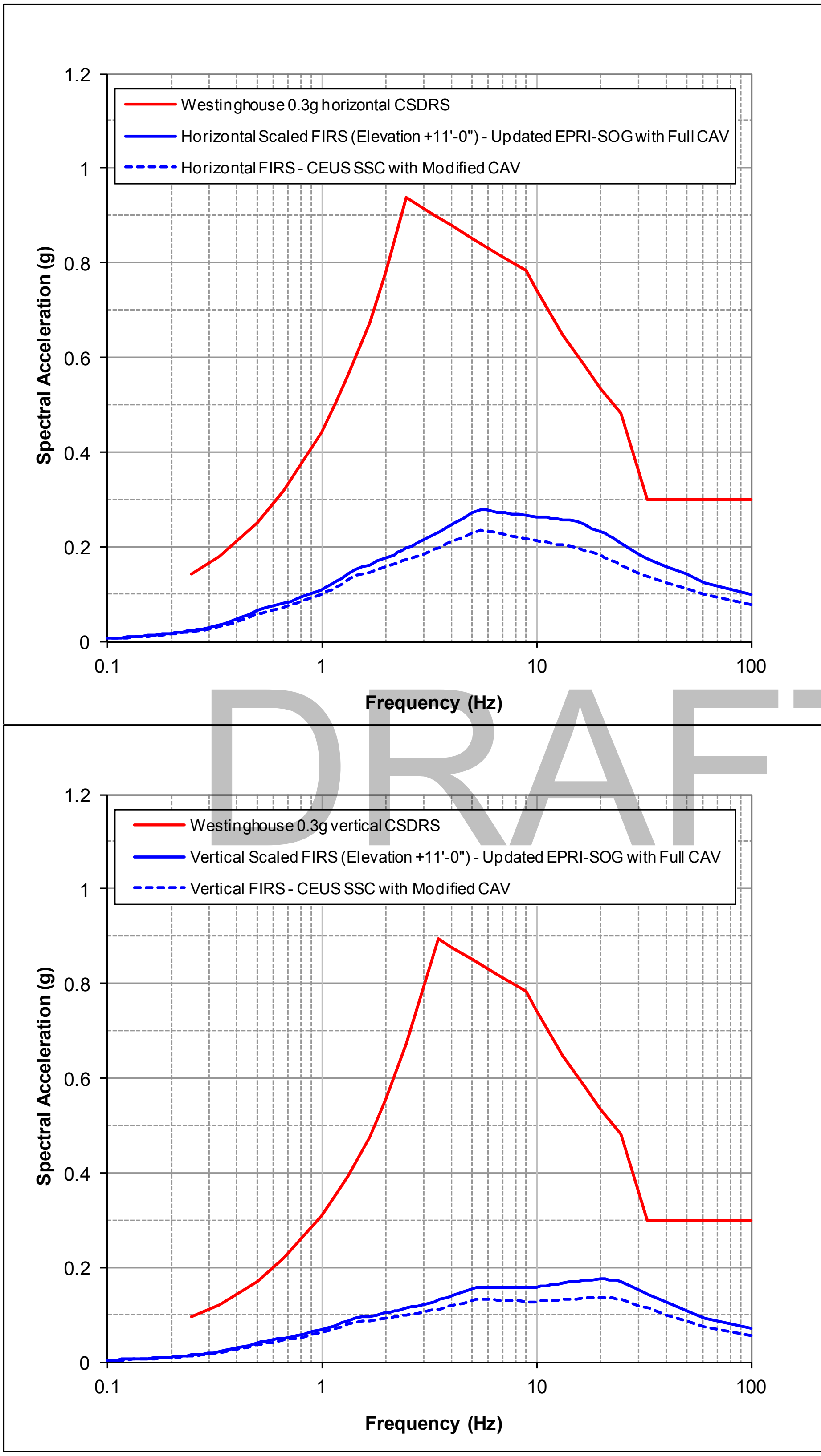

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Figure 2.5.2-358: Comparison of Reactor Building FIRS based on updated EPRI-SOG and CEUS SSC models

For the area under the Annex, Turbine, and Radwaste building footprint, in-situ soil will be replaced or improved to a depth of approximately 2.1 m (7 ft.) below existing grade (elevation 12.8 m [42 ft.] NAVD88). The plant design grade will be established at elevation 15.5 m (51 ft.) NAVD88 by placing engineered fill above the improved / replaced in-situ material. In addition, the earthwork design incorporates vertical and horizontal drains to prevent buildup of excess pore pressures that cause liquefaction as shown in Figures 2.5.4.8-205 and 2.5.4.8-206 for LNP 1 and 2 respectively.

