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Subject: **Response to Portion of NRC Request for Additional Information Letter No. 109 Related to ESBWR Design Certification Application – DCD Tier 2 Section 3.7 – Seismic Design - RAI Number 3.7-61**

Enclosure 1 contains GEH's response to the subject NRC RAI transmitted via the Reference 1 letter.

If you have any questions or require additional information regarding the information provided here, please contact me.

Sincerely,

Kathy Sedney for

James C. Kinsey
Vice President, ESBWR Licensing

D068
NRO

Reference:

1. MFN 07-555, Letter from U.S. Nuclear Regulatory Commission to Robert E. Brown, *Request For Additional Information Letter No. 109 Related To ESBWR Design Certification Application*, dated October 12, 2007

Enclosure:

1. MFN 07-626 - Response to Portion of NRC Request for Additional Information Letter No. 109 Related to ESBWR Design Certification Application – DCD Tier 2 Section 3.7 – Seismic Design - RAI Number 3.7-61

cc:	AE Cubbage	USNRC (with enclosure)
	GB Stramback	GEH/San Jose (with enclosure)
	RE Brown	GEH/Wilmington (with enclosure)
	DH Hinds	GEH/Wilmington (with enclosure)
	eDRF	0000-0076-4147

ENCLOSURE 1

MFN 07-626

Response to Portion of NRC Request for

Additional Information Letter No. 109

Related to ESBWR Design Certification Application

DCD Tier 2 Section 3.7 – Seismic Design

RAI Number 3.7-61

NRC RAI 3.7-61

The staff's review of DCD, Tier 2, Rev. 3 identified a number of changes to Section 3.7 that are not related to the 3.7 RAIs but these changes were made due to other RAIs.

(1) Last two sentences of Paragraph 1 of DCD Section 3.7 "Seismic Design", added in reference to RAI 3.8-9.

The staff's review of RAI 3.8-9 indicates that it is still open. The sentence, "The method of combination of peak dynamic responses to seismic and RBV loads is the Square Root of the Sum of the Squares (SRSS) in accordance with NUREG- 0484 Revision 1" is not acceptable. In addition, the sentence, "For reinforced concrete structures the section forces or stresses due to each dynamic load are combined in the most conservative manner by systematically varying the sign (+ or -), equivalent to the absolute sum method." is not acceptable. These two sentences should be deleted at this time. Appropriate wording, acceptable to the staff, can be added after the resolution of RAI 3.8-9.

(2) In Paragraph 2 of Section 3.7.2.1.1 "Time History Method," GEH added the following sentence in reference to RAI 3.12-4 S2.

"The approach for selecting the time step, Δt, is that the Δt used shall be small enough such that the use of ½ of Δt does not change the response by more than 10%."

The staff noted that this sentence was added in the middle of the discussion of the numerical integration time step for time-domain solutions. The added sentence is unacceptable in its present form and location in the discussion, since it supercedes the use of one-tenth of the shortest period of interest. Revise the paragraph.

(3) In Paragraph 4, Step 1, of Section 3.7.2.7, "Combination of Modal Responses", GEH added the following two sentences in reference to RAI 3.12-20 S2:

"The ZPA cutoff frequency is 100 Hz or f_{ZPA} as defined in Figures 1, 2 and 3 of Regulatory Guide 1.92. It is applicable to seismic and other building dynamic loads."

The staff noted that these two sentences were added in the middle of step 1. The added sentences are inappropriate in their present form and in the discussion. The wording should be changed to make the discussion clear.

(4) The following statement was added in Paragraph 1 of DCD Section 3.7.2.9, "Effect of Parameter Variation on Floor Response Spectra," in reference to RAI 3.12-6 S1:

"When the calculated floor acceleration time history is used in the time history analysis for piping and equipment, the uncertainties in the time history are accounted for by expanding and shrinking the time history within $1/(1\pm0.15)$ so as to change the frequency content of the time history within ±15%. Alternatively, a synthetic time history that is compatible with the broadened floor response spectra may be used.

The methods of peak broadening described above are applicable to seismic and other building dynamic loads."

