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MFN 06-135
Supplement 1

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Subject: **Response to Portion of RAI Letter Number 20 Related to ESBWR Design Certification Application – Seismic Design – RAI Numbers 3.7-2, 3.7-13, 3.7-17, 3.7-18, 3.7-19, 3.7-21, 3.7-22, 3.7-36, 3.7-40, 3.7-47, 3.7-48, 3.7-49, 3.7-51, and 3.7-53 – SUPPLEMENT 1**

Enclosure 1 contains supplemental responses to the subject RAIs resulting from the June 2006 NRC Seismic Audit. GE's original responses were transmitted via the Reference 1 letter. There are no changes to any of the remaining RAI responses contained in the Reference 1 letter.

If you have any questions about the information provided here, please let me know.

Sincerely,

Kathy Sedney for

David H. Hinds
Manager, ESBWR

D068

Enclosure:

1. MFN 06-135, Supplement 1 - Response to Portion of RAI Letter Number 20 Related to ESBWR Design Certification Application – Seismic Design – RAI Numbers 3.7-2, 3.7-13, 3.7-17, 3.7-18, 3.7-19, 3.7-21, 3.7-22, 3.7-36, 3.7-40, 3.7-47, 3.7-48, 3.7-49, 3.7-51, and 3.7-53 – SUPPLEMENT 1

Reference:

1. MFN 06-135, Letter from David H. Hinds to U. S. Nuclear Regulatory Commission, *Partial Response to RAI Letter Numbers 20 and 27 Related to ESBWR Design Certification Application – Seismic Design – DCD Sections 2.5 and 3.7 – RAI Numbers 2.5-2 through 2.5-7; 3.7-1 through 3.7-4, 3.7-6, 3.7-9, 3.7-10, 3.7-13 through 3.7-15, 3.7-17 through 3.7-23, 3.7-28, 3.7-31, 3.7-36, 3.7-40 through 3.7-49, 3.7-51, 3.7-53, and 3.7-56,* May 23, 2006

| | | |
|---|---|---|
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| | GB Stramback | GE/San Jose (with enclosures) |
| | eDRF | 0000-0057-2938 |

# ENCLOSURE 1

# MFN 06-135, SUPPLEMENT 1

**Response to Portion of RAI Letter Number 20**

**Related to ESBWR Design Certification Application**

**Seismic Design – RAI Numbers 3.7-2, 3.7-13, 3.7-17, 3.7-18,**

**3.7-19, 3.7-21, 3.7-22, 3.7-36, 3.7-40, 3.7-47, 3.7-48, 3.7-49,**

**3.7-51, and 3.7-53 – SUPPLEMENT 1**

## NRC RAI 3.7-2

*In the fifth paragraph in Page 3.7-1 (DCD Section 3.7), the applicant provided the seismic analysis and design criteria for the non-seismic (NS) SSCs. In order to assist the staff to complete its review, the applicant is requested to:*

*(a) (1) Identify the NS structures (which are to be designed to the International Building Code (IBC) seismic criteria) that are included in the scope of the ESBWR DCD; (2) explain why they are not classified as C-1 or C-II; and (3) identify where the seismic design basis calculations are described in the DCD.*

*(b) (1) Identify what NS equipment is seismically qualified (either by test or analysis) to IBC seismic criteria; and (2) described the technical rationale for such seismic qualification.*

*(c) Clarify what is the scope of the COL applicant's responsibility to implement IBC seismic design criteria for NS SSCs?*

## GE Response

(a) Please refer to DCD Tier 2 Table 3.2-1 for identification of NS structures. They are not classified as C-1 or C-II because their failure will not adversely affect the performance of safety related SSCs. Therefore, seismic design basis calculations are not done at this stage for the DCD.

(b) NS equipment including its anchorage is designed to IBC seismic criteria which is the standard industry practice for industry grade components.

(c) IBC seismic design requirements for NS SSCs will be used and the requirements will be implemented through design, fabrication, and installation specifications and purchase order documents.

A markup of DCD Tier 2 Section 3.7 was provided in MFN 06-135.