2.5.4.8.6 Median Centered Liquefaction Evaluations for 10^{-5} UHRS

As a sensitivity analysis, the median centered liquefaction potential (factor of safety <1.0) for 10^{-5} UHRS was evaluated. The methodology and design parameters used for 10^{-5} UHRS liquefaction analysis were the same as that used for design basis liquefaction analysis described in FSAR Subsection 2.5.4.8 except liquefaction was postulated when the computed factor of safety was <1.0 and the soil cyclic shear stress were computed for the 10^{-5} UHRS ground motions and the median shear wave velocity soil profile derived from the randomized soil profiles used to compute the 10^{-5} UHRS. In addition, the equivalent number of stress cycles was computed for the weighted average moment magnitude of 5.74 for the site. Tables 2.5.4.8-203A and 2.5.4.8-203B present liquefaction analysis results for 10^{-5} UHRS for LNP 1 and 2 respectively. The results include the computed factors of safety against liquefaction and the depth below the Annex, Radwaste, or Turbine Building foundation mat where liquefaction is postulated. Figures 2.5.4.8-207 and 2.5.4.8-208 show, in plan and elevation respectively, the location of the liquefaction zones identified in Table 2.5.4.8-203A for LNP 1. Figure 2.5.4.8-209 and Figure 2.5.4.8-210 show, in plan and elevation view respectively, the liquefaction zones identified in Table 2.5.4.8-203B for LNP 2. In these figures, the liquefaction zones with a factor of safety of less than or equal to 1.0 are shown by circles with yellow infill. For Unit 1, liquefiable zones were postulated in boreholes O-2, A-15, A-18/O-4, A-13, and B-28. Boreholes O-2, A-15 and A-18/O-4 are in the nuclear island excavation zone. Borehole A-13 (factor of safety = 1.0) is under the Radwaste Building, and B-28 is under the Annex Building. For Unit 2, liquefiable zones were postulated for boreholes B-01, B-07, B-07A, B-31, and B-33. Borehole B-01 is well away from the AP1000 footprint. Boreholes B-07, B-07A, B-31, and B-33 are under the Turbine Building. Based on these figures, it can be concluded that liquefiable zones under the LNP 1 and 2 footprints are confined to the northwest corner of the LNP 2 Turbine Building and in isolated random pockets under the remaining LNP 1 and 2 footprints. These conclusions for median centered liquefaction potential for 10^{-5} UHRS are the same as the conclusions for the design basis liquefaction analysis described in FSAR Subsection 2.5.4.8.

2.5.4.8.7 Liquefaction Potential Evaluations for CEUS SSC

The soils under the nuclear island (NI) will be excavated and backfilled with Roller Compacted Concrete (RCC). Thus, no liquefaction potential exists under the NI foundation. To evaluate the liquefaction potential of soils under the

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adjacent AB, TB, and RB, earthquake induced cyclic stresses in the soil column were based on ground motions consistent with the finished grade scaled PBSRS. As shown in Figure 2.5.2-357, the CEUS SSC PBSRS is enveloped by the update EPRI SOG scaled PBSRS. Thus, 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 HCLPF liquefaction potential of soils under the adjacent AB, TB, and RB, 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 AB, TB, and RB exceeds the 1.67*GMRS goal for the plant level HCLPF for the CEUS SSC ground motions.

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3.7 SEISMIC DESIGN

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add Subsection 3.7.1.1.1 as follows:

3.7.1.1.1 Design Ground Motion Response Spectra

LNP SUP 3.7-3 Figure 2.5.2-296 shows the comparison of the scaled horizontal and scaled vertical site-specific ground motion response spectra (GMRS) to the AP1000 certified design seismic design response spectra (CSDRS). The GMRS was developed as the Truncated Soil Column Surface Response (TSCSR) on the uppermost in-situ competent material (elevation 11 m (36 ft.) NAVD88) as described in Subsection 2.5.2.6.