The staff notes that the alternate approach to addressing parameter variation, when using the time history method for analysis of building-attached piping and equipment, is appropriate but it needs additional clarifications. Also, the title of the section is no longer inclusive of the information in the section.

(5) The staff noted that all COL information related to seismic design parameters (Section 3.7.5.1) was deleted. GEH referenced RAI 3.8-95 S1 as the basis for the deletion.

Rev.2 of DCD Section 3.7.5 included changes that resulted from 3.7 RAIs. Therefore, the affected 3.7 RAIs may have to be reopened, unless GEH (1) includes in Section 3.7.5 very specific references to other Tier 1 and Tier 2 sections for all 4 items that were included in Rev. 2; and (2) ensures that the exact wording that was previously accepted by the staff and incorporated in Rev. 2 is retained. The second condition is stipulated based on review of DCD Tier 2 Table 2.0-1, Rev. 3, related to the minimum shear wave velocity. The very specific wording previously accepted by the staff and incorporated in DCD Rev. 2, Section 3.7.5 has been modified, and may be subject to misinterpretation

The deletion of DCD Section 3.7.5 is not acceptable to the staff.

GEH Response

(1) The two sentences "The method of combination of peak dynamic responses to seismic and RBV loads is the Square Root of the Sum of the Squares (SRSS) in accordance with NUREG-0484 Revision 1. For reinforced concrete structures the section forces or stresses due to each dynamic load are combined in the most conservative manner by systematically varying the sign (+ or -), equivalent to the absolute sum method" will be deleted from DCD Tier 2 Section 3.7. Appropriate wording, acceptable to the staff, will be added after the resolution of NRC RAI 3.8-9.

(2) The third sentence in the second paragraph of DCD Tier 2 Subsection 3.7.2.1.1, "The approach for selecting the time step, Δt, is that the Δt used is small enough such that the use of ½ of Δt does not change the response by more than 10%", will be deleted. A new sentence, "The adequacy of the selected time step (Δt) is checked by ensuring that use of ½ of Δt does not change the response by more than 10%", will be added before the sentence, "For the frequency domain…", in the second paragraph of DCD Tier 2 Subsection 3.7.2.1.1.

(3) DCD Tier 2 Subsection 3.7.2.7, Step 1 will be revised to read as follows:

> "Determine the modal responses only for those modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA of the input response spectrum (f_{ZPA}). Examples of f_{ZPA} are shown in Figures 1, 2 and 3 of Regulatory Guide 1.92, Rev. 2. Combine such modes in accordance with the methods described above.
>
> When applying these methods to building dynamic loads other than seismic, it is acceptable to use a ZPA cutoff frequency of 100 Hz if the spectral acceleration at 100 Hz has not returned to the ZPA of the response spectrum."

(4) The second and third sentences in the first paragraph of DCD Tier 2 Subsection 3.7.2.9 will be deleted and replaced by a new paragraph to read:

> "When, in lieu of response spectrum analysis, the calculated floor acceleration time history is used to perform a time history analysis of piping and equipment, uncertainties are accounted for by expanding and shrinking the floor acceleration time history within $1/(1\pm0.15)$ so as to change the frequency content of the time history by $\pm15\%$. In this case, multiple time history analyses are performed. Alternatively, a single synthetic time history, which matches the broadened floor response spectra, may be used."

Also, the second paragraph of DCD Tier 2 Subsection 3.7.2.9 will be revised to read "The methods described above to account for the effect of parameter variation are applicable to seismic and other building dynamic loads."

The title of DCD Tier 2 Subsection 3.7.2.9 is consistent with SRP 3.7.2.9.

