CHAPTER 3

DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

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CHAPTER 3

DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

3.1 CONFORMANCE WITH NUCLEAR REGULATORY COMMISSION GENERAL DESIGN CRITERIA

This section of the referenced DCD is incorporated by reference with no departures or supplements.

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

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

3.2.1 SEISMIC CLASSIFICATION

Add the following text to the end of DCD Subsection 3.2.1.

LNP SUP 3.2-1

There are no safety-related structures, systems, or components outside the scope of the DCD, except for roller compacted concrete (RCC) which is classified as a seismic Category I, safety-related structure. See Table 3.2-201. Refer to Subsections 2.5.4.5 and 2.5.4.12 for a discussion of safety-related RCC.

The nonsafety-related structures, systems, and components outside the scope of the DCD are classified as non-seismic (NS).

3.2.1.3 Classification of Building Structures

Add the following text to the end of DCD Subsection 3.2.1.3.

LNP SUP 3.2-2

The seismic classification of the makeup water pump house (See Figure 1.1-201, Sheet 2), Unit 1 freshwater raw water pump house, Unit 2 freshwater raw water pump house, Unit 1 potable water pump house, and Unit 2 potable water pump house are provided in Table 3.2-201.

3.2.2 AP1000 CLASSIFICATION SYSTEM

Add the following text to the end of DCD Subsection 3.2.2.

LNP SUP 3.2-1

There are no safety-related structures, systems, or components outside the scope of the DCD, except for roller compacted concrete (RCC) which is classified as a seismic Category I, safety-related structure. See Table 3.2-201. Refer to Subsections 2.5.4.5 and 2.5.4.12 for a discussion of safety-related RCC.

Table 3.2-201
Seismic Classification of Building Structures

	Structure[1]	Category
LNP SUP 3.2-2	Unit 1 Freshwater Raw Water Pump House	NS
	Unit 2 Freshwater Raw Water Pump House	NS
	Makeup Water Pump House	NS
	Unit 1 Potable Water Pump House	NS
	Unit 2 Potable Water Pump House	NS
LNP SUP 3.2-1	Roller Compacted Concrete	C-I

C-I – Seismic Category I
C-II – Seismic Category II
NS – Non-seismic

Note:
1. Within the broad definition of seismic Category I and II structures, these buildings contain members and structural subsystems the failure of which would not impair the capability for safe shutdown. Examples of such systems would not impair the capability for safe shutdown. Examples of such systems would be elevators, stairwells not required for access in the event of a postulated earthquake, and nonstructural partitions in nonsafety-related areas. These substructures are classified as non-seismic.

3.3 WIND AND TORNADO LOADINGS

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

3.3.1.1 Design Wind Velocity

Add the following text to the end of DCD Subsection 3.3.1.1.

LNP COL 3.3-1
LNP COL 3.5-1

The wind velocity characteristics for the Levy Nuclear Plant, Units 1 and 2 (LNP 1 and 2) are given in Subsection 2.3.1.2.2. These values are bounded by the design wind velocity values given in DCD Subsection 3.3.1.1 for the AP1000 plant.

3.3.2.1 Applicable Design Parameters

Add the following text to the end of DCD Subsection 3.3.2.1.

LNP COL 3.3-1
LNP COL 3.5-1

The tornado characteristics for the LNP 1 and 2 are given in Subsection 2.3.1.2.2. These values are bounded by the tornado design parameters given in DCD Subsection 3.3.2.1 for the AP1000 plant.

The 10^{-7} annual non exceedance probability hurricane wind speed of 195 mph at the LNP site based on Regulatory Guide 1.221 is bounded by the design tornado wind speed given in DCD Subsection 3.3.2.1.

3.3.2.3 Effect of Failure of Structures or Components Not Designed for Tornado Loads

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Add the following text to the end of DCD Subsection 3.3.2.3.

STD COL 3.3-1
LNP COL 3.5-1

Consideration of the effects of wind and tornado due to failures in an adjacent AP1000 plant are bounded by the evaluation of the buildings and structures in a single unit.

3.3.3 COMBINED LICENSE INFORMATION

Add the following text to the end of DCD Subsection 3.3.3.

LNP COL 3.3-1 The LNP 1 and 2 site satisfies the site interface criteria for wind and tornado (see Subsections 3.3.1.1, 3.3.2.1, and 3.3.2.3) and will not have a tornado-initiated failure of structures and components within the applicant's scope that compromises the safety of AP1000 safety-related structures and components (see also Subsection 3.5.4).

Subsection 1.2.2 discusses differences between the plant specific site plan (see Figure 1.1-201) and the AP1000 typical site plan shown in DCD Figure 1.2-2.

There are no other structures adjacent to the nuclear island other than as described and evaluated in the DCD.

Missiles caused by external events separate from the tornado are addressed in Subsections 2.2 through 2.2.3, 3.5.1.3, 3.5.1.5, and 3.5.1.6.

3.4 WATER LEVEL (FLOOD) DESIGN

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

3.4.1.3 Permanent Dewatering System

Add the following text to the end of DCD Subsection 3.4.1.3.

LNP COL 3.4-1 No permanent dewatering system is required because site groundwater levels are two feet or more below site grade level as described in Subsection 2.4.12.5.

3.4.3 COMBINED LICENSE INFORMATION

Replace the first paragraph of DCD Subsection 3.4.3 with the following text.

LNP COL 3.4-1 The site-specific water levels given in Section 2.4 satisfy the interface requirements identified in DCD Section 2.4.

3.5 MISSILE PROTECTION

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

3.5.1.3 Turbine Missiles

Add the following text to the end of DCD Subsection 3.5.1.3.

STD SUP 3.5-1

The potential for a turbine missile from another AP1000 plant in close proximity has been considered. As noted in DCD Subsection 10.2.2, the probability of generation of a turbine missile (or P1 as identified in SRP 3.5.1.3) is less than 1 x 10^{-5} per year. This missile generation probability (P1) combined with an unfavorable orientation P2 x P3 conservative product value of 10^{-2} (from SRP 3.5.1.3) results in a probability of unacceptable damage from turbine missiles (or P4 value) of less than 10^{-7} per year per plant which meets the SRP 3.5.1.3 acceptance criterion and the guidance of Regulatory Guide 1.115. Thus, neither the orientation of the side-by-side AP1000 turbines nor the separation distance is pertinent to meeting the turbine missile generation acceptance criterion. In addition, the shield building and auxiliary building walls, roofs, and floors, provide further conservative, inherent protection of the safety-related SSCs from a turbine missile.

STD SUP 3.5-2

The turbine system maintenance and inspection program is discussed in Subsection 10.2.3.6.

3.5.1.4 Missiles Generated by Natural Phenomenon

Add the following text to the end of DCD Subsection 3.5.1.4.

LNP COL 3.3-1
LNP COL 3.5-1

Hurricane missiles are defined in accordance with Regulatory Guide 1.221, October 2011. The hurricane missile parameters considered for the LNP site are summarized in Table 3.5-202.

3.5.1.5 Missiles Generated by Events Near the Site

Add the following text to the end of DCD Subsection 3.5.1.5.

LNP COL 3.3-1
LNP COL 3.5-1

The gate house, administrative building, security control building, warehouse and shops, water service building, diesel-driven fire pump/enclosure, and miscellaneous structures are common structures that are at a nuclear power plant. They are of similar design and construction to those that are typical at nuclear power plants. Therefore, any missiles resulting from a tornado-initiated failure are not more energetic than the tornado missiles postulated for design of the AP1000.

The missiles generated by events near the site are discussed and evaluated in Subsection 2.2.3. The effects of external events on the safety-related components of the plant are insignificant.

3.5.1.6 Aircraft Hazards

Add the following text to the end of DCD Subsection 3.5.1.6.

LNP COL 3.3-1
LNP COL 3.5-1

The outer boundary of five airways is routed within 2 miles of the LNP site: V7-521, VR 1006, J119, Q110-116-118 and Q112 (shown on Figure 2.2.1-204). Thus, an aircraft hazards evaluation was performed for LNP 1 and 2.

The evaluation determined that the probability of small aircraft crashing on seismic category I structures (i.e. Containment/Shield Building and Auxiliary Building) is calculated to be 7.011 x 10^{-6} per year. This crash probability results in a core damage frequency (CDF) of 0.410 x 10^{-12} per year which is much smaller than the current plant CDF acceptance criteria of 1.0 x 10^{-8} per year. Therefore, small aircraft crash probability is acceptable. The probability of large aircraft crashing on seismic category I structures is calculated as 3.093 x 10^{-8} per year. This meets the acceptance criteria of 1 x 10^{-7} per year in Subsection 19.58.2.3.1 of DCD. Therefore, the probability of crash for large aircrafts is acceptable. The acceptance criteria and methodology are discussed below.

<u>Probabilistic Acceptance Criteria</u>

Based on discussion in Subsection 19.58.2.3.1 of the DCD, separate probabilistic acceptance criteria are used for small and large aircrafts. The definition of small and large aircraft is based on documented discussion with Westinghouse.

Small aircraft is an aircraft with less than 30 seats with pay load less than 7500 pounds. All aircraft not meeting the above small aircraft definition are considered as large aircraft.

- Acceptance Criteria for Large Aircraft:

  Total probability of crash on Seismic Category I structures must be less than 1 x 10^{-7} per year.

- Acceptance Criteria for Small Aircraft:

  Equation 19.58-1 of the DCD will be applied with the initiating event frequency (IEF) equal to the calculated small aircraft crash probability per year. The small aircraft crash probability is acceptable if the calculated core damage frequency is less than 1.0 x 10^{-8} per year.

The calculation details for the airways follows:

Calculation for Airways

Item 2 of Section III of SRP 3.5.1.6 (Reference 201) provides an equation to calculate probability of crash from a nearby airway. This equation contains a constant

C = in-flight crash rate per mile using the airway

For commercial aircraft, a C value of 4 x 10^{-10} per aircraft mile is provided in Reference 201. However, the reference does not provide C values for other types of aircraft (i.e., military aviation and general aviation). Because of the above unavailability of constant C for all aircraft types and since FAA does not provide clear flight information on specific airways, the Reference 201 equation for airways is not used in this assessment for airways.

Section 5.3.2 of DOE-STD-3014-96 (Reference 202) provides complete equations for calculating probability of aircraft crash from non-airport operations. The procedure is implemented using Tables in Appendix B of Reference 202.

The probability of crash from airways is calculated using the equation below:

$$P_{all\_airways} = \sum_{j} (N_i \cdot P_j \cdot f_j \cdot A_j) \qquad (1)$$

N_j*P_j = expected number of in-flight crashes per year for aircraft type j (occurrence per year)

f_j = conditional probability, given a crash, that the crash occurs within one-square-mile area surrounding the facility of interest (per square mile)

A_j = impact area of the buildings of facility for aircraft type j (square mile)

Effective plant impact area is calculated by considering only Seismic Category I structures. Per the DCD, this is restricted to Containment/Shield Building and Auxiliary Building. As required by Item 7 in Section III of SRP 3.5.1.6 (Reference 201), A_j must include appropriate fly-in area and skid area. Additional details are not provided in SRP. The methodology in Section B.4 of the DOE-STD-3014-96 (Reference 202) provides details for buildings of rectangular foot print and of constant height above grade.

The value of A_j depends on the aircraft type because of differences in wing spans, crash angle, and skid distance. Table 3.5-201 lists the total areas for different aircraft types.

Values of $N_j{*}P_j{*}f_j$ are provided in Table B-14 of Reference 202 for General aviation and in Table B-15 of Reference 202 for commercial and military aviations.

When Using Tables B-14 and B-15, the maximum value listed for Savannah River Site and average Continental United States (CONUS) was used. Savannah River Site information is included because Savannah River Site is closest of all sites listed in these tables to LNP site.

Calculated Crash Probability Results

The following aircraft types are considered as "small" aircrafts: air taxi, general aviation and small military. Large aircrafts are considered to be: air carrier and large military aircraft.

With the above identification of large and small aircrafts, the results are:

$$P_{small} = P_{small\ airway}$$

$$P_{small} = 7.011 \times 10^{-6} \text{ per year}$$

$$P_{large} = P_{large\_airway}$$

$$P_{large} = 3.093 \times 10^{-8} \text{ per year}$$

Conclusions from Probability Results

For large aircraft, acceptance criterion is 1 x 10^{-7} per year. Therefore, large aircraft crash probability of 3.093 x 10^{-8} is acceptable.