Plant design grade will be established at elevation 15.5 m (51 ft.) NAVD88 by placing engineered fill above in-situ material. Performance based surface horizontal and vertical response spectra (PBSRS) at the design grade elevation were developed as described in Subsection 2.5.2.6. Figure 2.5.2-297 presents the comparison of the AP1000 CSDRS with the scaled PBSRS for horizontal and vertical ground motions. The CSDRS envelops the scaled horizontal and the vertical PBSRS.

Figures 3.7-206 and 3.7-207 show the conceptual grading plan and the conceptual grading section for the LNP site respectively. The plant Nuclear Island (NI) footprint (approximately 0.8 acres for each unit) is small compared to the approximately 347 acres where fill will be placed to raise the existing grade level. The existing grade in the plant footprint area is at approximate elevation 12.8 m (42 ft.) NAVD88. The design grade in the 347 acre fill area will vary from elevation 15.2 m (50 ft.) NAVD88 to elevation 14.3 m (47 ft.) NAVD88. The large extent of the fill area compared to the NI footprint and because the PBSRS is higher than the GMRS for the LNP site, the fill to design grade was included in the DC/COL-ISG-017 free field response analysis and the SSI analysis presented in Subsection 3.7.2.4.1.

The backfill provides lateral support to the drilled shafts supporting the Turbine Building (TB), Annex Building (AB), and Radwaste Building (RB). Thus, the backfill will be controlled engineered fill under the footprint of the TB, AB, and RB and to a lateral extent of ~30 ft. beyond the building footprint as shown in Figure 3.7-208. The remainder of the fill required for site grading shown in Figure 3.7-206 will not be controlled engineered fill. As shown in Figure 3.7-209, the TB, AB, and RB buildings are supported on 3 ft., 4 ft., and 6 ft. diameter drilled shafts. The seismic II/I interaction evaluations show that for drilled shafts up to 6 ft. in diameter, the lateral stiffness of the drilled shafts is primarily dependent on the soil property of the top 16 ft. of soil. The ~30 ft. lateral extent of the controlled engineered fill corresponds to the lateral extent of the passive

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wedge for engineered fill with a friction angle of 34 degrees as specified in Table 2.5.4.5-201.

Add Subsection 3.7.1.1.2 as follows:

3.7.1.1.2 Foundation Input Response Spectra

The nuclear island is supported on 10.7 meters (35 feet) of roller compacted concrete over rock formations at the site as described in Subsection 2.5.4.5. As described in Subsection 2.5.2.6.6, foundation input response spectra (FIRS) were developed at elevation -7.3 m (-24 ft.) NAVD88, the base of planned excavation beneath the nuclear island. This FIRS was scaled to ensure that the computed soil column outcropping response (SCOR) at the AP1000 foundation elevation 3.4 m (11 ft.) NAVD88 meets the 0.1g minimum ZPA requirement of 10 CFR 50 Appendix S. The scaled SCOR FIRS at elevation -7 m (-24 ft.) NAVD88 and at elevation 3.4 m (11 ft.) NAVD88 are shown on Figures 3.7-201 and 3.7-205 respectively.

As shown in Figure 2.5.2-358, the CEUS SSC horizontal and vertical FIRS is enveloped by the updated EPRI SOG scaled horizontal and vertical FIRS used for site specific soil structure interaction analysis described in Subsection 3.7.2.4.1. Thus, the conclusions of the soil structure analysis presented in Subsections 3.7.2.4.1.5 and 3.7.2.4.1.6 are valid for the LNP site ground motions based on the CEUS SSC model.

The seismic Category II and non-seismic adjacent structures are supported on drilled shafts. The top of the basemat for the Annex Building, Radwaste Building, and the Turbine Building (except for the condenser pit area) is at design grade elevation 15.5 m (51 ft.) NAVD88. The PBSRS described in Subsection 3.7.1.1.1 (Figure 2.5.2-297 and Table 2.5.2-227) are used to compute the maximum relative displacements of the Annex Building, Turbine Building, and the Radwaste Building drilled shaft foundation with respect to the nuclear island to evaluate site-specific aspect of the seismic interaction of these buildings with the nuclear island.