**NRC RAI 3.7-2, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Remove the following from DCD Section 3.7 (p. 3.7-1 of DCD Rev. 1): "Non-seismic (NS) structures and equipment are those that do not fall into Seismic Category I or II definitions. These are shown on Table 3.2-1. NS structures and equipment are designed for seismic requirements in accordance with the International Building Code (IBC) Reference 3.7-1. The building structures are classified as Category IV (Power Generating Stations) with an Occupancy Importance Factor of 1.5. Either of the methods permitted by IBC, simplified analysis or dynamic analysis, is acceptable for determination of seismic loads on NS structures and equipment."*

**GE Response**

DCD Tier 2 Section 3.7 (p. 3.7-2) will be revised as recommended by the NRC in the next update as noted in the attached markup.

**NRC RAI 3.7-13**

*Because friction-bolted steel structures are designed to eliminate slip of the bolted joints by applying a preload, and consequently behave more like welded steel structures, the staff considers 4% SSE damping to be appropriate for friction-bolted steel structures. For ≥50% fill of cable, and in the absence of physical restraint, the staff considers 10% SSE damping to be acceptable for cable trays with all types of supports, including welded steel supports. While higher damping values may be justifiable on a case-by-case basis, DCD Figure 3.7-36 does not distinguish between different types of supports, which is a key parameter in determining the cable tray/support system damping response. In order to complete its review of DCD Section 3.7.1.2, the staff requests that the applicant submit the following additional information related to SSE damping values:*

*(1) Identify whether friction-bolted steel structures are employed in the ESBWR design, and if used, identify and justify the SSE damping value used in the design basis analyses.*

*(2) Provide a detailed technical basis for the applicability of DCD Figure 3.7-36 to all types of cable tray supports, or as an alternative, describe the types of cable tray supports that are applicable to the ESBWR design; define the damping value appropriate for each type of support; and provide the technical basis for the specified damping value.*

*(3) Define and provide technical justification for cable tray damping values when there are physical restraints to free cable motion (e.g., sprayed-on fire retardant material).*

**GE Response**

(1) The damping value for friction-bolted steel structures is 4%.

(2) The damping values for cable tray is reduced to a maximum of 15%.

(3) If spray-on fire retardants that restrain free cable motion are used, the maximum damping would be limited to 7% for cable trays on welded steel supports and 10% for cable trays on bolted steel supports.

Markups of DCD Tier 2 Table 3.7-1 and DCD Tier 2 Section 3.7.1.2 were provided in MFN 06-135.

**NRC RAI 3.7-13, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Provide additional basis for 15% damping with bolted supports and 10% damping with bolted supports with spray-on fire retardants for cable trays.*

**GE Response**

Based on ASCE/SEI 43-05, Table 3-2, the damping values for cable trays will be a maximum of 10%. If spray on fire retardants are used, the maximum damping would be limited to 7%.

DCD Tier 2 Table 3.7-1 will be revised in the next update as noted in the attached markup.

**NRC RAI 3.7-17**

*From the information provided in DCD Section 3.7.2.1.1, the staff cannot determine which of the methods described were actually used for the design basis seismic analyses of the building structures, or how they were implemented. Therefore, the applicant is requested to provide the following information related to DCD Section 3.7.2.1.1:*

*(1) For each building structure covered by DCD Section 3.7.2, identify the specific time history analysis method employed; describe the implementation of the method, including determination of the highest structural frequency of interest and determination/verification of an adequate integration time-step; and discuss how the analysis results were used*

*(2) If modal superposition time history analysis was employed, identify whether the alternative to the missing mass method documented in Appendix A to SRP Section 3.7.2 was used to account for the contribution of modes with frequencies above $f_{ZPA}$. If so, explain why it was used instead of the more accurate missing mass method; define the cutoff frequency; and explain how it was determined. The staff notes that the staff's position stated in Draft Regulatory Guide DG-1127 (DG-1127 was released for public comments in February 2005, and is scheduled to be published as Revision 2 of RG 1.92 in Spring 2006) does not accept this alternative procedure.*

**GE Response**

(1) The direct integration method of analysis in the time domain as described in DCD Tier 2 Section 3.7.2.1.1 is employed in the seismic analysis for the RB/FB complex and the CB. The highest structural frequency of interest is 33 Hz for generic site and 50 Hz for North Anna site in view of the frequency contents and peak spectra accelerations of the respective ground response spectrum. The integration time step $\Delta t$ is 0.002 sec for the generic site and 0.001 sec for the North Anna site in order to meet the general criteria described in DCD Tier 2 Section 3.7.2.1 for the maximum integration time step allowed. The adequacy of the selected $\Delta t$ is confirmed for solution convergence by using ½ $\Delta t$ to show no more than 10% change in response for the representative hard site. For the usage of analysis results, please see the response to RAI 3.7-6.