For small aircraft, apply Equation (19.58-1) of the DCD with conditional core damage probability (CCDP) of 5.85 x 10^{-8}. Plant core damage frequency is:

$$CDF_{small\_aircraft} = (7.011 \times 10^{-6}) \times (5.85 \times 10^{-8}) = 0.410 \times 10^{-12} \text{ per year}$$

The core damage frequency due to small aircraft crash is much smaller than the core damage frequency acceptance criteria of 1.0 x 10^{-8} per year, and the calculated small aircraft crash probability is acceptable.

3.5.2 PROTECTION FROM EXTERNALLY GENERATED MISSILES

Add the following text to the end of DCD Subsection 3.5.2.

LNP COL 3.5-1

Hurricane wind and missile velocities are based on an annual non exceedance probability of 10^{-7}, the same as that for tornados in Regulatory Guide 1.76

Revision 1. Thus, using the tornado missile structural acceptance criteria for the hurricane winds and missiles evaluation is appropriate.

The comparison between the DCD Tier 1 Table 5.0-1 tornado generated missile parameters and the Regulatory Guide 1.221, October 2011 (RG 1.221) based LNP site-specific hurricane generated missile parameters are summarized in Table 3.5-202. The hurricane generated missile velocities are based on maximum hurricane wind speed of 195 mph at the LNP site, using the figures and tables in RG 1.221. The LNP site-specific hurricane generated missiles evaluation can be summarized as follows:

- For the 1-in steel sphere, the DCD Tier 1 Table 5.0-1 velocity for tornado generated missiles bounds the hurricane generated missile. Thus, no additional evaluation is required.
- For the 6.625-in. diameter pipe missile, the LNP site specific hurricane generated missile horizontal velocity is 93 mph. For this missile the minimum concrete (f'_c=4,000 psi) thickness required to prevent penetration or scabbing is 17 inches. The LNP site specific hurricane generated missile vertical velocity is 58 mph. For this missile the minimum concrete (f'_c=4,000 psi) thickness required to prevent penetration or scabbing is less than 13 inches. As stated in DCD Subsection 3.5.3, the minimum thicknesses of the nuclear island exterior walls above grade and roof is 24 inches and 15 inches, respectively. The minimum concrete f'_c of 4,000 psi is used for LNP nuclear island structures per DCD Subsection 3.8.4.6.1.1. For impact, the energy of the 8 inch shell tornado missile specified in DCD Tier 1 Table 5.0-1 bounds the energy of the corresponding LNP site specific 6.625 inch pipe missile. Thus, the LNP nuclear island is adequately protected against the 6.625-in. diameter pipe hurricane generated missile.
- For the 4,000 lbs automobile missile, the LNP site specific hurricane generated missile vertical velocity is 58 mph. This is bounded by the DCD Tier 1 Table 5.0-1 tornado generated automobile missile vertical velocity of 74 mph and no further evaluation is required. For the 4,000 lbs automobile missile, the LNP site specific hurricane generated missile horizontal velocity is 120 mph. The 120 mph automobile horizontal missile velocity is greater than the DCD Tier 1 Table 5.0-1 tornado generated automobile missile horizontal missile velocity of 105 mph. Thus, for the hurricane generated automobile horizontal missile, an evaluation was performed to determine whether the LNP nuclear island exterior walls are adequate to withstand the effect of the automobile missile impact together with the 195 mph hurricane winds. This evaluation used the same methodology that was used for evaluation of the tornado generated automobile missile in DCD Subsection 3.5.2. Based on the evaluation, it was concluded that the LNP nuclear island is adequately protected against the hurricane generated automobile missile impact.

3.5.4 COMBINED LICENSE INFORMATION

Add the following text to the end of DCD Subsection 3.5.4.

LNP COL 3.5-1 The LNP site satisfies the site interface criteria for wind and tornado (see Subsections 3.3.1.1, 3.3.2.1, and 3.3.2.3) and will not have a tornado-initiated failure of structures and components within the applicant's scope that compromises the safety of AP1000 safety-related structures and components (see also Subsection 3.3.3).

Subsection 1.2.2 discusses differences between the plant specific site plan (see Figure 1.1-201) and the AP1000 typical site plan shown in DCD Figure 1.2-2.

There are no other structures adjacent to the nuclear island other than as described and evaluated in the DCD.

Missiles caused by external events separate from the tornado are addressed in Subsections 2.2 through 2.2.3, 3.5.1.3, 3.5.1.5, and 3.5.1.6.

3.5.5 REFERENCES

Add the following information at the end of DCD Subsection 3.5.5:

201. NUREG-0800, Standard Review Plant (SRP) 3.5.1.6, “Aircraft Hazards”, Rev. 3, March 2007.

202. Department of Energy Standard DOE-STD-3014-96, “Accident Analysis Into Hazardous Facilities”, October 1996.

Table 3.5-201
Impact Area for Combined Containment/Shield and Auxiliary Buildings for Different Aircrafts

LNP COL 3.3-1
LNP COL 3.5-1

Aircraft Type	A_j (mile^2)	
	Part I	Part II
Air Carrier	0.03415	0.01872
Air Taxi	0.01230	0.01630
General Aviation	0.00984	0.01290
Small Military	0.02035	0.01981
Large Military	0.02364	0.02529

LNP SUP 3.5-3

Table 3.5-202
Comparison between DCD Tornado and LNP Site-Specific Hurricane Missile Parameters

Missile Description	DCD Tornado Missile Velocity[a]	LNP Hurricane Missile Velocity [b]
Automobile (4,000 lbs)	105 mph horizontal 74 mph vertical	120 mph horizontal 58 mph vertical
8-in. Shell (275 lbs)	105 mph horizontal 74 mph vertical	-
6.625-in, diameter pipe (287 lbs)	-	93 mph horizontal 58 mph vertical
1-in. diameter steel sphere (0.147 lbs)	105 mph in most damaging direction	82 mph horizontal 58 mph vertical

Notes:
a) DCD Tier 1 Table 5.0-1
b) Based on RG 1.221 Table 2 and Figure 2

3.6 PROTECTION AGAINST THE DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

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

3.6.4.1 Pipe Break Hazard Analysis

Replace the last paragraph in DCD Subsection 3.6.4.1 with the following text.

STD COL 3.6-1

The as-designed pipe rupture hazards evaluation is made available for NRC review. The completed as-designed pipe rupture hazards evaluation will be in accordance with the criteria outlined in DCD Subsections 3.6.1.3.2 and 3.6.2.5. Systems, structures, and components identified to be essential targets protected by associated mitigation features (Reference is DCD Table 3.6-3) will be confirmed as part of the evaluation, and updated information will be provided as appropriate.

A pipe rupture hazard analysis is part of the piping design. The evaluation will be performed for high and moderate energy piping to confirm the protection of systems, structures, and components which are required to be functional during and following a design basis event. The locations of the postulated ruptures and essential targets will be established and required pipe whip restraints and jet shield designs will be included. The report will address environmental and flooding effects of cracks in high and moderate energy piping. The as-designed pipe rupture hazards evaluation is prepared on a generic basis to address COL applications referencing the AP1000 design.

The pipe whip restraint and jet shield design includes the properties and characteristics of procured components connected to the piping, components, and walls at identified break and target locations. The design will be completed prior to installation of the piping and connected components.

The as-built reconciliation of the pipe rupture hazards evaluation whip restraint and jet shield design in accordance with the criteria outlined in DCD Subsections 3.6.1.3.2 and 3.6.2.5 will be completed prior to fuel load (in accordance with DCD Tier 1 Table 3.3-6, item 8).

This COL item is also addressed in Subsection 14.3.3.

3.6.4.4 Primary System Inspection Program for Leak-before-Break Piping

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Replace the first paragraph of DCD Subsection 3.6.4.4 with the following text.

STD COL 3.6-4

Alloy 690 is not used in leak-before-break piping. No additional or augmented inspections are required beyond the inservice inspection program for leak-before-break piping. An as-built verification of the leak-before-break piping is required to verify that no change was introduced that would invalidate the conclusion reached in this subsection.

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 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 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 are 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.

As shown in Figure 2.5.2-357, the CEUS SSC PBSRS are 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 are 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 3.7.2.4.1.1 Soil Profiles for Soil Structure Analysis

LNP SUP 3.7-3 For the Soil Structure Analysis (SSI) analysis of the nuclear island (NI) the best estimate (BE), lower bound (LB), and upper bound (UB) soil profiles presented in Tables 2.5.2-228, 2.5.2-229, and 2.5.2-230 respectively were considered. In addition, to account for the potential degradation of soil shear modulus due to foundation installation, an additional Lower LB case (LLB) was also considered in the SSI analysis. The foundation construction activities that may affect the in-situ soil properties include installation of the drilled shafts, installation of the diaphragm wall, and installation of the rock anchors for the diaphragm wall. The construction methods and construction inspections used for installation of the drilled shafts, diaphragm wall, and the diaphragm wall anchors will minimize the extent of soil disturbance and avoid cave in. The holes for the anchors will be advanced using drilling techniques designed to minimize the disturbance to the surrounding soil. Such techniques may include the use of a casing, or drilling with water or drilling slurry (not air). The boreholes for the diaphragm wall anchors will be backfilled as the casing is extracted after the anchors are set in rock to avoid cave in. Alternatively, the casings will be backfilled and left in place. The drilled shaft construction methods and construction inspections and testing will follow guidance in ACI 336.1-01 and ACI 336.3R-93.

The volume of soil being disturbed by the drilled shaft installation, and diaphragm wall anchor installation is < 5 percent of the total soil volume in the vicinity of the NI. Assuming the disturbed soil around the drilled shaft and diaphragm wall anchors to have a soil shear modulus equal to half of the shear modulus of the corresponding soil layers, the average reduction in the soil shear modulus of the soil volume in the vicinity of the NI is < 2.5 percent. Thus, for the LLB soil profile, in-situ soil was conservatively assigned a shear modulus equal to 90 percent of the LB soil case as presented in Table 3.7-201. As shown in Table 3.7-201, the fill layer shear modulus was not changed from the LB shear modulus because of the large variation from the BE case already considered i.e., the coefficient of variation for the LB fill shear modulus is in the range of 4.02 to 6.13 from the BE fill shear modulus as shown in Table 3.7-201. Rock layer shear modulus for the LLB soil profile are the same as for the LB soil profile because the construction activities do not degrade the rock layer shear modulus.

3.7.2.4.1.2 DC/COL-ISG-017 Free Field Analysis

Design grade (elevation 15.5 m [51 ft.] NAVD88) deterministic surface spectra were developed using Subsection 5.2.1 of the Interim Staff Guidance DC/COL-ISG-017 as described in Subsection 2.5.2.6. The design grade surface response spectra from the three soil columns (best estimate, lower bound, and the upper bound properties) were developed using the scaled SCOR FIRS for elevation -7.3 m (-24 ft.) NAVD88, the base of planned excavation beneath the nuclear island. The three soil property profiles were developed based on the variation in the randomized soil profiles used for developing PBSRS and complying with SRP 3.7.2.II.4 guidance on soil property variation for SSI analysis. The shear wave velocity profiles for the upper bound (UB), best estimate (BE) and lower bound (LB) soil profiles are shown in Figure 2.5.2-298. The soil column profile and soil properties are presented in Tables 2.5.2-228, 229, and 230 for BE, LB, and UB cases respectively. Both horizontal and vertical SSI input response spectra were developed.

The envelope of the deterministic surface spectra for horizontal and vertical motions from the UB, LB, and BE envelops the PBSRS as required by DC/COL-ISG-017. This comparison is shown on Figures 3.7-202, 203, and 204. Figures 3.7-202 and 203 also present the comparison of the AP1000 CSDRS with the deterministic surface spectra from the UB, BE, and LB soil columns for the North-South (H1) and the East-West (H2) directions. The CSDRS envelops the deterministic surface spectra from the three soil columns for horizontal motions. For the vertical ground motions, Figure 3.7-204 presents the comparison of the AP1000 CSDRS with the deterministic surface spectra from the three soil columns for the vertical motions. The CSDRS does not envelop the deterministic surface spectra from the three soil columns in the high frequency range (greater than approximately 30 Hz). Thus, a LNP site-specific SSI analysis was performed.

3.7.2.4.1.3 Input Time Histories for Soil Structure Analysis

Input time histories for the SSI analysis were created in two steps. First, time histories were spectrally matched to the scaled SCOR FIRS at the base of the planned excavation (elevation -7.3 m (-24 ft.) NAVD88) shown in Figure 3.7-201. Then these time histories were input into the four (UB, BE, LB, and LLB) free field soil columns (full height to elevation 15.5 m (51 ft.) NAVD88) as outcropping motions and then output as in-column motion at the base of the excavation for use in the SSI analysis. As part of this process, the surface motion was computed for each of the four soil profiles and the SCOR FIRS was enhanced at intermediate frequencies to ensure that the surface motion envelops the PBSRS. The selected seed time history was the 1992 Landers Earthquake, Villa Park Serrano Ave station, chosen from the CEUS record library provided by NUREG/CR 6728. The seed time history was selected based on the seismological properties and spectral shape of both horizontal and vertical components. The selected time history represents a distance recording of a large (M 7.3) earthquake consistent with the dominant contribution to Levy site hazard by the Charleston source. Figures 3.7-210, 3.7-211, 3.7-212, and 3.7-213 show the in-column SSI input X, Y, and Z time histories at elevation -7.3 m (-24 ft.) NAVD88 for the Best Estimate (BE), Upper Bound (UB), Lower Bound (LB), and the Lower Lower Bound (LLB) soil profiles respectively.