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As shown in Figure 2.5.2-357, the CEUS SSC PBSRS is enveloped by the updated EPRI SOG scaled PBSRS used for site specific displacement of the Annex Building, Turbine Building, and the Radwaste Building as described in Subsections 3.7.2.8.1, 3.7.2.8.2, and 3.7.2.8.3. Thus the conclusions in these subsections of no seismic interaction between the Annex Building, Turbine Building, and Radwaste Building and the NI is valid for the LNP site ground motions based on the CEUS SSC model.

Add the following subsections after DCD Subsection 3.7.2.4.

3.7.2.4.1 Site Specific Soil Structure Analysis

LNP SUP 3.7-6

LNP SUP 3.7-3

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 from a Performance Based Surface Response Spectra (PBSRS) is less than 2.5 cm (1 in.). 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

As a sensitivity analysis, the median centered probable maximum relative displacements between the NI and the adjacent Turbine, Annex, and Radwaste Buildings' foundation mat were calculated for updated EPRI SOG 10^{-5} UHRS. The drilled shaft supported foundation mat lateral displacements were obtained from 21 randomly selected soil profiles from the set of several hundred randomized soil profiles used to develop the updated EPRI SOG 10^{-5} UHRS. The median shear wave velocity profile for the 21 soil profiles closely matches the median shear wave velocity profile for the entire set of randomized soil profiles used to develop the updated EPRI SOG 10^{-5} UHRS as shown in Figure 3.7-227. The probable maximum relative displacement between the NI and the TB, AB, and the RB foundation mats was computed by combining the soil column displacements for UHRS, the NI displacement at the design grade, and the

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Turbine, Annex, and Radwaste Buildings' foundation mat displacements for updated EPRI SOG 10^{-5} UHRS using the square root of the sum of squares (SRSS) method. The computed probable maximum median relative displacements between the NI and the adjacent Turbine, Annex, and Radwaste Buildings' foundation mat for updated EPRI SOG 10^{-5} UHRS are less than 2.5 cm. (1 in.). 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, Annex, and Radwaste Buildings' foundation mat 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 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 resulting from the relative displacements during the seismic event. Thus, no seismic interaction between the Turbine, Annex, and the Radwaste Buildings' foundation mat and the NI is expected for updated EPRI SOG 10^{-5} UHRS.

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To evaluate the HCLPF capacity for no seismic interaction between the AB, TB, and RB foundation mats and the NI, the relative displacement between the NI and the AB, TB, and RB foundations was computed based on the updated EPRI SOG 10^{-5} UHRS. 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 10^{-5} UHRS. Thus, HCLPF capacity for no seismic interaction between the AB, TB, and RB foundation mats and the NI exceeds the 1.67*GMRS goal for the plant level HCLPF for the CEUS SSC ground motions.

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3.7.2.12 Methods for Seismic Analysis of Dams