(5) The four items deleted from DCD Tier 2, Revision 2, Subsection 3.7.5 will be added back to DCD Tier 2 Subsection 3.7.5 with pointers to DCD Tier 2 Table 2.0-1 as follows:

- SSE ground response spectra: The sentence "See Table 2.0-1 for seismology requirements of site-specific SSE ground response spectra" will be added to DCD Tier 2 Subsection 3.7.5.
- Foundation bearing capacities: The sentence "See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity and liquefaction potential" will be added to DCD Tier 2 Subsection 3.7.5.
- Shear wave velocity: The sentence "See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity and liquefaction potential" will be added to DCD Tier 2 Subsection 3.7.5.
- Liquefaction potential: The sentence "See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity and liquefaction potential" will be added to DCD Tier 2 Subsection 3.7.5.

As discussed in a November 6, 2007 teleconference between the NRC and GEH, the four items deleted from DCD Tier 2, Revision 1, Subsection 3.8.6 will be added back to DCD Tier 2 Subsection 3.8.6 with pointers to DCD Tier 1 Table 2.15.1-2, DCD Tier 2 Table 2.0-1 and DCD Tier 2 Section 3.8 as follows:

- Foundation waterproofing: The sentences "The selected waterproofing material for the bottom of the basemat is a chemical crystalline powder that is added to the mud mat mixture forming a water proof barrier when cured. No membrane waterproofing is used under the foundations in the ESBWR" will be added to DCD Tier 2 Subsection 3.8.6.
- Site-specific physical properties and foundation settlement: The sentence "See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity, liquefaction potential and maximum settlement values for Seismic Category I buildings" will be added to DCD Tier 2 Subsection 3.8.6.

- Structural integrity pressure result: The sentence "See DCD Tier 1 Table 2.15.1-2 for the Structural Integrity Test (SIT) of the containment structure, which is an ITAAC item" will be added to DCD Tier 2 Subsection 3.8.6.
- Identification of Seismic Category I structures: The sentence "See Subsections 3.8.1, 3.8.2, 3.8.3 and 3.8.4 for identification of Seismic Category I structures" will be added to DCD Tier 2 Subsection 3.8.6.

DCD Impact

DCD Tier 2 Section 3.7, Subsections 3.7.2.1.1, 3.7.2.7, 3.7.2.9, 3.7.5 and 3.8.6 will be revised in the next update as noted in the attached markups.

3.7 SEISMIC DESIGN

For seismic design purposes, all structures, systems, and components of the ESBWR standard plant are classified into Seismic Category I (C-I), Seismic Category II (C-II), or Non-Seismic (NS) in accordance with the requirements to withstand the effects of the Safe Shutdown Earthquake (SSE) as defined in Section 3.2. For those C-I and C-II structures, systems and components in the reactor building complex, the effects of other dynamic loads caused by reactor building vibration (RBV) caused by suppression pool dynamics are also considered in the design. Although this section addresses seismic aspects of design and analysis in accordance with Regulatory Guide 1.70, the methods of this section are also applicable to RBV dynamic loadings, unless noted otherwise.

The safe shutdown earthquake (SSE) is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology, seismology, and specific characteristics of local subsurface material. It is the earthquake that produces the maximum vibratory ground motion for which Seismic Category I structures, systems and components (SSC) are designed to remain functional and within applicable stress, strain, and deformation limits. These systems and components are those necessary to ensure the following:

- The integrity of the reactor coolant pressure boundary (RCPB);
- The capability to shut down the reactor and maintain it in a safe condition; or
- The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the applicable guidelines exposures set forth in 10 CFR 100 (10 CFR 50.34(a)).

ESBWR response to an earthquake up to SSE may achieve shutdown of the reactor and maintenance of it in a safe condition using the Automatic Depressurization System and Gravity Driven Cooling System as described in the Probabilistic Risk Assessment. In this case, depressurization is accomplished in part with Depressurization Valves that remain open in order for the Gravity Driven Cooling System and the Passive Containment Cooling System to perform their safety functions.

Seismic Category II (C-II) includes all plant SSC which perform no safety-related function, and whose continued function is not required, but whose structural failure or interaction could degrade the functioning of a Seismic Category I structure, system or component to an unacceptable safety level, or could result in incapacitating injury to occupants of the control room. Thus, this category includes the SSC whose structural integrity, not their operational performance, is required. The methods of seismic analysis and design acceptance criteria for C-II SSC are the same as C-I; however, the procurement, fabrication and construction requirements for C-II SSC are in accordance with industry practices. Seismic Category- II (C-II) items are those corresponding to position C.2 of Regulatory Guide 1.29.