3.7.2.4.1.4 Soil Structure Analysis Models

The LNP specific SSI analyses utilize both three dimensional (3D) and two dimensional (2D) models and SASSI Subtraction and Direct methods for computing in-structure floor response spectra.

The Design-Basis 3D model consists of a NI20r-derived, 5-Layer, 75-foot embedded Finite Element Model (FEM) developed for the BE soil case using the SASSI Direct method of analysis. An 8-Layer, 75-foot embedded 3D FEM was developed for sensitivity analysis of the LNP BE, UB, LB and LLB site soil cases utilizing the SASSI Subtraction method, and to confirm that the BE case is the controlling soil case particularly in the high frequency range. The 3D models capture the three dimensional response effects for the various site soil cases; however, the models are limited by the mesh size and corresponding passing model frequency based on the LNP site profile shear wave velocity and layer thickness. Therefore, two 2D models were developed to address 3D mesh size modeling, potential frequency filtering due to the 3D model layering, and to evaluate the SASSI SITE profile lower boundary depth.

The 2D 'Coarse' model was created to simulate the 3D design-basis embedded model in 2D. The 2D 'Fine' model was created to meet the SASSI wavelength criteria consistent with the NRC Interim Staff Guidance DC/COL-ISG-01 (ISG-01) 50 hertz model refinement frequency, and meet the lower boundary criteria specified in ASCE 4-98 Section 3.3.3.2 of at least 8000 fps or three times the maximum foundation dimension (~750 feet). The 2D SSI analyses utilized the SASSI Direct method. The results of the 2D SSI analyses determine the frequency-dependent ratio of Fine-to-Coarse response spectra (≥ 1.0), (i.e. Bump Factor), which is subsequently applied to the 3D BE Design-Basis FRS for comparison to the AP1000 generic and HRHF FRS envelopes.

The Turbine Building (TB), Annex Building (AB), and Radwaste Building (RB) drilled shafts and the diaphragm wall was not modeled in the 3D SSI model. The absence of any adverse Category II/I interaction between the NI and the TB, AB, and RB for LNP is documented in Subsections 3.7.2.8.1, 3.7.2.8.2, and 3.7.2.8.3.

3.7.2.4.1.5 Soil Structure Analysis

SASSI SSI analyses using the 3D and 2D models were performed considering the simultaneous occurrences of the two horizontal and one vertical components of the time history. The input time history (Subsection 3.7.2.4.1.3) was applied as in-column motions at elevation -7.3 m (-24 ft.) NAVD88. The floor response time histories in the X, Y, and Z directions were obtained by algebraically combining the co-directional acceleration time histories from the three excitations. Floor response spectra (FRS) were generated for the six key AP1000 locations using 5 percent damping. These locations include: CIS at Reactor Vessel Support Elevation (Node 1761), ASB NE Corner at Control Room Floor (Node 2078), CIS at Operating Deck (Node 2199), ASB Corner of Fuel Building Roof at Shield Building (Node 2675), SCV near Polar Crane (Node 2788), and ASB Shield Building Roof Area (Node 3329).

The first SSI analysis was performed using the 3D 8-Layer embedded model and the BE, UB, LB and LLB soil profiles. The SASSI Subtraction method was used. The LNP specific broadened 5 percent damped FRS computed at the six key locations for 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. The figures show that the LNP FRS are enveloped by the AP1000 generic FRS at all of the six NI key nodes. The FRS also confirm that the BE soil profile FRS are the controlling FRS in the critical 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 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.

3.7.2.8.1 Annex Building

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

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 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.

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 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.

To evaluate the HCLPF capacity for no seismic interaction between the Annex Building, Turbine Building, and Radwaste Building foundation mats and the NI, the relative displacement between the NI and the Annex Building, Turbine Building, and Radwaste Building 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 method and modified CAV filter are enveloped by the updated EPRI-SOG 10^{-5} UHRS. Thus, HCLPF capacity for no seismic interaction between the Annex Building, Turbine Building, and Radwaste Building foundation mats and the NI exceeds the 1.67*GMRS goal for the plant level HCLPF for the CEUS SSC ground motions.

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.

3.7.4.2.1 Triaxial Acceleration Sensors

Add the following text to the end of DCD Subsection 3.7.4.2.1.

STD COL 3.7-5 A free-field sensor will be located and installed to record the ground surface motion representative of the site. It will be located such that the effects associated with surface features, buildings, and components on the recorded ground motion will be insignificant. The trigger value is initially set at 0.01g.

3.7.4.4 Comparison of Measured and Predicted Responses

Add the following text to the end of DCD Subsection 3.7.4.4.

LNP COL 3.7-2 Post-earthquake operating procedures utilize the guidance of EPRI Reports NP5930, TR-100082, and NP-6695, as modified and endorsed by the NRC in Regulatory Guides 1.166 and 1.167. A response spectrum check up to 10Hz and the cumulative absolute velocity will be calculated based on the recorded motions at the free field instrument. If the operating basis earthquake ground motion is exceeded or significant plant damage occurs, the plant must be shutdown in an orderly manner.

STD COL 3.7-2 In addition, the procedures address measurement of the post-seismic event gaps between the new fuel rack and walls of the new fuel storage pit, between the individual spent fuel racks, and from the spent fuel racks to the spent fuel pool walls, and provide for appropriate corrective actions to be taken if needed (such as repositioning the racks or analysis of the as-found condition).

3.7.4.5 Tests and Inspections

Add the following text to the end of DCD Subsection 3.7.4.5.

STD SUP 3.7-2

Installation and acceptance testing of the triaxial acceleration sensors described in DCD Subsection 3.7.4.2.1 is completed prior to initial startup. Installation and acceptance testing of the time-history analyzer described in DCD Subsection 3.7.4.2.2 is completed prior to initial startup.

3.7.5 COMBINED LICENSE INFORMATION

3.7.5.1 Seismic Analysis of Dams

LNP COL 3.7-1

This COL Item is addressed in Subsection 3.7.2.12.

3.7.5.2 Post-Earthquake Procedures

LNP COL 3.7-2
STD COL 3.7-2

This COL Item is addressed in Subsection 3.7.4.4.

3.7.5.3 Seismic Interaction Review

Replace DCD Subsection 3.7.5.3 with the following text.

STD COL 3.7-3

The seismic interaction review will be updated for as-built information. This review is performed in parallel with the seismic margin evaluation. The review is based on as-procured data, as well as the as-constructed condition. The as-built seismic interaction review is completed prior to fuel load.

3.7.5.4 Reconciliation of Seismic Analyses of Nuclear Island Structures

Replace DCD Subsection 3.7.5.4 with the following text.

STD COL 3.7-4

The seismic analyses described in DCD Subsection 3.7.2 will be reconciled for detailed design changes, such as those due to as-procured or as-built changes in component mass, center of gravity, and support configuration based on as-procured equipment information. Deviations are acceptable based on an evaluation consistent with the methods and procedure of DCD Section 3.7 provided the amplitude of the seismic floor response spectra, including the effect due to these deviations, does not exceed the design basis floor response spectra by more than 10 percent. This reconciliation will be completed prior to fuel load.

3.7.5.5 Free Field Acceleration Sensor

STD COL 3.7-5 This COL Item is addressed in Subsection 3.7.4.2.1.

3.7.6 REFERENCES

Add the following at the end of DCD Subsection 3.7.6:

201. Darendeli, M.B, Development of a New Family of Normalized Modulus Reduction and Material Damping Curves, Ph.D Thesis, University of Texas, Austin, 2001.

202. Menq, F.Y., Dynamic Properties of Sandy and Gravelly Soils, Ph.D Thesis, University of Texas, Austin, 2003.

203. Power, M., B. Chiou, N. Abrahamson, Y. Bozorgnia, T. Shantz, and C. Roblee, An Overview of the NGA Project, Earthquake Spectra, v. 24, p. 3-21, 2008.

LNP SUP 3.2-6
LNP SUP 3.7-3

Table 3.7-201
Lower Lower Bound (LLB) Soil Profile for SSI Analysis

Layer	Layer Thickness (ft.)	D[(a)] (ft.)	Unit Weight (kcf)	BE[(c)] V_s[(b)] (ft/sec)	LB[(d)] V_s[(b)] (ft/sec)	LB[(d)] G[(g)] (ksf)	LLB[(e)] G[(g)] (ksf)	LLB[(e)] - G[(g)] COV[(f)]	Description
1	2.5	2.5	110	836	373	476	476	4.02	Fill
2	2.5	5.0	110	824	342	400	400	4.81	Fill
3	2.5	7.5	110	796	315	339	339	5.38	Fill
4	3.5	11.0	110	788	300	307	307	5.92	Fill
5	2.0	13.0	110	796	301	310	310	5.97	Fill
6	2.0	15.0	110	786	294	296	296	6.13	Fill
7	3.5	18.5	120	1,503	1,123	4,702	4,232	0.99	In -situ Soil
8	2.5	21.0	120	1,500	1,115	4,632	4,169	1.01	In -situ Soil
9	1.0	22.0	120	1,500	1,115	4,632	4,169	1.01	In -situ Soil
10	3.5	25.5	120	1,501	1,074	4,301	3,871	1.17	In -situ Soil
11	3.5	29.0	120	1,496	1,070	4,270	3,843	1.17	In -situ Soil
12	6.7	35.7	120	1,482	1,111	4,596	4,137	0.98	In -situ Soil
13	4.3	40.0	120	1,476	1,100	4,507	4,056	1.00	In -situ Soil
14	2.4	42.4	120	1,476	1,100	4,507	4,056	1.00	In -situ Soil
15	8.3	50.7	130	2,267	1,851	13,830	12,447	0.67	In -situ Soil
16	8.3	59.0	130	2,266	1,850	13,822	12,440	0.67	In -situ Soil
17	7.2	66.2	130	2,254	1,841	13,680	12,312	0.67	In -situ Soil
18	7.2	73.4	130	2,251	1,838	13,639	12,275	0.67	In -situ Soil
19	1.6	75.0	138	2,772	2,264	21,960	19,764	0.67	In -situ Soil
20		> 75.0					Rock[(i)]		Rock

Notes:
a) D: Depth from Design Grade (EL +51 ft.) to bottom of Layer
b) Vs: Layer Shear wave velocity
c) BE: Best Estimate soil profile (Table 17 of Calculation LNG-0000-X7C-044 Rev. 1)
d) LB: Lower Bound soil profile (Table 18 of Calculation LNG-0000-X7C-044 Rev. 1)
e) LLB: Lower Lower Bound soil profile
f) COV: Coefficient of variation
g) G: Shear Modulus
i) Rock profile same as LB rock profile

Units:
ft.: Feet
kcf: Kips per cubic feet
ksf: Kips per square feet

Table 3.7-202
Median Soil Profile to 10^{-5} UHRS Relative Displacements Calculations