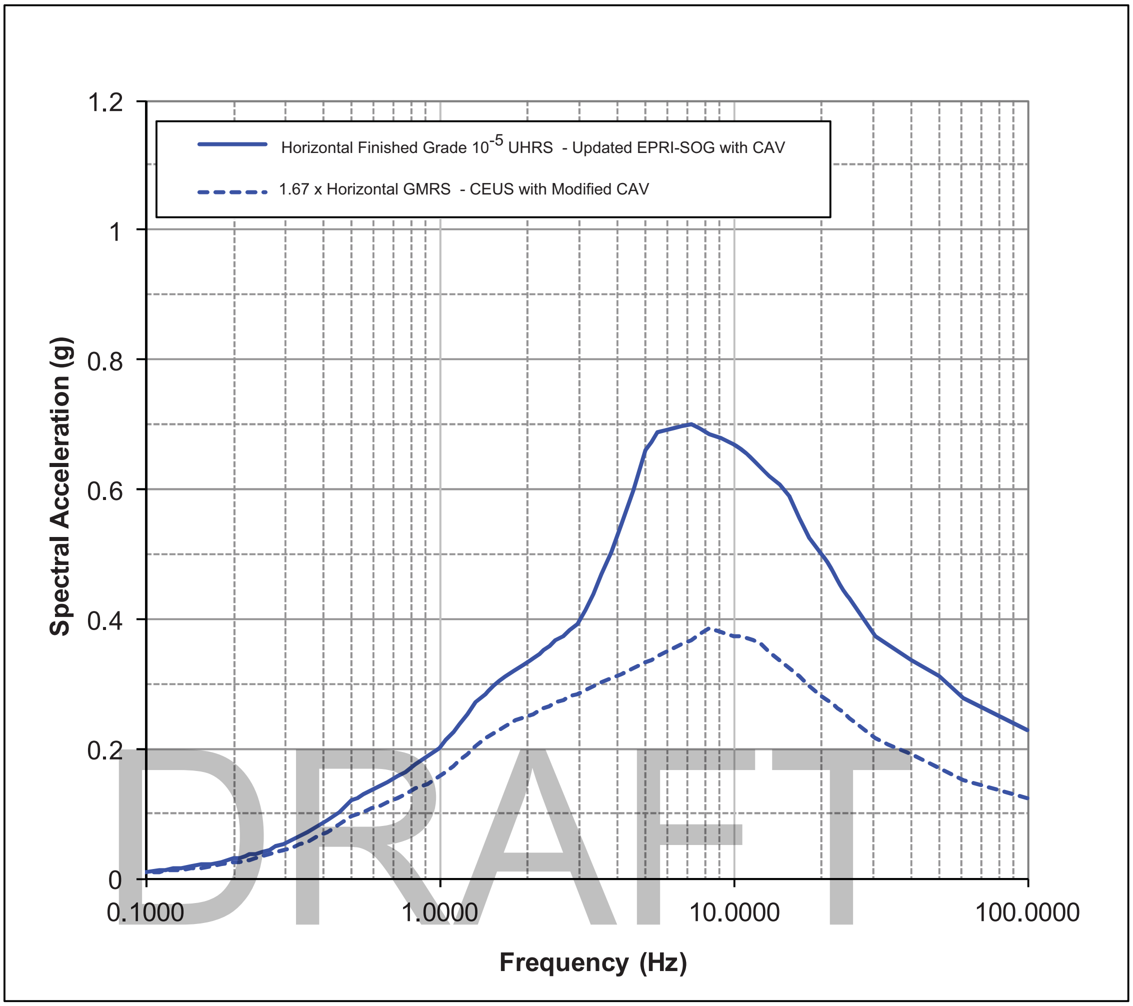
Add the following text to the end of DCD Subsection 3.7.2.12.

LNP COL 3.7-1 There are no existing dams that can affect the site interface flood level as specified in DCD Subsection 2.4.1.2 and discussed in FSAR Subsection 2.4.4.

3.7.4.1 Comparison with Regulatory Guide 1.12

Add the following text to the end of DCD Subsection 3.7.4.1.

STD SUP 3.7-1 Administrative procedures define the maintenance and repair of the seismic instrumentation to keep the maximum number of instruments in-service during plant operation and shutdown in accordance with Regulatory Guide 1.12.



Progress Energy Florida

Levy Nuclear Plant
Units 1 and 2
Part 2, Final Safety Analysis Report

Comparison of the horizontal 10^{-5} UHRS using the Updated EPRI-SOG model with 1.67 x the GMRS using the CEUS SSC model with modified CAV