The Operating Basis Earthquake (OBE) is a design requirement. For the ESBWR OBE ground motion is chosen to be one-third of the SSE ground motion. Therefore, no explicit response or design analysis is required to show that OBE design requirements are met. This is consistent with Appendix S to 10 CFR 50. The effects of low-level earthquakes (lesser magnitude than the SSE) on fatigue evaluation and plant shutdown criteria are addressed in Subsections 3.7.3.2 and 3.7.4.4, respectively.

3.7.1.2 Percentage of Critical Damping Values

Damping values of various structures and components are shown in Table 3.7-1 for use in SSE dynamic analysis. These damping values are consistent with Regulatory Guide 1.61 SSE damping except for the damping value of cable trays and conduits.

The damping values shown in Table 3.7-1 for cable trays and conduits are based on the results of over 2000 individual dynamic tests conducted by Bechtel/ANCO for a variety of raceway configurations (Reference 3.7-5). The damping value of conduit systems (including supports) is 7% constant. For HVAC ducts and supports the damping value is 7% for companion angle construction, 10% for pocket lock construction and 4% for welded construction.

For ASME Section III, Division 1 Class 1, 2, and 3, and ASME B31.1 piping systems, the damping values of Table 3.7-1 or alternative damping values specified in Figure 3.7-37 are used. The damping values shown in Table 3.7-1 are applicable to all modes of a structure or component constructed of the same material. Damping values for systems composed of subsystems with different damping properties are obtained using the procedures described in Subsection 3.7.2.13.

3.7.1.3 Supporting Media for Category I Structures

The Seismic Category I structures have concrete mat foundations supported on soil, rock or compacted backfill. The embedment depth, dimensions of the structural foundation, and total structural height for each structure are given in Subsection 3.8.5.1. The soil conditions considered for soil-structural interaction analysis are described in Appendix 3A.

3.7.2 Seismic System Analysis

This section applies to building structures that constitute primary structural systems (RB, FB, CB, and FWSC). The reactor pressure vessel (RPV) is not a primary structural component but, due to its dynamic interaction with the supporting structure, it is considered as another part of the primary system of the reactor building for the purpose of dynamic analysis. Table 3.7-3 provides a summary of methods of seismic analysis for primary building structures.

3.7.2.1 Seismic Analysis Methods

Analysis can be performed using any of the following methods:

- time history method;
- response spectrum method;
  - singly- or multi-supported system with Uniform Support Motion (USM); or
  - multi-supported system with Independent Support Motion (ISM); or
- static coefficient method.

3.7.2.1.1 Time History Method

The response of a multi-degree-of-freedom linear system subjected to external forces and/or uniform support excitations is represented by the following differential equations of motion in the matrix form:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{P\} \tag{3.7-1}$$

where,

$[M]$ = mass matrix

$[C]$ = damping matrix

$[K]$ = stiffness matrix

$\{u\}$ = column vector of time-dependent relative displacements

$\{\dot{u}\}$ = column vector of time-dependent relative velocities

$\{\ddot{u}\}$ = column vector of time-dependent relative accelerations

$\{P\}$ = column vector of time-dependent applied forces

= $-[M]\{\ddot{x}_g\}$ for support excitation in which $\{\ddot{x}_g\}$ is column vector of time-dependent support accelerations

The above equation can be solved by modal superposition or direct integration in the time domain, or by the complex frequency response method in the frequency domain. For the time domain solution, the numerical integration time step is sufficiently small to accurately define the dynamic excitation and to render stability and convergency of the solution up to the highest frequency (or shortest period) of significance. For most of commonly used numerical integration methods (such as Newmark β-method and Wilson θ-method), the maximum time step is limited to one-tenth of the shortest period of significance. The adequacy of the selected time step (Δt) is checked by ensuring that use of ½ of Δt does not change the response by more than 10%. For the frequency domain solution, the dynamic excitation time history is digitized with time steps no larger than the inverse of two times the highest frequency of significance and the frequency interval is selected to accurately define the transfer functions at structural frequencies within the range of significance.