LNP COL 2.5-9

Layer	Thickness (ft)	Total Depth (ft)	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)	Damping Ratio (%)	Compression Wave Velocity (ft/sec)	Elevation of Layer Base (ft)
1	2.5	2.5	110	828.7	1.5	1590.2	48.5
2	2.5	5	110	804.6	2.2	1590.2	46.0
3	2.5	7.5	110	761.9	2.9	1590.2	43.5
4	3.5	11	110	744.2	3.5	1590.2	40.0
5	2	13	110	742.7	3.9	5000.0	38.0
6	2	15	110	730.5	4.2	5000.0	36.0
7	3.5	18.5	120	1461.6	3.1	5600.0	32.5
8	2.5	21	120	1454.1	3.3	5600.0	30.0
9	1	22	120	1454.1	3.3	5600.0	29.0
10	3.5	25.5	120	1457.0	2.1	5600.0	25.5
11	3.5	29	120	1442.3	2.2	5600.0	22.0
12	6.9	35.9	120	1434.1	2.1	5600.0	15.1
13	4.1	40	120	1419.4	2.4	5600.0	11.0
14	2.8	42.8	120	1419.4	2.4	5600.0	8.2
15	8.4	51.2	130	2221.9	1.7	7550.0	-0.2
16	8.4	59.6	130	2221.2	1.8	7550.0	-8.6
17	7.1	66.7	130	2206.2	2.0	7550.0	-15.7
18	7.1	73.8	130	2202.1	2.0	7550.0	-22.8
19	1.2	75	138	2768.2	1.4	8700.0	-24.0
20	24.6	99.6	138	2768.2	1.4	8700.0	-48.6
21	47.4	147	138	2685.3	1.4	8550.0	-96.0
22	61.3	208.3	138	3369.3	1.4	10600.0	-157.3
23	17.9	226.2	138	3313.8	1.4	9450.0	-175.2
24	24.1	250.3	120	3204.8	1.8	7250.0	-199.3
25	24.6	274.9	120	3177.0	1.8	7250.0	-223.9
26	40	314.9	120	3522.5	1.3	7900.0	-263.9
27	42	356.9	120	3356.5	1.3	7900.0	-305.9
28	38.4	395.3	140	4130.9	0.9	8900.0	-344.3
29	59.4	454.7	140	3361.0	0.9	8100.0	-403.7
30	59.4	514.1	140	3712.0	0.9	9000.0	-463.1
31	242.7	756.8	140	4537.1	0.9	11000.0	-705.8
32	355.8	1112.6	140	5928.9	0.9	14400.0	-1061.6
33	249.4	1362	150	7276.9	0.7	17850.0	-1311.0
34	252.9	1614.9	150	5087.2	0.7	12350.0	-1563.9
35	148.3	1763.2	150	7277.1	0.7	17400.0	-1712.2
36	106.1	1869.3	150	6240.9	0.7	14900.0	-1818.3
37	199	2068.3	150	7165.6	0.7	17500.0	-2017.3
38	601.2	2669.5	150	5424.6	0.8	13000.0	-2618.5
39	149.2	2818.7	150	5949.2	0.8	14200.0	-2767.7
40	192.7	3011.4	150	6195.7	0.8	14950.0	-2960.4
41	652.3	3663.7	150	5155.8	0.8	12600.0	-3612.7
42	603.7	4267.4	150	5553.3	0.8	13450.0	-4216.4
43	96.6	4364	150	4797.8	0.8	11500.0	-4313.0
44	Halfspace	4364	169	9382.7	0.1	16100.0	-4313.0

Table 3.7-203
Ratio of Horizontal RG 1.60 FIRS and Site Specific (SS) FIRS

LNP SUP 3.7-6
LNP SUP 3.7-3

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

LNP SUP 3.7-6
LNP SUP 3.7-3

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

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

LNP SUP 3.7-6
LNP SUP 3.7-3

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

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

LNP SUP 3.7-5

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

3.8 DESIGN OF CATEGORY I STRUCTURES

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

3.8.3.7 In-Service Testing and Inspection Requirements

Replace the existing DCD statement with the following:

STD COL 3.8-5

The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

3.8.4.7 Testing and In-Service Inspection Requirements

STD COL 3.8-5

Replace the existing DCD final statement of the subsection with the following:

The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

3.8.5.1 Description of the Foundations

Add the following text after paragraph one of DCD Subsection 3.8.5.1.

STD SUP 3.8-1

The depth of overburden and depth of embedment are given in Subsection 2.5.4.

3.8.5.7 In-Service Testing and Inspection Requirements

Replace the existing DCD first statement with the following:

STD COL 3.8-5

The inspection program for structures is identified in Section 17.6. This inspection program is consistent with the requirements of 10 CFR 50.65 and the guidance in Regulatory Guide 1.160.

Add Subsections 3.8.5.9, 3.8.5.10, and 3.8.5.11 following the last paragraph of DCD Subsection 3.8.5.8:

3.8.5.9 Drilled Shaft Foundations Design and Installation

LNP SUP 3.8-2 The Seismic Category II and nonsafety-related adjacent buildings (Turbine Building, Annex Building, and Radwaste Building) are supported on drilled shafts as shown in Figure 3.7-209. The following conceptual drilled shaft design and installation information will be incorporated in the final design drawings and associated construction specifications:

- The conceptual layout and sizes of the drilled shafts used to support the adjacent buildings is shown in Figure 3.7-209. The conceptual design of the drilled shaft socket shows that a 10 ft. socket length is sufficient for current loading provided that the rock has a minimum RQD of 25 percent. The load capacity of the drilled shaft socket is based on the average from the AASHTO and NAVFAC methods. The construction of the drilled shaft foundation for the TB, AB, and RB buildings will consider the measured RQD and the final building loads.

- The rock socket is designed on the basis that the rock surrounding the socket will have a RQD of at least 25 percent over the full depth of the rock socket. The design of the rock socket will take no credit for the rock above the rock socket, having a depth of 2 ft., regardless of the RQD of the rock in this zone. The top of rock, design top of the rock socket, and design bottom of the rock socket will be specified on the construction drawings. A pilot hole will be drilled at the location of each shaft, with core obtained over depth of the expected socket plus at least two socket diameters. If the pilot hole indicates that the RQD does not meet design requirements, the rock socket will be extended to a new design depth based on the core obtained from the pilot holes. The drilled shaft will derive its vertical load carrying capacity entirely from the rock socket. Thus, soil properties will not be measured in the pilot hole program.

- Shaft excavation through the overburden will be performed using a construction methodology designed to minimize the disturbance to the surrounding soils. This may include installing the drilled shafts using either the “dry” or “wet” methods. Advancing the hole through the overburden with air shall be prohibited. A steel casing will be used to maintain the sidewalls of the hole as the drilled shaft is excavated. The steel casing will extend from the ground surface to the top of the rock, and will be “twisted” into rock. The steel casing will be left in place as a permanent feature.

- Rock sockets will be telescoped downward inside the casing with a lip at the top of the socket to allow for seating of the casing. Rock sockets will be cleaned, pumped dry if practicable, and inspected before concrete is placed. The inspection of the rock socket will preferably be through remote visual observations. If this is not practicable, a Shaft Inspection Device (SID) will be used. The time lapse between inspection and cleaning, and concrete placement shall be minimized to limit degradation of the exposed sidewalls and bottom of the excavation.

- The acceptance criteria for the inspecting engineer/geologist shall be as follows:
    - The bottom of the socket shall be free of significant quantities of deleterious material, loose cuttings and muck. If the dry method of construction is specified, the excavation shall be reasonably dry and ready to receive concrete. Pumping shall be used to achieve a reasonably dry socket bottom, if necessary. If it is not practicable to achieve a reasonably dry socket bottom in the judgment of the inspecting engineer/geologist, the contractor may place loose cement at the bottom immediately prior to placing tremie concrete. If the rate of water inflow is excessive in the judgment of the inspecting engineer/geologist, the inspecting engineer/geologist may call for grouting on a case-by-case basis, or wet construction methods for concrete placement will be followed as specified in ACI 336.1-01 and ACI 336.3R-93.
    - For 6 ft. diameter drilled shafts, the exposed side wall rock of the socket will be judged by the inspecting engineer/geologist to have an RQD equal to or greater than 25 percent based on counting fractures, joints and bedding planes on the exposed side wall of the socket. The inspecting engineer/geologist preferably will use remote visual observations to inspect the sidewall of the socket. Field notes and sketches shall be kept by the inspecting engineer/geologist. For 3 ft. and 4 ft. diameter drilled shafts, the minimum RQD determination may be made from the rock core data obtained during the pilot hole program.
- During construction or inspection of the drilled-out socket, if it is determined that, in spite of the core retrieved from the pilot holes drilled in advance of the rock socket drilling, the RQD is not at least 25 percent over the full depth of the socket, the following measures will be taken:
    - If the core from the pilot hole indicates that the RQD is improving with depth and the rock socket design depth can be achieved by drilling the socket deeper to a reasonable depth (approximately one shaft diameter), the design socket depth will be extended.
    - If based on the cuttings from the socket drilling and/or the core from the pilot holes, there is no basis for drilling a deeper socket, then the rock at the base of the socket already drilled will be grouted (mix and design pressure to be determined at the time of construction). For 3- and 4-ft. diameter sockets, the grouting may be achieved with a packer system installed in the pilot hole. For 6-ft. diameter sockets, two or three grout holes will be drilled and grouted over a depth equal to the design depth of socket plus one diameter.

- The contractor shall drill a test drilled shaft to verify the constructability of his proposed casing installation procedure, rebar cage installation procedure and tremie operation. The test shaft will be drilled and tested by geophysical means to assure the integrity of the completed concrete shaft.

3.8.5.10 Construction Sequence of Civil Work

The conceptual design and construction methods of LNP site-specific civil work within and around the nuclear island and the Seismic Category II and nonsafety-related adjacent buildings' footprint are summarized in the following Subsections:

- Diaphragm Walls and Grouting: Subsection 2.5.4.5.1
- Excavation: Subsections 2.5.4.5.2 and 2.5.4.5.3
- Roller Compacted Concrete Bridging Mat: Subsection 2.5.4.5.4
- Construction Dewatering: Subsection 2.5.4.6.2
- Engineering Backfill Properties and Extent: Subsections 2.5.4.5.4 and 3.7.1.1.1
- Vertical and Horizontal Drains for Liquefaction Mitigation: Subsection 2.5.4.8.5
- Drilled Shaft Foundation for Adjacent Buildings: Subsections 2.5.4.5.2 and 3.8.5.9

The design of the excavation and the temporary works necessary for excavation and construction of the Bridging Mat involves construction practices, which if not carried out in a conservative manner, could lead to distress to the excavation and surrounding soils outside the Nuclear Island (NI) excavation. Thus, the design drawings and associated construction specifications will include the following:

- The design and construction of the starter trench, design of the slurry mix, width and thickness of diaphragm wall panels, slurry mixing and conveyance system and procedures and timing of tremie concrete operations to avoid the potential of collapse of the slurry trench walls.
- The choice and availability of backup equipment and stockpile material to deal with trench collapses, excavation delays, and slurry trench maintenance.
- The mixing and conveyance of tremie concrete to the trench excavation, the placement procedure of the tremie concrete (to be pumped under pressure, not gravity fed), placement of the reinforcing steel cage, design and placement of longitudinal reinforcing steel at panel joints, design and

construction of the panel joints to assure water tightness and structural integrity to ensure the integrity of the diaphragm wall.

- The design, installation, testing and monitoring of the anchors to assure adequate capacity, minimum disturbance of the soil outside the diaphragm wall. Anchor holes will be advanced with water or drilling slurry, not air. Directional drilling where interference with drill shafts is possible shall be identified in the design drawings and the construction specifications.

- Design, installation, testing, maintenance and monitoring of shallow dewatering wells to drain the "bathtub," deep wells to relieve uplift pressure on the grouted zone beneath the Bridging Mat excavation, and piezometers to monitor the rate of dewatering and piezometric levels. Iron fouling and bacterial fouling on the well screens and in the pumps will be anticipated and accounted in the design. Replacement pumps will be maintained on site.

- Grouting pressures, grout penetrability, grout mix stability, grout mix constituents and grout mix design must all be designed and specified to prevent hydro-fracturing but adequate penetration and grout-take to effect a watertight grout zone beneath the Bridging Mat. The Grouting Intensity Number (GIN) Methodology for grouting in angled holes shall be specified. This method takes into account the volume of grout injected into the rock as well as the grouting pressure to target the appropriate grout take and penetration while avoiding hydro-fracturing.

- Excavation of the soil within the "bathtub" will be scheduled such as to allow for installation and testing of the anchors in a methodical manner so as not to overstress the diaphragm wall and allow for mapping of the excavation walls as the excavation proceeds downward.

- Excavation, cleanup, mapping and treatment of the rock surface, including dental concrete work, and removal of unsuitable material at the top of rock all shall be done without encroaching on the integrity of the diaphragm wall, anchors, and pressure relief wells.

- Design and construction of the horizontal and vertical drains to relieve excess pore pressures under dynamic conditions shall assure their performance over the life of the plant. Contamination of the drains shall be prevented. Vertical drains will not be made of fabric or paper. Bacterial clogging and iron clogging will be addressed in the design. Limestone shall not be used for the drains.

The civil construction is anticipated to consist of work packages that will be implemented in sequence. However, given the large size of the AP1000 NI footprint, overlapping schedules for the work packages is likely as the civil construction progresses from one area to another. The work packages would consist of the following activities and would generally be implemented in the sequence presented below:

- Site mobilization including erection of RCC Batch plants; build aggregate, ash, and cement stockpiles, mobilize excavation equipment; implement erosion and sedimentation control program; clearing, grubbing and stripping; installation of temporary surface drainage features; implement construction security program; and construct access roads.

- Grouting to form the bottom of the “Bathtub”; installation of shallow dewatering wells; diaphragm wall construction; and construction instrumentation installation.