Figure 3.7-228

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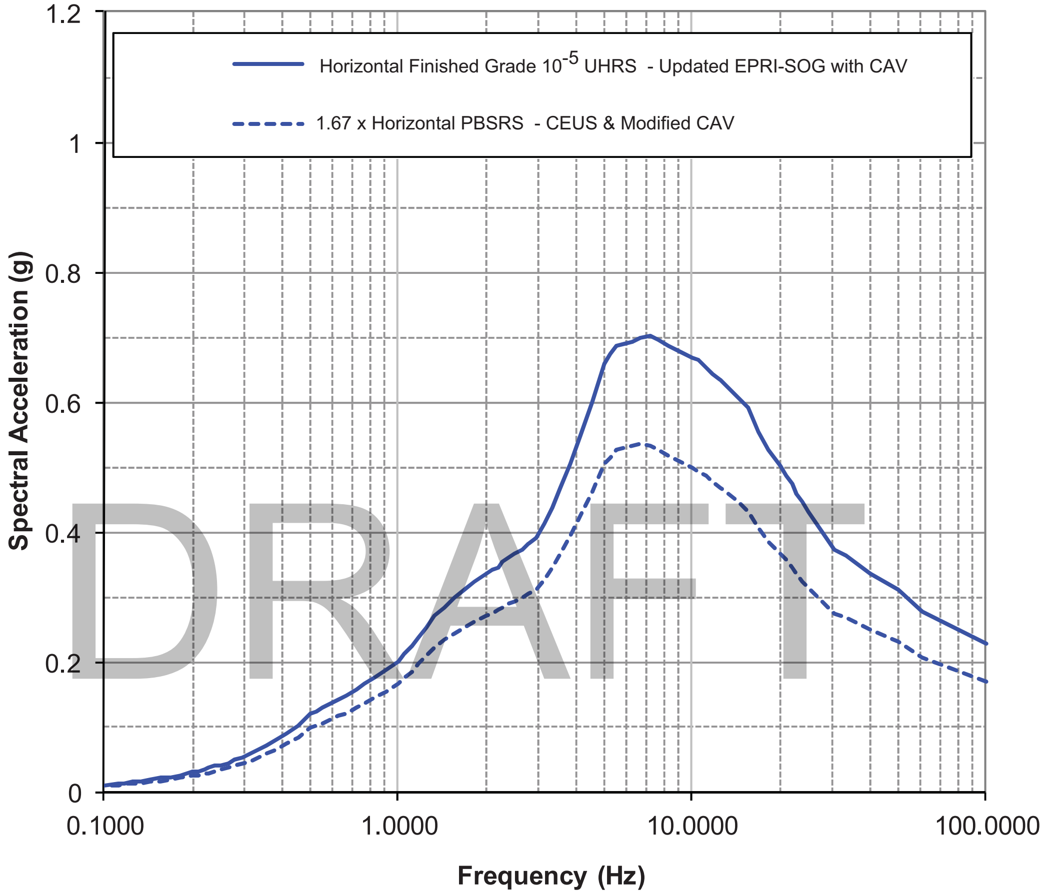

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Units 1 and 2
Part 2, Final Safety Analysis Report

Comparison of the horizontal 10^{-5} UHRS using the Updated EPRI-SOG model with 1.67 x the PBSRS using the CEUS SSC model with modified CAV

Figure 3.7-229 Rev 5

19.55 SEISMIC MARGIN ANALYSIS

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Add the following Subsection after DCD Subsection 19.55.6.2:

19.55.6.3 Site-Specific Seismic Margin Analysis

LNP COL 19.59.10-6 The LNP GMRS was developed as the Truncated Soil Column Surface Response (TSCSR) on the uppermost in-situ competent material at elevation 11 m (36 ft.) NAVD88 as described in Subsection 2.5.2.6. Since plant design grade will be established at elevation 15.5 m (51 ft.) NAVD88 by engineered fill above in-situ material as noted in Subsection 2.5.4.5, performance based surface horizontal and vertical response spectra (PBSRS) at the design grade scaled to meet 10 CFR Part 50 Appendix S requirements were developed as described in Subsection 2.5.2.6. Both the LNP scaled GMRS and the scaled PBSRS are enveloped by the AP1000 Certified Seismic Response Spectra as documented in Subsection 2.5.2.6. In addition, LNP site-specific SSI analysis was performed to evaluate the effect of the LNP unique foundation conditions on seismic demand. It was determined that the LNP site-specific seismic floor response spectra (FRS) at the six key locations are enveloped by the AP1000 CSDRS based FRS at the six key locations. In addition, the LNP maximum bearing pressure is less than the CSDRS based maximum bearing pressure of 24 ksf for soft rock sites. For the 24 ksf bearing pressure, the LNP site specific bearing factor of safety is greater than the acceptable factor of safety for static and dynamic loadings (Subsection 2.5.4.10.1.1). The LNP SSI analysis results are documented in Subsection 3.7.1.1.1. Thus, LNP site unique foundation conditions do not lower the High Confidence Low Probability of Failure (HCLPF) values calculated for the certified design.