The modal superposition method is used when the equation of motion (Equation 3.7-1) can be decoupled using the transformation,

$$\{u\} = [\phi]\{q\} \tag{3.7-2}$$

where,

$[\phi]$ = mode shape matrix; often mass normalized, i.e., $[\phi]^T[M][\phi] = [1]$

$\{q\}$ = column vector of normal or generalized coordinates

Substituting Equation 3.7-2 into Equation 3.7-1 and multiplying each term by the transposition of the mode shape matrix results in the uncoupled equation of motion due to the orthogonality of the mode shapes (note that the orthogonality condition of the damping matrix is assumed). For systems subjected to base acceleration excitation, $\ddot{x}_g$, the equation of motion for the jth mode is

$$\ddot{q}_j + 2\lambda_j\omega_j\dot{q}_j + \omega_j^2 q_j = -\Gamma_j\ddot{x}_g \tag{3.7-3}$$

$$R_{pI} = \left[\sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{ij} R_{pi} R_{pj} \right]^{1/2} \tag{3.7-10A}$$

where R_{pI} = combined periodic response for the Ith component of seismic input motion (I = 1, 2, 3, for one vertical and two horizontal components), ε_{ij} = the modal correlation coefficient for modes i and j, R_{pi} = periodic response or periodic component of a response of mode i, R_{pj} = periodic response or periodic component of a response of mode j, and n = number of modes considered in the combinations of modal responses.

For completely correlated modes i and j, $\varepsilon_{ij} = 1$; for partially correlated modes i and j, $0 < \varepsilon_{ij} < 1$; for uncorrelated modes i and j, $\varepsilon_{ij} = 0$.

The modal correlation coefficients are uniquely defined, depending on the method chosen for evaluating the correlation, as follows.

Rosenblueth provided the first significant mathematical approach to the evaluation of modal correlation for seismic response spectrum analysis. It is based on the application of random vibration theory, utilizing a finite duration of white noise to represent seismic loading. A formula for calculation of the coefficient ε_{ij} as a function of modal frequencies (f_i, f_j), modal damping ratios (λ_i, λ_j), and the time duration of strong earthquake motion (t_D) was derived as follows:

$$\varepsilon_{ij} = \left[1 + \left(\frac{f'_i - f'_j}{\lambda'_i f_i + \lambda'_j f_j} \right)^2 \right]^{-1} \tag{3.7-10B}$$

where

$$f'_i = f_i \left[1 - \lambda_i^2 \right]^{1/2}$$

$$\lambda'_i = \lambda_i + \frac{1}{\pi t_D f_i}$$

and f'_j, λ'_j are similarly defined.

Appendix D to Reference 3.7-17 tabulates numerical values of ε_{ij} for the Rosenblueth formula as a function of frequency, frequency ratio, and strong motion duration time for constant modal damping of 1%, 2%, 5% and 10%. The effect of t_D is most significant at 1% damping and low frequency. For 5% and 10% damping, t_D = 10 sec. and 1,000 sec. produced similar values for ε_{ij} regardless of frequency. The most significant result is that ε_{ij} is highly dependent on the damping ratio; for 2%, 5% and 10% damping, ε_{ij} = 0.2, 0.5 and 0.8, respectively, at a frequency ratio of 0.9 (modal frequencies within 10%).

For modal combination involving high-frequency modes, the following procedure applies:

Step 1. Determine the modal responses only for those modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA of the input response spectrum (f_{ZPA}). Examples of f_{ZPA} are shown in Figures 1, 2 and 3 of Regulatory Guide 1.92, Rev. 2. Combine such modes in accordance with the methods described above.

When applying these methods to building dynamic loads other than seismic, it is acceptable to use a ZPA cutoff frequency of 100 Hz if the spectral acceleration at 100 Hz has not returned to the ZPA of the response spectrum.