- Dewatering the “Bathtub”; excavation for the NI foundation and RCC Bridging mat, installation of diaphragm wall anchors with a layout that avoids interference with Turbine Building (TB), Annex Building (AB), and Radwaste Building (RB) drilled shafts, cleaning of diaphragm wall during excavation; side-wall mapping of excavation cuts, mapping and preparation of rock surface for RCC Bridging mat construction.

- RCC Bridging mat construction; construction of the AP1000 basemat, construction of NI structure walls, and backfilling of NI structure walls, all sequenced in accordance with the AP1000 DCD.

- Installation of shallow dewatering system (eductors, well points and/or sumps); excavation to required grades, installation of vertical drains; installation of horizontal drains; and drilling of pilot holes for the drilled shafts for the TB.

- Installation of the drilled shafts for the TB.

- Clean-up and final grade excavation, construction of the shaft caps, and construction of the TB foundations and below grade walls.

- Grade all building areas per grading plan; installation of drilled shafts for the AB and RB, construction of AB and RB drilled shaft caps, construction of AB and RB foundations; and installation of site drainage system.

The final determination of the activities to be included in each specific work package will be determined by the contractor prior to the start of construction. However, the sequence of when each package will be implemented will generally follow the sequence specified above.

3.8.5.11 Roller Compacted Concrete Strength and Constructability Verification Program

LNP SUP 3.8-3 A Roller Compacted Concrete (RCC) bridging mat will support the LNP Nuclear Island (NI) foundation as described in Subsections 2.5.4.5.4 and 2.5.4.12. This subsection describes the RCC strength and constructability verification program for LNP that was completed and that is planned post-COL.

3.8.5.11.1 Experience from Large Scale Commercial RCC Projects

LNP RCC construction will follow industry standard methods that have been successfully been implemented on large commercial RCC projects. This provides assurance that LNP RCC bridging mat can be successfully constructed and will have the desired strength.

United States Army Corp of Engineers Engineering Manual EM 1110-2-2006 (USACE EM 1110-2-2006) describes standard equipment and practices that are used during RCC construction. These practices include guidance for developing RCC mixes, procedures for RCC placement and compaction, and for lift surface preparation. The LNP RCC construction specifications will specify RCC mixing, placement, and compaction equipment, as well as procedures associated with each to be consistent with USACE EM 1110-2-2006 guidelines and incorporate practices from the successful commercial projects. The LNP RCC construction specifications will also specify additional requirements for nuclear safety grade Quality Assurance.

Reference 201 compares the RCC mixes, aggregates, cement, and fly ash from three large commercial RCC projects to those planned to be used for LNP RCC bridging mat. The report concludes that the properties of the aggregates, cement, and fly ash planned for LNP will meet or exceed the requirements used for these successful commercial projects.

Quality control and inspection during production construction, as described in Subsection 2.5.4.12 and Reference 202, “Post-COL Roller Compacted Concrete Test Plan Levy Nuclear Plant,” Revision 3, will ensure that the mixing, placement, and compaction of production RCC complies with the LNP RCC construction specifications.

The report “Previous Commercial Testing Results Levy Nuclear Plant,” Revision 0, (Reference 203) describes the RCC testing results from three large commercial RCC projects. The following can be concluded from these commercial testing results:

- The compressive strengths measured during production construction exceeded those that were measured during pre-construction mix design laboratory testing. Thus, laboratory testing during RCC mix design provides reasonable assurance that the desired RCC compressive strength will be achieved or exceeded during production construction.

- The measured modulus of elasticity from commercial testing correlates well with that computed using ACI 318-99 section 8.5.1 method. Thus use of ACI 318-99 equation for modulus of elasticity in RCC design is appropriate.

- The USACE EM 1110-2-2006 correlation that the direct tensile strength of RCC is approximately 75 percent of the split tensile strength trends close to the ACI 318-99 equation 22-2 for tensile strength. Thus, the use of ACI 318-99 equation 22-2, for tensile strength in RCC design, is appropriate.

- Shear tests performed on pre-cracked (at lift joints) block samples show that the friction angle when concrete bedding mix is used is greater than the 45 degrees design value provided in USACE EM 1110-2-206. Thus, the use of 45 degrees friction angle for shear capacity in RCC design across lift joints is appropriate.

3.8.5.11.2 LNP Pre-COL RCC Testing

For design, RCC nominal strength capacities were established using ACI 349 and ACI 318 equations and USACE EM 1110-2-206 guidance. The Finite Element Model (FEM) of the RCC Bridging Mat has confirmed that these capacities are adequate for the anticipated loading conditions and postulated conservative karst sizes and configurations as described in Subsection 2.5.4.5.4.

The LNP RCC Mix Design program is described in “56-Day Report Phase II Mix Design Program Levy Nuclear Plant,” Revision 1 (Reference 204). The RCC mix design program, sixteen trial RCC mixes were tested and all yielded RCC compressive strengths greater than the 2500 psi used in the FEM analysis. Five trial bedding mixes were tested and all yielded a compressive strength of greater than 4000 psi. The RCC mix and bedding mix design program evaluated the effects of water-cementitious material ratio, fly ash replacement percentage, fly ash sources, and aggregate sources with respect to strength and workability. This mix design program demonstrated that design workability and strength requirements can be achieved with the trial mixes and constituent materials procured for the program. The program concluded with the selection of a single RCC mix and a bedding mix that is workable, and meets design compressive strength while minimizing the cement content for favorable thermal characteristics. The results of the mix design program will be used to develop the LNP RCC construction specification for RCC constituents mix proportions and properties of the constituent materials.

For the selected LNP design RCC mix, laboratory testing was performed as described in “90-day Report Phase III Specialty Testing Program Levy Nuclear Plant,” Revision 0, (Reference 205). In this program RCC test cylinders and three RCC test panels measuring approximately 7 ft. x 7 ft. x 2 ft. were cast. The RCC panels were cast in two layers, with bedding mix between the two layers. The RCC constitutive materials used for this phase were from the same sources as for the RCC Mix design program. Test panels with Joint Maturity Values (JMV) of approximately 2500 Degree Hours (“warm” joint) and with JMV of approximately

4700 Degree Hours ("cold" joint) were constructed. In the panel with the "cold joint", layer 1 was green cut prior to placing the bedding mix and the second layer of RCC.

The compressive and split tensile strength test results from laboratory cast cylinders in this program were consistent with past RCC experience and validated use of ACI 318-99 equations for tensile strength and modulus of elasticity. The tests also verified that the selected LNP RCC mix yields compressive strength greater than the specified 2500 psi and split cylinder tensile strength consistent with ACI 318-99 correlations. However, preliminary testing on cored cylinders from the test panels indicated that the concrete in the test panels did not attain the desired compressive or tensile strengths. This low strength is believed to be due to the constructability issues related to construction of the laboratory-scale test panels that required the use of small mixing and compaction equipment. As stated before, LNP production RCC construction will use mixing, placement, and compaction equipment consistent with USACE EM 1110-2-2006 guidance and comparable to that used in large successful commercial projects.

Three block shear samples cut from the test panels were tested. These bi-axial shear tests yielded shear strengths at least 1.67 times the maximum design demand shear across lift joints even though the test panels did not achieve the desired compressive strength at 90-days.

3.8.5.11.3 LNP Post-COL RCC Testing

Post-COL RCC and bedding mix strength verification and constructability testing will be performed on a large test pad as described in Phase IV testing of Reference 202, "Post-COL Roller Compacted Concrete Test Plan Levy Nuclear Plant," Revision 3. This testing is being performed post-COL but prior to construction of the LNP bridging mat for the following reasons:

- Due to the limitation on mixing and compaction equipment sizes that can be used in a laboratory setting, the required compaction cannot be achieved in a laboratory setting. A larger scale test pad in an open field setting is required.

- Because RCC design strength is specified as the 365-day strength, it is not practical to perform destructive testing on the RCC bridging mat during construction on cored or block cut test specimens.

RCC strength verification and constructability testing will be performed post-COL at the LNP site. The post-COL RCC strength and constructability testing will verify that the specified RCC compressive strength, ACI 318 specified tensile strength, and USACE EM 1110-2-2006 specified shear strengths across lift joints can be achieved. A RCC test pad measuring approximately 42 ft. x 40 ft. x 6 ft. will be constructed with the specified RCC and bedding mixes. The test pad construction will use mixing, placement, and compaction procedures and equipment comparable to those that will be used during LNP RCC bridging mat construction. The constitutive materials for the RCC mix will be comparable to that used in the RCC mix design program (Reference 204). Six inch cores from the RCC test pad will be used to verify that the design compressive strength is achieved and the split cylinder strength meets ACI 318-99 requirements. Blocks cut from the RCC test pad similar in size to those used in the pre-COL specialty testing program will be used to verify that USACE EM 1110-2-2006 shear strength are achieved across lift joints. Shear test specimens with Joint Maturity Values (JMV) of approximately 2500 Degree Hours ("warm" joint) and with JMV of approximately 4700 Degree Hours ("cold" joint) will be tested. Thermocouples or thermistors will be used to monitor JMV. "Cold" joints will be green cut prior to placement of the subsequent lift.

The geometry and loading on the LNP RCC bridging mat is such that there is no tension across lift joints. Thus, no tensile strength tests across lift joints will be performed.

The post-COL strength verification and constructability test report with 90-day test results will be completed at least 180-days prior to start of LNP RCC bridging mat construction.

3.8.5.11.4 LNP RCC Testing During Production Construction

The production RCC bridging mat will not be cut for testing. Testing of the production mat will be confirmatory, using non destructive testing methods to ensure that the construction of the RCC and bedding joints is in accordance with the RCC construction specifications. The report "Post-COL Roller Compacted Concrete Test Plan Levy Nuclear Plant," Revision 3, (Reference 202) describes the testing that will occur during construction of the RCC bridging mat, including quality control testing. According to USACE EM 1110-2-2006, the characteristics required to obtain good RCC and bond strength at the lift joint include good-quality aggregate, good mixture workability and compaction effort, rapid covering of lift joints by subsequent lifts, and the use of bedding mix. These items will be addressed in the RCC construction specifications and construction Quality Control program for RCC bridging mat construction as follows:

The quality of the aggregate, cement, and fly ash from multiple sources was evaluated during the RCC mix design program. The RCC construction specifications will specify aggregate, cement, and fly ash sources and quality requirements comparable to those used for the LNP RCC mix design program (Reference 204). Coarse aggregate complying with ASTM C33 will be used. Type II cement complying with ASTM C 150 and ASTM C 186 and Class F fly ash complying with ASTM C 618 requirements will be used. To ensure the quality and uniformity of the RCC during production, the aggregate will be tested daily for conformance to construction specifications for gradation and moisture content. Monthly tests of each aggregate during construction will verify that it continues to meet requirements for specific gravity, organic impurities, and LA Abrasion.

RCC mix workability will be measured by Vebe testing for RCC and slump testing for bedding mix. The selected RCC and bedding mix from the RCC Mix Design program had acceptable workability. During construction, Vebe time for RCC and slump of bedding mix will be measured at least once per shift to monitor the workability of the mixes. Other properties, such as the temperature of the RCC at the point of placement, and the air content of the RCC will also be monitored. During RCC construction, thermocouples or thermistors will be used to monitor Joint Maturity Value (JMV).

The RCC construction specifications, non destructive testing and quality controls during construction together with implementing procedures and equipment comparable to those used on past successful RCC projects, pre-COL RCC mix design testing, the pre-COL RCC testing, and the planned post-COL RCC testing using a large test pad provides sufficient assurance that the LNP design compressive and tensile strengths, and shear strengths across lift joints will be achieved during the RCC bridging mat construction using the RCC and bedding mix, mixing and placement procedures and equipment, and the compaction equipment specified for construction.

3.8.6.5 Structures Inspection Program

STD COL 3.8-5 This item is addressed in Subsections 3.8.3.7, 3.8.4.7, 3.8.5.7, and 17.6.

3.8.6.6 Construction Procedures Program

Add the following to the end of Subsection 3.8.6.6:

STD COL 3.8-6 Construction and inspection procedures for concrete filled steel plate modules address activities before and after concrete placement, use of construction mock-ups, and inspection of modules before and after concrete placement as discussed in DCD Subsection 3.8.4.8. The procedures will be made available to NRC inspectors prior to use.