(2) Modal superposition time history analysis was not employed in the building seismic analyses. However, as a general criterion for the treatment of missing mass effect using the modal superposition method, the second to last paragraph in DCD Tier 2 Section 3.7.2.7 will be deleted.

A markup of DCD Tier 2 Section 3.7.2.7 was provided in MFN 06-135.

**NRC RAI 3.7-17, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Develop a roadmap table to identify the analysis method, model utilized, computer code used and the use of the analysis output.*

**GE Response**

DCD Tier 2 Table 3.7-3 has been developed to identify the analysis method, model utilized, computer code used and the use of the analysis output. DCD Tier 2 Section 3.7.2 will be revised to include reference to this new DCD Tier 2 Table 3.7-3.

DCD Tier 2 Section 3.7.2 will be revised and DCD Tier 2 Table 3.7-3 will be added in the next update as noted in the attached markups.

**NRC RAI 3.7-18**

*From the information provided in DCD Section 3.7.2.1.2, the staff cannot determine whether response spectrum methods were actually used for the design basis seismic analyses of the building structures. Therefore, the staff requests that the applicant identify, for each building structure covered by DCD Section 3.7.2, whether the response spectrum analysis method was employed; describe the implementation of the analysis methods, including the method used to account for the contribution of modes with frequencies above* $f_{ZPA}$*; and discuss how the analysis results were used.*

**GE Response**

Response spectrum methods were not used for the design basis seismic analyses of the building structures documented in DCD Tier 2 Section 3.A.

**NRC RAI 3.7-18, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Clearly state in the DCD that the response spectrum method is not used for seismic analysis of primary building structures.*

**GE Response**

DCD Tier 2 Section 3.7.2.1.2 will be revised as recommended by the NRC in the next update as noted in the attached markup.

**NRC RAI 3.7-19**

*From the information provided in DCD Section 3.7.2.1.3, the staff cannot determine whether the static coefficient method was actually used for the design basis seismic analyses of the building structures. Therefore, the staff requests that the applicant identify, for each building structure covered by DCD Section 3.7.2, whether the static coefficient method was employed; describe the implementation of this method and the technical basis for its use; and discuss how the results were used.*

**GE Response**

Static coefficient method was not used for the design basis seismic analyses of the building structures documented in DCD Tier 2 Section 3.A.

**NRC RAI 3.7-19, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Clearly state in the DCD that the static coefficient method is not used for seismic analysis of primary building structures.*

**GE Response**

DCD Tier 2 Section 3.7.2.1.3 will be revised as recommended by the NRC in the next update as noted in the attached markup.

**NRC RAI 3.7-21**

*The staff requests that the applicant describe in detail in the DCD how it has implemented the general criteria contained in the third paragraph of DCD Section 3.7.2.3 (i.e., rotary inertia may be neglected since its contribution to the total kinetic energy of the system is small; two- or one-dimensional models may be used if the directional coupling effect is negligible; structures are generally designed to keep eccentricities as small as practical to minimize lateral/torsional coupling and torsional response) in the seismic design/analysis of the primary structural systems covered by DCD Section 3.7.2.*

**GE Response**

As described in DCD Tier 2 Section 3A.7, rotary inertia, torsional degrees of freedom and eccentricities are explicitly considered in the three-dimensional stick model of the primary building structures.