As shown in Figures 2.5.2-355 and 2.5.2-357, both the CEUS SSC GMRS and the PBSRS are enveloped by the AP1000 CSDRS. As discussed in Subsection 3.7.1.1.2, the CEUS SSC LNP site specific floor response spectra (FRS) at the six key locations are bounded by the CSDRS FRS. In addition, the CEUS SSC LNP site specific nuclear island maximum bearing pressure is less than the 24 ksf design value. Thus, LNP site unique foundation conditions and CEUS SSC ground motions do not lower the High Confidence Low Probability of Failure (HCLPF) values calculated for the certified design.

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The soils under the LNP 1 and LNP 2 nuclear islands (NI) foundations will be excavated to rock and backfilled with Roller Compacted Concrete (RCC), as discussed in Subsection 2.5.4.5.3. For the NI, this eliminates any potential site-specific effects such as seismically induced liquefaction settlements, slope stability, foundation failure or relative settlements that would lower the HCLPF values calculated for the certified design. As described in Subsection 2.5.4.8, the LNP site-specific soil conditions also do not affect the nuclear island sliding and

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overturning stability based on Westinghouse analysis. Thus, LNP site-specific soil conditions do not lower the HCLPF values calculated for the certified design.

As described in Subsection 2.5.4.8, LNP site-specific liquefaction analysis (for PBSRS) was performed for soil beyond the nuclear island perimeter which will be left in place. Based on the liquefaction analysis, it was concluded that liquefiable zones under the LNP 1 and 2 footprints are confined to the northwest corner of the Unit 2 Turbine Building and in isolated random pockets under the remaining LNP 1 and 2 footprints. The LNP earthwork design will incorporate vertical and horizontal drains that will prevent liquefaction in the northwest corner of the Unit 2 Turbine Building and in isolated random pockets under the remaining LNP 1 and 2 footprints. The extent of these horizontal and vertical drains is shown in Figures 2.5.4.8-205 and 2.5.4.8-206. Liquefaction analysis was also performed for 10^{-5} uniform hazard response spectra (UHRS) for soil beyond the nuclear island perimeter which will be left in place as is described in Subsection 2.5.4.8. Based on this liquefaction analysis, it can be concluded that liquefiable zones under the LNP 1 and 2 footprints for 10^{-5} UHRS are confined soil zones where LNP earthwork design will incorporate vertical and horizontal drains that prevent liquefaction (Figures 2.5.4.8-205 and 2.5.4.8-206). The 10^{-5} UHRS is greater than 1.67 times the LNP scaled GMRS and the scaled PBSRS developed using the updated EPRI SOG model, and the GMRS and the PBSRS developed using the CEUS SSC model and modified CAV filter. Thus, liquefaction potential of soil beyond the nuclear island perimeter which will be left in place has the potential to drive the plant level HCLPF; however the soil liquefaction HCLPF exceeds the 1.67*GMRS goal for the plant level HCLPF.

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Seismic Category II structures (Annex Building [AB] and the first bay of the Turbine Building [TB]) and nonsafety-related structures (rest of the TB and Radwaste Building [RB]) adjacent to the NI will be supported on drilled shaft foundations. The Seismic Category II/I interaction issues between the adjacent drilled shaft supported structures and the NI have been addressed in Subsections 3.7.2.8.1, 3.7.2.8.2, and 3.7.2.8.3. The probable maximum relative displacements between the NI and the adjacent Turbine, Annex, and Radwaste Buildings' foundation mat for the PBSRS and the 10^{-5} UHRS are less than the 50 mm (2.0 inch) gap between the NI and the adjacent buildings' foundation mats. The 10^{-5} UHRS is greater than 1.67 times higher than the LNP scaled GMRS and the scaled PBSRS developed using the updated EPRI SOG model and the GMRS and the PBSRS developed using the CEUS SSC model and modified CAV filter. Thus, Seismic Category II/I interaction between the NI and the adjacent buildings has the potential to drive the plant level HCLPF; however the HCLPF for Seismic Category II/I interaction between the NI and the adjacent buildings exceeds the 1.67*GMRS goal for the plant level HCLPF.