Step 2. For each degree-of-freedom (DOF) included in the dynamic analysis, determine the fraction of DOF mass included in the summation of all modes included in Step 1. This fraction d_i for each DOF i is given by the following equation:

$$d_i = \sum_{n=1}^{N} \left[\left(c_{n,j} \right) \left(\phi_{n,i} \right) \right] \tag{3.7-11}$$

where

n = mode number ($1, 2, \ldots, N$)

N = the number of modes included in Step 1

$\phi_{n,i}$ = eigenvector value for mode n and DOF i

j = direction of input motion

$c_{n,j}$ = participation factor for mode n in the jth direction:

$$c_{n,j} = \frac{\{\phi_n\}^T [m] \{\delta_{ij}\}}{\{\phi_n\}^T [m] \{\phi_n\}}$$

where δ_{ij} is the Kronecker delta, which is 1 if DOF i is in the direction of the earthquake input motion j and 0 if DOF i is a rotation or not in the direction of the earthquake input motion j. This assumes that the three orthogonal directions of earthquake input motion are coincident with the DOF directions. Also, $[m]$ is the mass matrix.

Next, determine the fraction of DOF mass not included in the summation of these modes:

$$e_i = d_i - \delta_{ij} \tag{3.7-12}$$

Step 3. Higher modes can be assumed to respond in phase with the ZPA and, thus, with each other; hence, these modes are combined algebraically, which is equivalent to pseudostatic response to the inertial forces from these higher modes excited at the ZPA. The pseudostatic inertial forces associated with the summation of all higher modes for each DOF i are given by the following:

$$P_i = (ZPA)(M_i)(e_i) \tag{3.7-13}$$

where P_i is the force or moment to be applied at DOF i, M_i is the mass or mass moment of inertia associated with DOF i.

The structure is then statically analyzed for this set of pseudostatic inertial forces applied to all degrees of freedom to determine the maximum responses associated with high-frequency modes not included in Step 1.

This procedure requires the computation of individual modal responses only for lower-frequency modes. Thus, the more difficult higher-frequency modes need not be determined. The procedure ensures inclusion of all modes of the structural model and proper representation of DOF masses.

3.7.2.8 Interaction of Non-Category I Structures with Seismic Category I Structures

The interfaces between Seismic Category I and non-Seismic Category I structures, systems and components are designed for the dynamic loads and displacements produced by both the Category I and non-Category I structures, systems and components. All non-Category I structures, systems and components meet at least one of the following requirements:

(1) The collapse of any non-Category I structure, system or component does not cause the non-Category I structure, system or component to strike a Seismic Category I structure, system or component. SSCs in this category are classified as NS. Any NS structure postulated to fail under SSE (except the Radwaste Building) is located at least a distance of its height above grade from C-I structures.

(2) The collapse of any non-Category I structure, system or component does not impair the integrity of Seismic Category I structures, systems or components. This is demonstrated by showing that the impact loads on the Category I structure, system or component resulting from collapse of an adjacent non-Category I structure, because of its size and mass, are either negligible or smaller than those considered in the design (e.g., loads associated with tornado, including missiles). SSCs in this category are classified as NS.

(3) The non-Category I structures, systems or components are analyzed and designed to prevent their failure under SSE conditions in a manner such that the margin of safety of these structures, systems or components is equivalent to that of Seismic Category I structures, systems or components. SSCs in this category are classified as C-II.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

Floor response spectra calculated according to the procedures described in Subsection 3.7.2.5 are peak broadened by ±15% to account for uncertainties in the structural frequencies owing to uncertainties in the material properties of the structure and soil and to approximations in the modeling techniques used in the analysis.

When, in lieu of response spectrum analysis, the calculated floor acceleration time history is used to perform a time history analysis of piping and equipment, uncertainties are accounted for by expanding and shrinking the floor acceleration time history within 1/(1±0.15) so as to change the frequency content of the time history by ±15%. In this case, multiple time history analyses are performed. Alternatively, a single synthetic time history, which matches the broadened floor response spectra, may be used.

The methods described above to account for the effect of parameter variation are applicable to seismic and other building dynamic loads.