Rotary inertia of the RPV & internals are neglected because its contributions to both the total plant response and the RPV & internals response is small. The small response contributions follow from the fact that the physical geometry of the RPV & internals is axisymmetrical and is modeled as an axisymmetric, mathematical, center-line, beam-element model. Furthermore, the RPV direct support (the RPV, Pedestal) is also an axisymmetric structure and keeps the eccentricities about the vertical, center-line axis as small as practical to minimize lateral/torsional coupling and torsional response. In addition, both the seismic, free-field excitation and the non-seismic suppression pool hydrodynamic loads are characterized by essentially zero rotational components about the model vertical, center-line axis. Consequently, the RPV & internals torsional degrees-of-freedom (DOFs) are not excited by the seismic and the non-seismic suppression pool hydrodynamic loads. Therefore, the RPV & internals torsional rotary inertia can be neglected in the analytical models.

The RPV& internals rotary inertia about each of two, horizontal, orthogonal axes are also neglected in the analytical models. Sensitivity studies completed during the initial development of GE Boiling Water Reactor (BWR) RPV & internals analytical models illustrated that the model responses were essentially the same whether or not the horizontal rotary inertia components were included. This is due to the fact that the natural frequencies of the pure rotational modes tended to be well above the Zero Period Acceleration (ZPA) frequencies of both the seismic and non-seismic excitations. Consequently, the pure rotational modes contributed essentially zero to the overall response of both the RPV & internals as well as those of the primary structure.

**NRC RAI 3.7-21, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*DCD Figure 3A.7-4 does not show eccentricities of individual sticks.*

**GE Response**

DCD Tier 2 Figure 3A.7-4 will be revised to refer to DCD Tier 2 Figures 3A.7-1 through 3A.7-3 for eccentricities in the next update as noted in the attached markup.

## NRC RAI 3.7-22

*The second sentence in the second paragraph on page 3.7-10 (DCD Section 3.7.2.3) states that the mass properties in the model include all contributions expected to be present at the time of dynamic excitation, such as dead weight, fluid weight, attached piping and equipment weight, and appropriate part of the live load. For the modeling of live load, the staff requests the applicant to describe, in the DCD, which part and the amount of live and snow loads that are included in the seismic models. (The staff position is that 25% of the floor live load or 75% of the roof snow load, whichever is applicable, should be included as mass in the global seismic models.)*

## GE Response

Masses in the seismic model included 25% of the live load and 100% of the roof snow load. DCD Tier 2 Section 3.7.2.3, 4th paragraph and DCD Tier 2 Section 3A.7.1, 5th paragraph will be revised to clarify the amount of live and snow loads included in the seismic models.

Markups of DCD Tier 2 Sections 3.7.2.3 and 3A.7.1 were provided in MFN 06-135.

**NRC RAI 3.7-22, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Remove the inconsistency in the DCD with regards to one listing for 75% and the other for 100% of snow load.*

**GE Response**

DCD Tier 2 Section 3.7.2.3 will be revised by adding the phrase, "For design, 100% of roof snow load is used." in the next update as noted in the attached markup.

**NRC RAI 3.7-36**

*In DCD Appendix 3A, Tables 3A.7-1 through 3A.7-14, the applicant presented the eigenvalue analysis results. Based on the data presented, it appears that the highest modal frequencies considered in the modal time history analyses of the RB/FB are in the range of 10.83 Hz (soft soil) to 11.89 Hz (hard rock). For the CB, it appears that the highest modal frequency considered in the modal time history analyses is 29.10 Hz. The staff requests the applicant include the following information in the DCD:*

*(a) Discuss whether only the modes listed in the cited tables were included in the modal time history analyses. If not, then identify the additional modes included in each time history analysis and provide the basis for their inclusion. If yes, then identify the modes excluded from each time history analysis, up to $f_{ZPA}$ of the spectrum, and provide the basis for their exclusion.*

*(b) Discuss how the missing mass (modal mass corresponding to modes with frequencies higher than the analysis cut-off frequency) was included in the seismic response analyses. The staff notes that the 10% criteria stated on page 3.7-10 of the DCD is no longer considered acceptable to the staff (RAI 3.7-17 provides the basis for not accepting the 10% criteria).*

**GE Response**

(a) As stated in the response to RAI 3.7-17, modal superposition time history analysis was not employed. The direct integration method in the time domain is employed for the seismic analyses. For clarification purposes, a footnote "Modal information shown is not used in the response analysis performed by the direct integration method" will be added to Tables 3A.7-1 through 3A.7-14.

(b) Please see the response to RAI 3.7-17.