The LNP RCC bridging mat is designed to span the postulated (conservative) design basis karst void of 10 ft. The failure of the RCC bridging mat can result in displacement of the AP1000 nuclear island foundation in excess of the maximum 6 in. displacements specified in DCD Tier 1 Table 5.0-1. In the AP1000 PRA-based Seismic Margin Assessment, the RCC bridging mat failure is conservatively assumed to fall within the gross structural collapse event modeled

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in the hierarchical event tree discussed in DCD Section 19.55. As gross structural collapse is assumed to directly lead to core damage, failure of the RCC bridging mat has the potential to drive the plant level high confidence low probability of failure (HCLPF) value. The HCLPF capacity of the RCC mat was calculated as >0.14g using the conservative deterministic failure margin (CDFM) methodology of Reference 19.55.7-201. The >0.14g HCLPF capacity of the RCC bridging mat exceeds the overall plant HCLPF acceptance criteria of 1.67*scaled GMRS using the updated EPRI SOG model and the 1.67*GMRS developed using the CEUS SSC model and modified CAV filter.

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Table 19.55-201 summarizes the HCLPF capacities of the LNP site-specific design features (e.g., RCC bridging mat, potential against soil liquefaction, and Seismic Category II/I interaction between the nuclear island and the adjacent buildings).

Thus, it can be concluded that the Seismic Margin Assessment analysis documented in Section 19.55 is applicable to the LNP site. Exceeding the HCLPF capacities for soil liquefaction and Seismic Category II/I interaction effects of buildings adjacent to the nuclear island will not affect the plant level HCLPF capacity. The RCC bridging mat HCLPF capacity, while potentially driving the plant-level HCLPF, exceeds the plant level HCLPF goal of 1.67*scaled GMRS using the updated EPRO SOG model and the GMRS developed using the CEUS SSC model and modified CAV filter.

19.55.7 REFERENCES

Add the following information at the end of DCD Subsection 19.55.7:

201. EPRI Report No. NP-6041-SL, “A Methodology for Assessment of Nuclear Power Plant Seismic Margin”, Revision 1, August 1991.

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Table 19.55-201
HCLPF Capacities for LNP Site Specific Design Features

LNP COL 19.59.10-6

Description	HCLPF Capacity[(a)]	HCLPF/GMRS[(b)]	Basis
Soil Liquefaction Potential under Adjacent Buildings	> 0.14g	> 1.67 GMRS	(c)
Seismic II/I Interaction Potential	> 0.14g	> 1.67 GMRS	(d)
RCC bridging mat	>0.14g	>1.67 GMRS	(e)

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Notes:

a) LNP scaled Ground Motion Response Spectra (GMRS) peak ground acceleration (PGA) is 0.084g using updated EPRI SOG model (Subsection 2.5.2.6). The GMRS PGA using CEUS SSC model and modified CAV filter is 0.073g (Subsection 2.5.2.7).

b) HCLPF Capacity as a fraction of LNP updated EPRI SOG scaled GMRS PGA.

c) Liquefaction potential of soils under the adjacent buildings was evaluated for the LNP updated EPRI SOG 10^{-5} annual exceedance probability Uniform Hazard Response Spectra (10^{-5} UHRS). The LNP updated EPRI SOG 10^{-5} UHRS is greater than 1.67*scaled GMRS using the updated EPRI SOG model (Subsection 2.5.2.6) and the CEUS SSC GMRS with the modified CAV filter (Subsection 2.5.2.7).

d) Relative displacement between the NI and adjacent buildings for the LNP updated EPRI SOG 10^{-5} UHRS is less than the gap provided. The LNP updated EPRI SOG 10^{-5} UHRS is greater than 1.67*scaled GMRS using the updated EPRI SOG model (Subsection 2.5.2.6) and the CEUS SSC GMRS with the modified CAV filter (Subsection 2.5.2.7).

e) HCLPF capacity calculated using conservative deterministic failure margin method of Reference 19.55.7-201.

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