3.8.7 REFERENCES

Add the following information at the end of DCD Subsection 3.8.7:

201. Paul C. Rizzo Associates, “Previous Commercial RCC Experience Levy Nuclear Plant,” Revision 1, May 2011.

202. Paul C. Rizzo Associates, “Post-COL Roller Compacted Concrete Test Plan Levy Nuclear Plant,” Revision 3, May 2011.

203. Paul C. Rizzo Associates, "Previous Commercial RCC Testing Results Levy Nuclear Plant," Revision 0, October 2010.

204. Paul C. Rizzo Associates, "56-Day Report Phase II Mix Design Program Levy Nuclear Plant," Revision 1, April 2011.

205. Paul C. Rizzo Associates, "90-Day Report Phase III Specialty Testing Program Levy Nuclear Plant," Revision 0, May 2011.

3.9 MECHANICAL SYSTEMS AND COMPONENTS

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

3.9.3.1.2 Loads for Class 1 Components, Core Support, and Component Supports

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Add the following after the last paragraph under DCD subheading Request 3) and prior to DCD subheading Other Applications.

STD COL 3.9-5 PRESSURIZER SURGE LINE MONITORING

General

The pressurizer surge line is monitored at the first AP1000 plant to record temperature distributions and thermal displacements of the surge line piping, as well as pertinent plant parameters. This monitoring occurs during the hot functional testing and first fuel cycle. The resulting monitoring data is evaluated to verify that the pressurizer surge line is within the bounds of the analytical temperature distributions and displacements.

Subsequent AP1000 plants (after the first AP1000 plant) confirm that the heatup and cooldown procedures are consistent with the pertinent attributes of the first AP1000 plant surge line monitoring. In addition, changes to the heatup and cooldown procedures consider the potential impact on stress and fatigue analyses consistent with the concerns of NRC Bulletin 88-11.

The pressurizer surge line monitoring activities include the following methodology and requirements:

Monitoring Method

The pressurizer surge line pipe wall is instrumented with outside mounted temperature and displacement sensors. The data from this instrumentation is supplemented by plant computer data from related process and control parameters.

Locations to be Monitored

In addition to the existing permanent plant temperature instrumentation, temperature and displacement monitoring will be included at critical locations on the surge line. The additional locations utilized for monitoring during the hot functional testing and the first fuel cycle (see Subsection 14.2.9.2.22) are selected based on the capability to provide effective monitoring.

Data Evaluation

Data evaluation is performed at the completion of the monitoring period (one fuel cycle). The evaluation includes a comparison of the data evaluation results with the thermal profiles and transient loadings defined for the pressurizer surge line, accounting for expected pipe outside wall temperatures. Interim evaluations of the data are performed during the hot functional testing period, up to the start of normal power operation, and again once three months worth of normal operating data has been collected, to identify any unexpected conditions in the pressurizer surge line.

3.9.3.4.4 Inspection, Testing, Repair, and/or Replacement of Snubbers

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Add the following text after the last paragraph of DCD Subsection 3.9.3.4.4:

STD SUP 3.9-3 a. Snubber Design and Testing

1. A list of snubbers on systems which experience sufficient thermal movement to measure cold to hot position is included in Table 3.9-201.

2. The snubbers are tested to verify they can perform as required during the seismic events, and under anticipated operational transient loads or other mechanical loads associated with the design requirements for the plant. Production and qualification test programs for both hydraulic and mechanical snubbers are carried out by the snubber vendors in accordance with the snubber installation instruction manual required to be furnished by the snubber supplier. Acceptance criteria for compliance with ASME Section III Subsection NF, and other applicable codes, standards, and requirements, are as follows:

   - Snubber production and qualification test programs are carried out by strict adherence to the manufacturer's snubber installation and instruction manual. This manual is prepared by the snubber manufacturer and subjected to review for compliance with the applicable provisions of the ASME Pressure Vessel and Piping Code of record. The test program is periodically audited during implementation for compliance.

   - Snubbers are inspected and tested for compliance with the design drawings and functional requirements of the procurement specifications.

   - Snubbers are inspected and qualification tested. No sampling methods are used in the qualification tests.

- Snubbers are load rated by testing in accordance with the snubber manufacturer's testing program and in compliance with the applicable sections of ASME QME-1-2007, Subsection QDR and the ASME Code for Operation and Maintenance of Nuclear Power Plants (OM Code), Subsection ISTD.

- Design compliance of the snubbers per ASME Section III Paragraph NF-3128, and Subparagraphs NF-3411.3 and NF-3412.4.

- The snubbers are tested for various abnormal environmental conditions. Upon completion of the abnormal environmental transient test, the snubber is tested dynamically at a frequency within a specified frequency range. The snubber must operate normally during the dynamic test. The functional parameters cited in Subparagraph NF-3412.4 are included in the snubber qualification and testing program. Other parameters in accordance with applicable ASME QME-1-2007 and the ASME OM Code will be incorporated.

- The codes and standards used for snubber qualification and production testing are as follows:

  – ASME B&PV Code Section III (Code of Record date) and Subsection NF.

  – ASME QME-1-2007, Subsection QDR and ASME OM Code, Subsection ISTD.

- Large bore hydraulic snubbers are full Service Level D load tested, including verifying bleed rates, control valve closure within the specified velocity ranges and drag forces/breakaway forces are acceptable in accordance with ASME, QME-1-2007 and ASME OM Codes.

3. Safety-related snubbers are identified in Table 3.9-201, including the snubber identification and the associated system or component, e.g., line number. The snubbers on the list are hydraulic and constructed to ASME Section III, Subsection NF. The snubbers are used for shock loading only. None of the snubbers are dual purpose or vibration arrestor type snubbers.

b. Snubber Installation Requirements

Installation instructions contain instructions for storage, handling, erection, and adjustments (if necessary) of snubbers. Each snubber has an installation location drawing that contains the installation location of the snubber on the pipe and structure, the hot and cold settings, and additional information needed to install the particular snubber.

STD COL 3.9-3 The description of the snubber preservice and inservice testing programs in this section is based on the ASME OM Code 2001 Edition through 2003 Addenda. The initial inservice testing program incorporates the latest edition and addenda of the ASME OM Code approved in 10 CFR 50.55a(f) on the date 12 months before initial fuel load. Limitations and modifications set forth in 10 CFR 50.55a are incorporated.

c. Snubber Preservice Examination and Testing

The preservice examination plan for applicable snubbers is prepared in accordance with the requirements of the ASME Code for Operation and Maintenance of Nuclear Power Plants (OM Code), Subsection ISTD, and the additional requirements of this Section. This examination is made after snubber installation but not more than 6 months prior to initial system preoperational testing. The preservice examination verifies the following:

1. There are no visible signs of damage or impaired operational readiness as a result of storage, handling, or installation.

2. The snubber load rating, location, orientation, position setting, and configuration (attachments, extensions, etc.) are according to design drawings and specifications.

3. Snubbers are not seized, frozen or jammed.

4. Adequate swing clearance is provided to allow snubber movements.

5. If applicable, fluid is to the recommended level and is not to be leaking from the snubber system.

6. Structural connections such as pins, fasteners and other connecting hardware such as lock nuts, tabs, wire, cotter pins are installed correctly.

If the period between the initial preservice examination and initial system preoperational tests exceeds 6 months, reexamination of Items 1, 4, and 5 is performed. Snubbers, which are installed incorrectly or otherwise fail to meet the above requirements, are repaired or replaced and re-examined in accordance with the above criteria.

A preservice thermal movement examination is also performed, during initial system heatup and cooldown. For systems whose design operating temperature exceeds 250ºF (121ºC), snubber thermal movement is verified.

Additionally, preservice operational readiness testing is performed on snubbers. The operational readiness test is performed to verify the parameters of ISTD 5120. Snubbers that fail the preservice operational readiness test are evaluated to determine the cause of failure, and are retested following completion of corrective action(s).

Snubbers that are installed incorrectly or otherwise fail preservice testing requirements are re-installed correctly, adjusted, modified, repaired or replaced, as required. Preservice examination and testing is re-performed on installation-corrected, adjusted, modified, repaired or replaced snubbers as required.

d. Snubber Inservice Examination and Testing

Inservice examination and testing of safety-related snubbers is conducted in accordance with the requirements of the ASME OM Code, Subsection ISTD. Inservice examination is initially performed not less than two months after attaining 5 percent reactor power operation and is completed within 12 calendar months after attaining 5 percent reactor power. Subsequent examinations are performed at intervals defined by ISTD-4252 and Table ISTD-4252-1. Examination intervals, subsequent to the third interval, are adjusted based on the number of unacceptable snubbers identified in the current interval.

An inservice visual examination is performed on the snubbers to identify physical damage, leakage, corrosion, degradation, indication of binding, misalignment or deformation and potential defects generic to a particular design. Snubbers that do not meet visual examination requirements are evaluated to determine the root cause of the unacceptability, and appropriate corrective actions (e.g., snubber is adjusted, repaired, modified, or replaced) are taken. Snubbers evaluated as unacceptable during visual examination may be accepted for continued service by successful completion of an operational readiness test.

Snubbers are tested inservice to determine operational readiness during each fuel cycle, beginning no sooner than 60 days before the start of the refueling outage. Snubber operational readiness tests are conducted with the snubber in the as-found condition, to the extent practical, either in-place or on a test bench, to verify the test parameters of ISTD-5210. When an in-place test or bench test cannot be performed, snubber subcomponents that control the parameters to be verified are examined and tested. Preservice examinations are performed on snubbers after reinstallation when bench testing is used (ISTD-5224), or on snubbers where individual subcomponents are reinstalled after examination (ISTD-5225).

Defined test plan groups (DTPG) are established and the snubbers of each DTPG are tested according to an established sampling plan each fuel cycle. Sample plan size and composition is determined as required for the selected sample plan, with additional sampling as may be required for that sample plan based on test failures and failure modes identified. Snubbers that do not meet test requirements are evaluated to determine root cause of the failure, and are assigned to failure mode groups (FMG) based on the evaluation, unless the failure is considered unexplained or isolated. The number of unexplained snubber failures, not assigned to a FMG, determines the additional testing sample. Isolated failures do not require additional testing. For unacceptable snubbers, additional testing is conducted for the DTPG or FMG until the appropriate sample plan completion criteria are satisfied.

Unacceptable snubbers are adjusted, repaired, modified, or replaced. Replacement snubbers meet the requirements of ISTD-1600. Post-maintenance examination and testing, and examination and testing of repaired snubbers, is done to verify as acceptable the test parameters that may have been affected by the repair or maintenance activity.

Service life for snubbers is established, monitored and adjusted as required by ISTD-6000 and the guidance of ASME OM Code Nonmandatory Appendix F.

3.9.6 INSERVICE TESTING OF PUMPS AND VALVES

Revise the third sentence of the third paragraph of DCD Subsection 3.9.6, and add information between the third and fourth sentences as follows:

STD COL 3.9-4

The edition and addenda to be used for the inservice testing program are administratively controlled; the description of the inservice testing program in this section is based on the ASME OM Code 2001 Edition through 2003 Addenda. The initial inservice testing program incorporates the latest edition and addenda of the ASME OM Code approved in 10 CFR 50.55a(f) on the date 12 months before initial fuel load. Limitations and modifications set forth in 10 CFR 50.55a are incorporated.

Revise the fifth sentence of the sixth paragraph of DCD Subsection 3.9.6 as follows:

STD COL 3.9-4

Alternate means of performing these tests and inspections that provide equivalent demonstration may be developed in the inservice test program ~~as described in subsection 3.9.8~~.

Revise the first two sentences of the final paragraph of DCD Subsection 3.9.6 to read as follows:

STD COL 3.9-4

A preservice test program, which identifies the required functional testing, is to be submitted to the NRC prior to performing the tests and following the start of construction. The inservice test program, which identifies requirements for functional testing, is to be submitted to the NRC prior to the anticipated date of commercial operation as described above.

Add the following text after the last paragraph of DCD Subsection 3.9.6:

Table 13.4-201 provides milestones for preservice and inservice test program implementation.

3.9.6.2.2 Valve Testing

Add the following prior to the initial paragraph of DCD Subsection 3.9.6.2.2:

STD COL 3.9-4

Valve testing uses reference values determined from the results of preservice testing or inservice testing. These tests that establish reference and IST values are performed under conditions as near as practicable to those expected during the IST. Reference values are established only when a valve is known to be operating acceptably.

Pre-conditioning of valves or their associated actuators or controls prior to IST testing undermines the purpose of IST testing and is not allowed. Pre-conditioning includes manipulation, pre-testing, maintenance, lubrication, cleaning, exercising, stroking, operating, or disturbing the valve to be tested in any way, except as may occur in an unscheduled, unplanned, and unanticipated manner during normal operation.

Add the following sentence to the end of the fourth paragraph under the heading “Manual/Power-Operated Valve Tests”:

STD COL 3.9-4

Stroke time is measured and compared to the reference value, except for valves classified as fast-acting (e.g., solenoid-operated valves with stroke time less than 2 seconds), for which a stroke time limit of 2 seconds is assigned.