Markups of DCD Tier 2 Tables 3A.7-1 through 3A.7-14 were provided in MFN 06-135.

**NRC RAI 3.7-36, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*(1) GE should use 0.005 second time steps for generation of artificial time history of 40 seconds in duration to adequately match the broad envelope spectrum. The SASSI 2000 program used by GE is limited to 4096 input steps.*

*(2) GE to revise RB/FB stick model to include coupling in the vertical direction between the RB and RCCV.*

**GE Response**

(1) The artificial time histories compatible with single envelope target spectrum have been developed for 40 second duration with 0.005 second time steps. See the DCD Tier 2 Section 3.7.1 markup associated with the response to RAI 3.7-30 for details. GE has increased SASSI 2000 capability to handle 8192 input steps.

(2) The RB/FB stick model has been revised to include coupling in the vertical direction between the RB and RCCV. The revised model was provided to the NRC in MFN 06-278.

No DCD change will be made in response to this RAI Supplement.

**NRC RAI 3.7-40**

*In DCD Section 3.7.2.5, the applicant stated that direct spectra generation, without resorting to time history, is an acceptable alternative method for developing floor response spectra. The staff notes that application of the direct spectra generation method will require a detailed staff review of the technical basis and sample calculations that demonstrate results equivalent to using time history analysis. Therefore, the staff requests the applicant to (1) identify the specific applications of the direct spectra generation method in the ESBWR design/analysis; (2) describe the methodology used to confirm equivalency to the time history analysis method; and (3) submit numerical results of the comparative analyses.*

**GE Response**

The direct spectra generation methodology is not applied to the ESBWR primary structure models to generate in-structure Floor Response Spectra (FRS). However for ESBWR application, the methodology will be applied to generate in-equipment Required Response Spectra (RRS) in subsystems such as piping systems, equipment control panels, local racks, etc.

The GE Nuclear Energy developed direct spectra generation methodology is an Independent Support Motion (ISM), response spectrum methodology for generation of in-structure response spectra. It is based on stochastic calculus and statistical theory. The response spectra spectral accelerations are directly calculated based on the subsystem Eigen Data Set (obtained from the subsystem eigen analysis) and the components of the independent support motion response spectra, which excite the subsystem.

Numerical results, including response spectrum plots, of the comparative analyses considered in the verification of the ERSIN computer code are provided in the attachment.

**NRC RAI 3.7-40, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Revise DCD Section 3D.4.6.1 to include the components that will be analyzed using ERSIN computer code.*

**GE Response**

DCD Tier 2 Section 3D.4.6.1 will be revised to include components to be analyzed by ERSIN computer code in the next update as noted in the attached markup.

**NRC RAI 3.7-47**

*In DCD 3.7.2.13, the applicant presented several methods to develop composite modal damping when an SSC consists of structural elements with different damping properties. The applicant stated that for use in modal superposition (modal time history or response spectrum) analyses, the composite modal damping ratio can be obtained based on either stiffness-weighting or mass-weighting. The composite modal damping calculated by either method is limited to 20%. Additional approaches applicable to frequency domain analysis and direct integration time history analysis are also presented.*

*The staff requests the applicant to identify which of the methods described in DCD Section 3.7.2.13 were actually used in the design basis seismic analyses of the building structures (RB/FB and CB). Describe the specific applications of each method.*

**GE Response**

See the response to RAI 3.7-46.

DCD Tier 2 Section 3.7.2.13 will be revised to identify specific applications.

A markup of DCD Tier 2 Section 3.7.2.13 was provided in MFN 06-135.

**NRC RAI 3.7-47, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Revise DCD Section 3.7.2.13 markup provided to further clarify which method was used to treat damping ratios in the seismic analysis. In addition, clarify the alternate method associated with Equation 3.7-18 in DCD Section 3.7.2.13.*

**GE Response**

DCD Tier 2 Section 3.7.2.13 will be clarified in the next update as noted in the attached markup.