Following plant shutdown, post-shutdown inspections and tests are performed in accordance with Reference 3.7-10, as permitted by Regulatory Guide 1.167, to determine the physical condition of the plant and its readiness to resume operation. After plant is restarted (or prior to restart if the earthquake caused significant damage to the plant per Reference 3.7-10 definition), long-term evaluations are carried out for engineering assessments of plant structures and equipment using the actual event records to assure their long-term reliability in accordance with Reference 3.7-10 guidelines, as permitted by Regulatory Guide 1.167.

3.7.4.5 In-Service Surveillance

The seismic instrumentation operates during all modes of plant operation including periods of plant shutdown. The maintenance and repair procedures keep the maximum number of instruments in service during plant operation and shutdown. The walkdown inspection following a felt earthquake ensures the safety condition of the plant.

Each of the seismic instruments is demonstrated operable by the performance of the channel check, channel calibration, and channel functional test operations. The channel checks are performed every two weeks for the first three months of service after startup. After the initial three-month period and three consecutive successful checks, the channel checks are performed on a monthly basis. The channel calibration are performed during each refueling. The channel functional test is performed every 6 months.

3.7.5 Site-Specific Information

(1) See Table 2.0-1 for seismology requirements of site-specific SSE ground response spectra.

(2) See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity and liquefaction potential.

3.7.6 References

3.7-1 (Deleted)

3.7-2 Dominion Nuclear North Anna, LLC, "North Anna Early Site Permit Application," Revision 4, May 2005.

3.7-3 Exelon Generation Company, LLC, "Clinton Early Site Permit Application," Revision 0, September 2003.

3.7-4 System Energy Resources, INC, "Grand Gulf Early Site Permit Application," Revision 0, October 2003.

3.7-5 P. Koss, "Seismic Testing of Electrical Cable Support Systems, Structural Engineers of California Conference," San Diego, September 1979.

3.7-6 L. K. Liu, "Seismic Analysis of the Boiling Water Reactor, symposium on seismic analysis of pressure vessel and piping components, First National Congress on Pressure Vessel and Piping," San Francisco, California, May 1971.

3.7-7 M. P. Singh, "Seismic Design Input for Secondary Systems, ASCE Mini-Conference on Civil Engineering and Nuclear Power," Vol. II, Boston, April 1979.

where F_s and F_p are the shearing and sliding resistance, and passive soil pressure resistance, respectively. F_d is the maximum lateral seismic force including any dynamic active earth pressure, and F_h is the maximum lateral force due to loads other than seismic loads.

The factor of safety against flotation is defined as:

$$FS = F_{DL}/F_B$$

where F_{DL} is the downward force due to dead load and F_B is the upward force due to buoyancy.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The foundations of Seismic Category I structures are constructed of reinforced concrete using proven methods common to heavy industrial construction. For further discussion, see Subsection 3.8.1.6.

3.8.5.7 Testing and In-Service Inspection Requirements

The foundations of Seismic Category I structures are monitored per NUREG-1801 and 10 CFR 50.65 as clarified in RG 1.160, in accordance with Section 1.5 of RG 1.160.

3.8.6 Special Topics

3.8.6.1 Foundation Waterproofing

The selected waterproofing material for the bottom of the basemat is a chemical crystalline powder that is added to the mud mat mixture forming a water proof barrier when cured. No membrane waterproofing is used under the foundations in ESBWR.

3.8.6.2 Site-Specific Physical Properties and Foundation Settlement

See Table 2.0-1 for soil properties requirements of site-specific foundation bearing capacities, minimum shear wave velocity, liquefaction potential and maximum settlement values for Seismic Category I buildings.

3.8.6.3 Structural Integrity Pressure Result

See DCD Tier 1 Table 2.15.1-2 for the Structural Integrity Test (SIT) of the containment structure, which is an ITAAC item.

3.8.6.4 Identification of Seismic Category I Structures

See Subsections 3.8.1, 3.8.2, 3.8.3 and 3.8.4 for identification of Seismic Category I structures.