Add the following paragraph after the fifth paragraph under the heading “Manual/Power-Operated Valve Tests”:

STD COL 3.9-4

During valve exercise tests, the necessary valve obturator movement is verified while observing an appropriate direct indicator, such as indicating lights that signal the required changes of obturator position, or by observing other evidence or positive means, such as changes in system pressure, flow, level, or temperature that reflects change of obturator position.

Insert new second sentence of the paragraph containing the subheading "Power-Operated Valve Operability Tests" in DCD Subsection 3.9.6.2.2 (immediately following the first sentence of the DCD paragraph) to read:

STD COL 3.9-4 The POVs include the motor-operated valves.

Add the following sentence as the last sentence of the paragraph containing the subheading "Power-Operated Valve Operability Tests" in DCD Subsection 3.9.6.2.2:

STD COL 3.9-4 Table 13.4-201 provides milestones for the MOV program implementation.

Insert the following as the last sentence in the paragraph under the bulleted item titled "Risk Ranking" in DCD Subsection 3.9.6.2.2:

STD COL 3.9-4 Guidance for this process is outlined in the JOG MOV PV Study, MPR-2524-A.

Insert the following text after the last paragraph under the sub-heading of "Power-Operated Valve Operability Tests" and before the sub-heading "Check Valve Tests" in DCD Subsection 3.9.6.2.2:

STD COL 3.9-4 **Active MOV Test Frequency Determination** - The ability of a valve to meet its design basis functional requirements (i.e. required capability) is verified during valve qualification testing as required by procurement specifications. Valve qualification testing measures valve actuator actual output capability. The actuator output capability is compared to the valve's required capability defined in procurement specifications, establishing functional margin; that is, that increment by which the MOV's actual output capability exceeds the capability required to operate the MOV under design basis conditions. DCD Subsection 5.4.8 discusses valve functional design and qualification requirements. The initial inservice test frequency is determined as required by ASME OM Code Case OMN-1, Revision 1 (Reference 202). The design basis capability testing of MOVs utilizes guidance from Generic Letter 96-05 and the JOG MOV Periodic Verification PV Program. Valve functional margin is evaluated following subsequent periodic testing to address potential time-related performance degradation, accounting for applicable uncertainties in the analysis. If the evaluation shows that the functional margin will be reduced to less than established acceptance criteria within the established test interval, the test interval is decreased to less than the time for the functional margin to decrease below acceptance criteria. If there is not sufficient data to determine test frequency as described above, the test frequency is limited to not exceed two (2) refueling cycles or three (3) years, whichever is longer, until sufficient data exist to extend the test frequency. Appropriate justification is provided for any increased test interval, and the maximum test interval shall not exceed 10 years. This is to ensure that each MOV in the IST program will have adequate margin (including consideration for aging-related degradation, degraded voltage, control switch repeatability, and load-sensitive MOV behavior) to remain operable until the next scheduled test, regardless of its risk categorization or safety significance. Uncertainties associated with performance of these periodic verification tests and use of the test results (including those associated with measurement equipment and potential degradation mechanisms) are addressed appropriately. Uncertainties may be considered in the specification of acceptable valve setup parameters or in the interpretation of the test results (or a combination of both). Uncertainties affecting both valve function and structural limits are addressed.

Maximum torque and/or thrust (as applicable) achieved by the MOV (allowing sufficient margin for diagnostic equipment inaccuracies and control switch repeatability) are established so as not to exceed the allowable structural and undervoltage motor capability limits for the individual parts of the MOV.

Solenoid-operated valves (SOVs) are tested to confirm the valve moves to its energized position and is maintained in that position, and to confirm that the valve moves to the appropriate failure mode position when de-energized.

Other Power-Operated Valve Operability Tests – Power-Operated valves other than active MOVs are exercised quarterly in accordance with ASME OM ISTC, unless justification is provided in the inservice testing program for testing these valves at other than Code mandated frequencies.

Although the design basis capability of power-operated valves is verified as part of the design and qualification process, power-operated valves that perform an active safety function are tested again after installation in the plant, as required, to ensure valve setup is acceptable to perform their required functions, consistent with valve qualification. These tests, which are typically performed under static (no flow or pressure) conditions, also document the “baseline” performance of the valves to support maintenance and trending programs. During the testing, critical parameters needed to ensure proper valve setup are measured. Depending on the valve and actuator type, these parameters may include seat load, running torque or thrust, valve travel, actuator spring rate, bench set and regulator supply pressure. Uncertainties associated with performance of these tests and use of the test results (including those associated with measurement equipment and potential degradation mechanisms) are addressed appropriately. Uncertainties may be considered in the specification of acceptable valve setup parameters or in the interpretation of the test results (or a combination of both). Uncertainties affecting both valve function and structural limits are addressed.

Additional testing is performed as part of the air-operated valve (AOV) program, which includes the key elements for an AOV Program as identified in the JOG AOV program document, Joint Owners Group Air Operated Valve Program Document, Revision 1, December 13, 2000 (References 203 and 204). The AOV program incorporates the attributes for a successful power-operated valve long-term periodic verification program, as discussed in Regulatory Issue Summary 2000-03, Resolution of Generic Safety Issue 158: Performance of Safety-Related Power-Operated Valves Under Design Basis Conditions, by incorporating lessons learned from previous nuclear power plant operations and research programs as they apply to the periodic testing of air- and other power-operated valves included in the IST program. For example, key lessons learned addressed in the AOV program include:

- Valves are categorized according to their safety significance and risk ranking.

- Setpoints for AOVs are defined based on current vendor information or valve qualification diagnostic testing, such that the valve is capable of performing its design-basis function(s).

- Periodic static testing is performed, at a minimum on high risk (high safety significance) valves, to identify potential degradation, unless those valves are periodically cycled during normal plant operation, under conditions that meet or exceed the worst case operating conditions within the licensing basis of the plant for the valve, which would provide adequate periodic demonstration of AOV capability. If required based on valve qualification or operating experience, periodic dynamic testing is performed to re-verify the capability of the valve to perform its required functions.

- Sufficient diagnostics are used to collect relevant data (e.g., valve stem thrust and torque, fluid pressure and temperature, stroke time, operating and/or control air pressure, etc.) to verify the valve meets the functional requirements of the qualification specification.

- Test frequency is specified, and is evaluated each refueling outage based on data trends as a result of testing. Frequency for periodic testing is in accordance with References 203 and 204, with a minimum of 5 years (or 3 refueling cycles) of data collected and evaluated before extending test intervals.

- Post-maintenance procedures include appropriate instructions and criteria to ensure baseline testing is re-performed as necessary when maintenance on the valve, repair or replacement, have the potential to affect valve functional performance.

- Guidance is included to address lessons learned from other valve programs specific to the AOV program.

- Documentation from AOV testing, including maintenance records and records from the corrective action program are retained and periodically evaluated as a part of the AOV program.

Insert the following paragraph as the last paragraph under the sub-heading of "Power-Operated Valve Operability Tests" (following the previously added paragraph) and just before the sub-heading "Check Valve Tests" in DCD Subsection 3.9.6.2.2.

STD COL 3.9-4

Successful completion of the preservice and IST of MOVs, in addition to MOV testing as required by 10 CFR 50.55a, demonstrates that the following criteria are met for each valve tested: (i) valve fully opens and/or closes as required by its safety function; (ii) adequate margin exists and includes consideration of diagnostic equipment inaccuracies, degraded voltage, control switch repeatability, load-sensitive MOV behavior, and a margin for degradation; and (iii) maximum torque and/or thrust (as applicable) achieved by the MOV (allowing sufficient margin for diagnostic equipment inaccuracies and control switch repeatability) does not exceed the allowable structural and undervoltage motor capability limits for the individual parts of the MOV.

Add the paragraph below as the last paragraph of FSAR Subsection 3.9.6.2.2 prior to the subheading "Check Valve Tests":

STD COL 3.9-4

The attributes of the AOV testing program described above, to the extent that they apply to and can be implemented on other safety-related power-operated valves, such as electro-hydraulic valves, are applied to those other power-operated valves.

Add the following new paragraph under the heading "Check Valve Tests" in DCD Subsection 3.9.6.2.2:

STD COL 3.9-4

Preoperational testing is performed during the initial test program (refer to DCD Subsection 14.2) to verify that valves are installed in a configuration that allows correct operation, testing, and maintenance. Preoperational testing verifies that piping design features accommodate check valve testing requirements. Tests also verify disk movement to and from the seat and determine, without disassembly, that the valve disk positions correctly, fully opens or fully closes as expected, and remains stable in the open position under the full spectrum of system design-basis fluid flow conditions.

Add the following new last paragraphs under the subheading "Check Valve Exercise Tests" in DCD Subsection 3.9.6.2.2

STD COL 3.9-4

Acceptance criteria for this testing consider the specific system design and valve application. For example, a valve's safety function may require obturator movement in both open and closed directions. A mechanical exerciser may be used to operate a check valve for testing. Where a mechanical exerciser is used, acceptance criteria are provided for the force or torque required to move the check valve's obturator. Exercise tests also detect missing, sticking, or binding obturators.

When operating conditions, valve design, valve location, or other considerations prevent direct observation or measurements by use of conventional methods to determine adequate check valve function, diagnostic equipment and nonintrusive techniques are used to monitor internal conditions. Nonintrusive tests used are dependent on system and valve configuration, valve design and materials, and include methods such as ultrasonic (acoustic), magnetic, radiography, and use of accelerometers to measure system and valve operating parameters (e.g., fluid flow, disk position, disk movement, disk impact, and the presence or absence of cavitation and back-tapping). Nonintrusive techniques also detect valve degradation. Diagnostic equipment and techniques used for valve operability determinations are verified as effective and accurate under the PST program.

Testing is performed, to the extent practicable, under normal operation, cold shutdown, or refueling conditions applicable to each check valve. Testing includes effects created by sudden starting and stopping of pumps, if applicable, or other conditions, such as flow reversal. When maintenance that could affect valve performance is performed on a valve in the IST program, post-maintenance testing is conducted prior to returning the valve to service.

Add the following new paragraph under the heading "Other Valve Inservice Tests" following the Explosively Actuated Valves paragraph in DCD Subsection 3.9.6.2.2:

STD COL 3.9-4 Industry and regulatory guidance is considered in development of IST program for squib valves. In addition, the IST program for squib valves incorporates lessons learned from the design and qualification process for these valves such that surveillance activities provide reasonable assurance of the operational readiness of squib valves to perform their safety functions.

3.9.6.2.3 Valve Disassembly and Inspection

Add the following paragraph as the new second paragraph of DCD Subsection 3.9.6.2.3:

STD COL 3.9-4 During the disassembly process, the full-stroke motion of the obturator is verified. Nondestructive examination is performed on the hinge pin to assess wear, and seat contact surfaces are examined to verify adequate contact. Full-stroke motion of the obturator is re-verified immediately prior to completing reassembly. At least one valve from each group is disassembled and examined at each refueling outage, and all the valves in each group are disassembled and examined at least once every eight years. Before being returned to service, valves disassembled for examination or valves that received maintenance that could affect their performance are exercised with a full- or part-stroke. Details and bases of the sampling program are documented and recorded in the test plan.

Add Subsections 3.9.6.2.4 and 3.9.6.2.5 following the last paragraph of DCD Subsection 3.9.6.2.3:

STD COL 3.9-4

3.9.6.2.4 Valve Preservice Tests

Each valve subject to inservice testing is also tested during the preservice test period. Preservice tests are conducted under conditions as near as practicable to those expected during subsequent inservice testing. Valves (or the control system) that have undergone maintenance that could affect performance, and valves that have been repaired or replaced, are re-tested to verify performance parameters that could have been affected are within acceptable limits. Safety and relief valves and nonreclosing pressure relief devices are preservice tested in accordance with the requirements of the ASME OM Code, Mandatory Appendix I.

Preservice tests for valves are performed in accordance with ASME OM, ISTC-3100.

3.9.6.2.5 Valve Replacement, Repair, and Maintenance

Testing in accordance with ASME OM, ISTC-3310 is performed after a valve is replaced, repaired, or undergoes maintenance. When a valve or its control system has been replaced, repaired, or has undergone maintenance that could affect valve performance, a new reference value is determined, or the previous value is reconfirmed by an inservice test. This test is performed before the valve is returned to service, or immediately if the valve is not removed from service. Deviations between the previous and new reference values are identified and analyzed. Verification that the new values represent acceptable operation is documented.