## NRC RAI 3.7-48

*DCD Section 3.7.2.14 describes the theory and analysis method for calculating the seismic Category I structure overturning moments. As a result of its review, the staff requests the applicant provide the following additional information:*

*In DCD Section 3.7.2.14, the applicant described the use of an energy method to evaluate the stability of structures against seismically induced overturning moments. The applicant is requested to provide a more detailed description of the analysis method, including an explanation of how the energy components for the embedment ($W_p$) and buoyancy ($W_b$) are determined, and the technical justification for the two equations given for the velocity terms ($V_h$ and $V_v$).*

## GE Response

The analysis method to evaluate the stability of structures against seismically induced overturning moments is based on the energy method shown in the following reference.

> BC-TOP-4-A, Rev.3, *Seismic Analyses of Structures and Equipment for Nuclear Power Plants,* November 1974, Bechtel Power Corporation

Energy components for the embedment ($W_p$) is illustrated in Figure 3.7-48 (1).

Let $d$ be the depth of embedment and $d'$ be the submerged depth in case the ground water table is above the elevation of the base. The structure is assumed to rotate about the toe edge $R$ (or $L$) for the overturning evaluation. To simplify the analysis for practical purposes, only the passive soil pressure developed on the toe-side is considered, and the wall frictions and the rather complicated actions of the soil on the other side of the structure are neglected. The passive pressure diagram conventionally constructed would be modified to be consistent with the assumption that the structure rotates about the edge $R$. Granular and free-draining soil conditions are also assumed. Figures 3.7-48 (1) (a) to (c) show the resultant idealized pressure diagram for different elevations of the ground water table when it is above the base (i.e., $d' > 0$). In these figures, the control parameter $P_{dry}$ is given by:

$$P_{dry} = k_P \gamma_{soil} d \qquad (1)$$

and the parameter $P_{sub}$ (for $d' > 0$) is given by:

$$P_{sub} = P_{dry} - d'\gamma_{water} \qquad (2)$$

where $k_P$, $\gamma_{soil}$ and $\gamma_{waterl}$ are the coefficients for passive soil pressure, the unit weight of soil and the unit weight of water, respectively.

For the structure to reach the overturning position, the additional work required to be done against the side soil is, according to Figure 3.7-48 (1) (a):

$$W_P = \int_0^d P(z)bz\tan\theta dz = b\tan\theta\int_0^d P(z)zdz \qquad (3)$$

in which *p(z)* is the idealized passive soil pressure at the elevation *z* above the base, $\theta$ is the angle of rotation at the overturning position, and b is the effective length of the structure normal to the plane of rotation. The effective length *b* is the structural dimension normal to the plane of rotation for rectangular structures, and 0.8 of the diameter for cylindrical structures. For the case that the ground water table is below the base, Eq. (3) gives:

$$W_{P(d'\le 0)} = \frac{1}{8}P_{dry}bd^2\tan\theta \qquad (4)$$

and for the extreme case that the water table is at the ground surface:

$$W_{P(d'=d)} = \frac{1}{8}P_{sub}bd^2\tan\theta \qquad (5)$$

<u>Energy components for the buoyancy ($W_b$)</u> is illustrated in Figure 3.7-48 (2).

When the ground water table is above the base ($d'$>0), the buoyant force has the effect of increasing the overturning potential of the structure. Such an effect would be appreciable when the submerged depth, *d'*, is appreciable. It is accounted for in the analysis by subtracting from $E_0$ the work done by buoyant force.

The buoyant force acts at the centroid of the volume of the water displaced by the submerged portion of the structure, and its magnitude varies from position to position during the overturning process. At any position before overturning takes place, let the centroid of the displaced volume of water be located at a height of *z* above the elevation of the edge *R* and let the buoyant force be *B(z)*. Denoted by $W_b$, the work done by the buoyant force is equal to:

$$W_b = \int_{z_a}^{z_b} B(z)dz \qquad (6)$$

in which, according to Figure 3.7-48 (2), $z_a$ and $z_b$ are the height of the centroid of buoyant force above the edge *R* for the equilibrium position (a) and the tipping over position (b) respectively. Note that is equal to $d'/2$. For practical purposes, Eq. (6) is approximated by:

$$W_b = (z_b + z_a)[B(z_b) - B(z_a)]/2 + B(z_a)(z_b - z_a) \qquad (7)$$

The reason for the use of the two equations given for the velocity terms ($V_h$ and $V_v$) is that the two expressions in DCD Tier 2 Equation 3.7-21 are the SRSS method of combination

to obtain the maximum value of total velocity response in view of non-simultaneous occurrence of the peak values of ground velocity and relative velocity response.