3.9.6.3 Relief Requests

Insert the following text after the first paragraph in DCD Subsection 3.9.6.3:

STD COL 3.9-4

The IST Program described herein utilizes Code Case OMN-1, Revision 1, “Alternative Rules for the Preservice and Inservice Testing of Certain Electric Motor-Operated Valve Assemblies in Light Water Reactor Power Plants” (Reference 202). Code Case OMN-1 establishes alternate rules and requirements for preservice and inservice testing to assess the operational readiness of certain motor operated valves in lieu of the requirements set forth in ASME OM Code Subsection ISTC.

OMN-1, Alternative Rules for the Preservice and Inservice Testing of Certain MOVs

Code Case OMN-1, Revision 1, “Alternative Rules for the Preservice and Inservice Testing of Certain Electric Motor Operated Valve Assemblies in Light Water Reactor Power Plants,” establishes alternate rules and requirements for preservice and inservice testing to assess the operational readiness of certain motor-operated valves in lieu of the requirements set forth in OM Code Subsection ISTC. However, Regulatory Guide 1.192, “Operation and Maintenance Code Case Acceptability, ASME OM Code,” June 2003, has not yet endorsed OMN-1, Revision 1.

Code Case OMN-1, Revision 0, has been determined by the NRC to provide an acceptable level of quality and safety when implemented in conjunction with the conditions imposed in Regulatory Guide 1.192. NUREG-1482, Revision 1, “Guidelines for Inservice Testing at Nuclear Power Plants,” recommends the implementation of OMN-I by all licensees. Revision 1 to OMN-1 represents an improvement over Revision 0, as published in the ASME OM-2004 Code. OMN-1 Revision 1 incorporates the guidance on risk-informed testing of MOVs from OMN-11, "Risk-Informed Testing of Motor-Operated Valves," and provides additional guidance on design basis verification testing and functional margin, which eliminates the need for the figures on functional margin and test intervals in Code Case OMN-1.

The IST Program implements Code Case OMN-1, Revision 1, in lieu of the stroke-time provisions specified in ISTC-5120 for MOVs, consistent with the guidelines provided in NUREG-1482, Revision 1, Section 4.2.5.

Regulatory Guide 1.192 states that licensees may use Code Case OMN-1, Revision 0, in lieu of the provisions for stroke-time testing in Subsection ISTC of the 1995 Edition up to and including the 2000 Addenda of the ASME OM Code when applied in conjunction with the provisions for leakage rate testing in ISTC-3600 (1998 Edition with the 1999 and 2000 Addenda). Licensees who choose to apply OMN-1 are required to apply all of its provisions. The IST program incorporates the following provisions from Regulatory Guide 1.192:

(1) The adequacy of the diagnostic test interval for each motor-operated valve (MOV) is evaluated and adjusted as necessary, but not later than 5 years or three refueling outages (whichever is longer) from initial implementation of OMN-1.

(2) The potential increase in CDF and risk associated with extending high risk MOV test intervals beyond quarterly is determined to be small and consistent with the intent of the Commission's Safety Goal Policy Statement.

(3) Risk insights are applied using MOV risk ranking methodologies accepted by the NRC on a plant-specific or industry-wide basis, consistent with the conditions in the applicable safety evaluations.

(4) Consistent with the provisions specified for Code Case OMN-11 the potential increase in CDF and risk associated with extending high risk MOV test intervals beyond quarterly is determined to be small and consistent with the intent of the Commission's Safety Goal Policy Statement.

Compliance with the above items is addressed in Section 3.9.6.2.2. Code Case OMN-1, Revision 1, is considered acceptable for use with OM Code-2001 Edition with 2003 Addenda. Finally, consistent with Regulatory Guide 1.192, the benefits of performing any particular test are balanced against the potential adverse effects placed on the valves or systems caused by this testing.

3.9.8 COMBINED LICENSE INFORMATION

3.9.8.2 Design Specifications and Reports

Add the following text after the second paragraph in DCD Subsection 3.9.8.2.

STD COL 3.9-2

Design specifications and design reports for ASME Section III piping are made available for NRC review. Reconciliation of the as-built piping (verification of the thermal cycling and stratification loading considered in the stress analysis discussed in DCD Subsection 3.9.3.1.2) is completed by the COL holder after the construction of the piping systems and prior to fuel load (in accordance with DCD Tier 1 Section 2 ITAAC line item for the applicable systems).

3.9.8.3 Snubber Operability Testing

STD COL 3.9-3

This COL Item is addressed in Subsection 3.9.3.4.4.

3.9.8.4 Valve Inservice Testing

STD COL 3.9-4

This COL Item is addressed in Subsections 3.9.6, 3.9.6.2.2, 3.9.6.2.3, 3.9.6.2.4, 3.9.6.2.5 and 3.9.6.3.

3.9.8.5 Surge Line Thermal Monitoring

STD COL 3.9-5

This COL item is addressed in Subsection 3.9.3.1.2 and Subsection 14.2.9.2.22.

3.9.8.7 As-Designed Piping Analysis

Add the following text at the end of DCD Subsection 3.9.8.7.

STD COL 3.9-7 The as-designed piping analysis is provided for the piping lines chosen to demonstrate all aspects of the piping design. A design report referencing the as-designed piping calculation packages, including ASME Section III piping analysis, support evaluations and piping component fatigue analysis for Class 1 piping using the methods and criteria outlined in DCD Table 3.9-19 is made available for NRC review.

This COL item is also addressed in Subsection 14.3.3.

3.9.9 REFERENCES

Add the following information at the end of DCD Subsection 3.9.9:

201. Not used.

202. ASME Code Case OMN-1, Revision 1, “Alternative Rules for the Preservice and Inservice Testing of Certain Electric Motor-Operated Valve Assemblies in Light Water Reactor Power Plants.”

203. Joint Owners Group Air Operated Valve Program Document, Revision 1, December 13, 2000.

204. USNRC, Eugene V. Imbro, letter to Mr. David J. Modeen, Nuclear Energy Institute, Comments on Joint Owners’ Group Air Operated Valve Program Document, dated October 8, 1999.

STD SUP 3.9-3

Table 3.9-201
Safety Related Snubbers

System	Snubber (Hanger) No.	Line #	System	Snubber (Hanger) No.	Line #
CVS	APP-CVS-PH-11Y0164	L001	RNS	APP-RNS-PH-12Y2060	L006
PXS	APP-PXS-PH-11Y0020	L021A	SGS	APP-SGS-PH-11Y0001	L003B
RCS	APP-RCS-PH-11Y0039	L215	SGS	APP-SGS-PH-11Y0002	L003B
RCS	APP-RCS-PH-11Y0067	L005B	SGS	APP-SGS-PH-11Y0004	L003B
RCS	APP-RCS-PH-11Y0080	L112	SGS	APP-SGS-PH-11Y0057	L003A
RCS	APP-RCS-PH-11Y0081	L215	SGS	APP-SGS-PH-11Y0058	L004B
RCS	APP-RCS-PH-11Y0082	L112	SGS	APP-SGS-PH-11Y0063	L003A
RCS	APP-RCS-PH-11Y0090	L118A	SGS	APP-SGS-PH-11Y0065	L005B
RCS	APP-RCS-PH-11Y0099	L022B	SGS	APP-SGS-PH-12Y0136	L015C
RCS	APP-RCS-PH-11Y0103	L003	SGS	APP-SGS-PH-12Y0137	L015C
RCS	APP-RCS-PH-11Y0105	L003	SGS	APP-SGS-PH-11Y0470	L006B
RCS	APP-RCS-PH-11Y0112	L032A	SGS	APP-SGS-PH-11Y2002	L006A
RCS	APP-RCS-PH-11Y0429	L225B	SGS	APP-SGS-PH-11Y2021	L006A
RCS	APP-RCS-PH-11Y0528	L005A	SGS	APP-SGS-PH-11Y3101	L006B
RCS	APP-RCS-PH-11Y0539	L225C	SGS	APP-SGS-PH-11Y3102	L006B
RCS	APP-RCS-PH-11Y0550	L011B	SGS	APP-SGS-PH-11Y3121	L006B
RCS	APP-RCS-PH-11Y0551	L011A	SGS	APP-SGS-PH-11Y0463	L006A
RCS	APP-RCS-PH-11Y0553	L153B	SGS	APP-SGS-PH-11Y0464	L006A
RCS	APP-RCS-PH-11Y0555	L153A	SGS	SG 1 Snubber A (1A)	(1)
RCS	APP-RCS-PH-11Y2005	L022A	SGS	SG 1 Snubber B (1B)	(1)
RCS	APP-RCS-PH-11Y2101	L032B	SGS	SG 2 Snubber A (2A)	(1)
RCS	APP-RCS-PH-11Y2117	L225A	SGS	SG 2 Snubber B (2B)	(1)

(1) These snubbers are on the upper lateral support assembly of the steam generators.

3.10 SEISMIC AND DYNAMIC QUALIFICATION OF SEISMIC CATEGORY I MECHANICAL AND ELECTRICAL EQUIPMENT

This section of the referenced DCD is incorporated by reference with no departures or supplements.

3.11 ENVIRONMENTAL QUALIFICATION OF MECHANICAL AND ELECTRICAL EQUIPMENT

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

3.11.5 COMBINED LICENSE INFORMATION ITEM FOR EQUIPMENT QUALIFICATION FILE

Add the following text to the end of DCD Subsection 3.11.5.

STD COL 3.11-1

The COL holder is responsible for the maintenance of the equipment qualification file upon receipt from the reactor vendor. The documentation necessary to support the continued qualification of the equipment installed in the plant that is within the Environmental Qualification (EQ) Program scope is available in accordance with 10 CFR Part 50 Appendix A, General Design Criterion 1.

EQ files developed by the reactor vendor are maintained as applicable for equipment and certain post-accident monitoring devices that are subject to a harsh environment. The contents of the qualification files are discussed in DCD Section 3D.7. The files are maintained for the operational life of the plant.

For equipment not located in a harsh environment, design specifications received from the reactor vendor are retained. Any plant modifications that impact the equipment use the original specifications for modification or procurement. This process is governed by applicable plant design control or configuration control procedures.

Central to the EQ Program is the EQ Master Equipment List (EQMEL). This EQMEL identifies the electrical and mechanical equipment or components that must be environmentally qualified for use in a harsh environment. The EQMEL consists of equipment that is essential to emergency reactor shutdown, containment isolation, reactor core cooling, or containment and reactor heat removal, or that is otherwise essential in preventing significant release of radioactive material to the environment. This list is developed from the equipment list provided in AP1000 DCD Table 3.11-1. The EQMEL and a summary of equipment qualification results are maintained as part of the equipment qualification file for the operational life of the plant.

Administrative programs are in place to control revision to the EQ files and the EQMEL. When adding or modifying components in the EQ Program, EQ files are generated or revised to support qualification. The EQMEL is revised to reflect these new components. To delete a component from the EQ Program, a deletion justification is prepared that demonstrates why the component can be deleted. This justification consists of an analysis of the component, an associated circuit review if appropriate, and a safety evaluation. The justification is released and/or referenced on an appropriate change document. For changes to the EQMEL, supporting documentation is completed and approved prior to issuing the changes. This documentation includes safety reviews and new or revised EQ files. Plant modifications and design basis changes are subject to change process reviews, e.g. reviews in accordance with 10 CFR 50.59 or Section VIII of Appendix D to 10 CFR Part 52, in accordance with appropriate plant procedures. These reviews address EQ issues associated with the activity. Any changes to the EQMEL that are not the result of a modification or design basis change are subject to a separate review that is accomplished and documented in accordance with plant procedures.

Engineering change documents or maintenance documents generated to document work performed on an EQ component, which may not have an impact on the EQ file, are reviewed against the current revision of the EQ files for potential impact. Changes to EQ documentation may be due to, but not limited to, plant modifications, calculations, corrective maintenance, or other EQ concerns.

Table 13.4-201 provides milestones for EQ implementation.

APPENDIX 3A HVAC DUCTS AND DUCT SUPPORTS

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3B LEAK-BEFORE-BREAK EVALUATION OF THE AP1000 PIPING

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3C REACTOR COOLANT LOOP ANALYSIS METHODS

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3D METHODOLOGY FOR QUALIFYING AP1000 SAFETY-RELATED ELECTRICAL AND MECHANICAL EQUIPMENT

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3E HIGH-ENERGY PIPING IN THE NUCLEAR ISLAND

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3F CABLE TRAYS AND CABLE TRAY SUPPORTS

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3G NUCLEAR ISLAND SEISMIC ANALYSES

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3H AUXILIARY AND SHIELD BUILDING CRITICAL SECTIONS

This section of the referenced DCD is incorporated by reference with no departures or supplements.

APPENDIX 3I EVALUATION FOR HIGH FREQUENCY SEISMIC INPUT

This section of the referenced DCD is incorporated by reference with no departures or supplements.