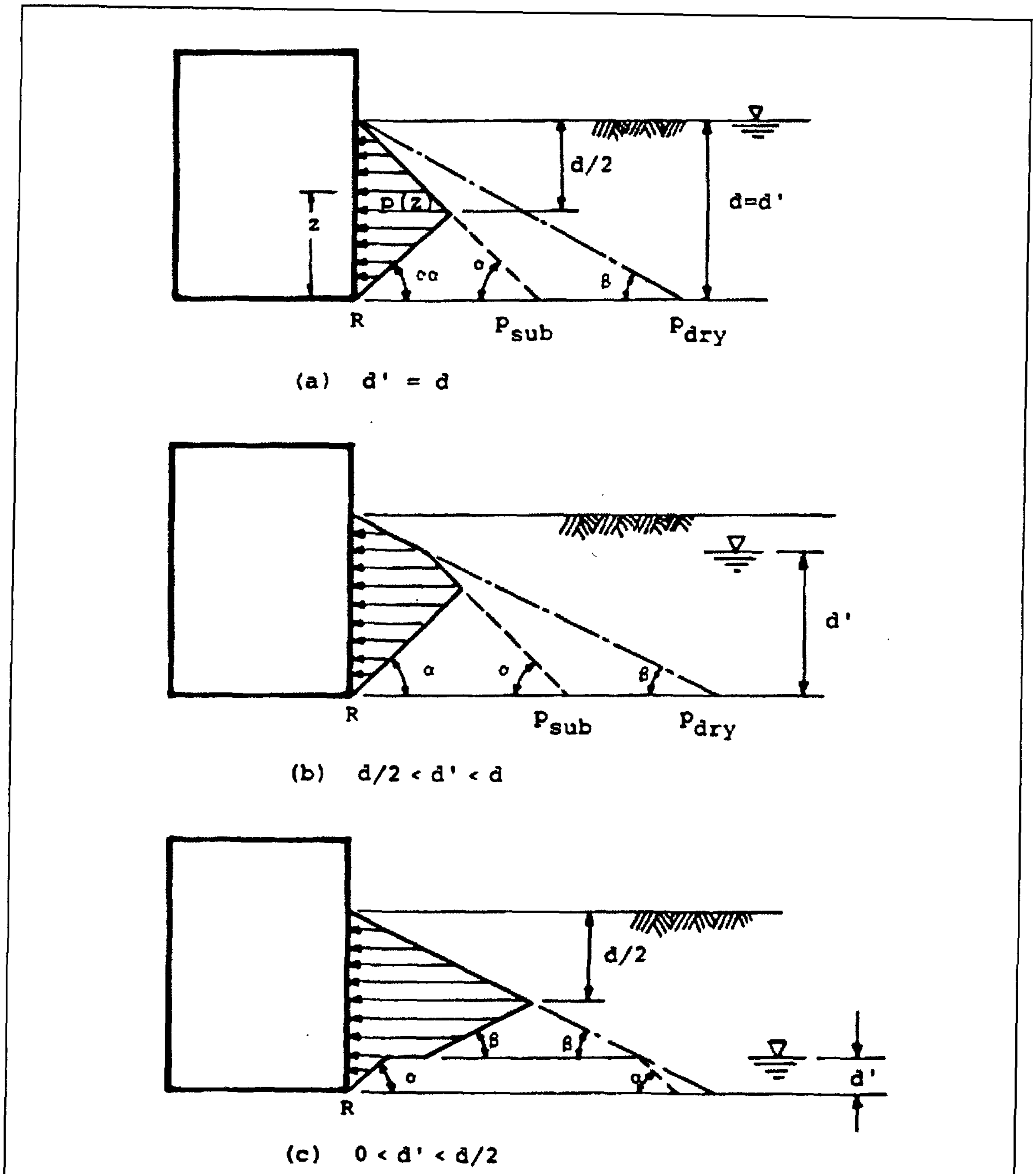



**Figure 3.7-48 (1) Passive Soil pressure for Energy Components of Embedment**

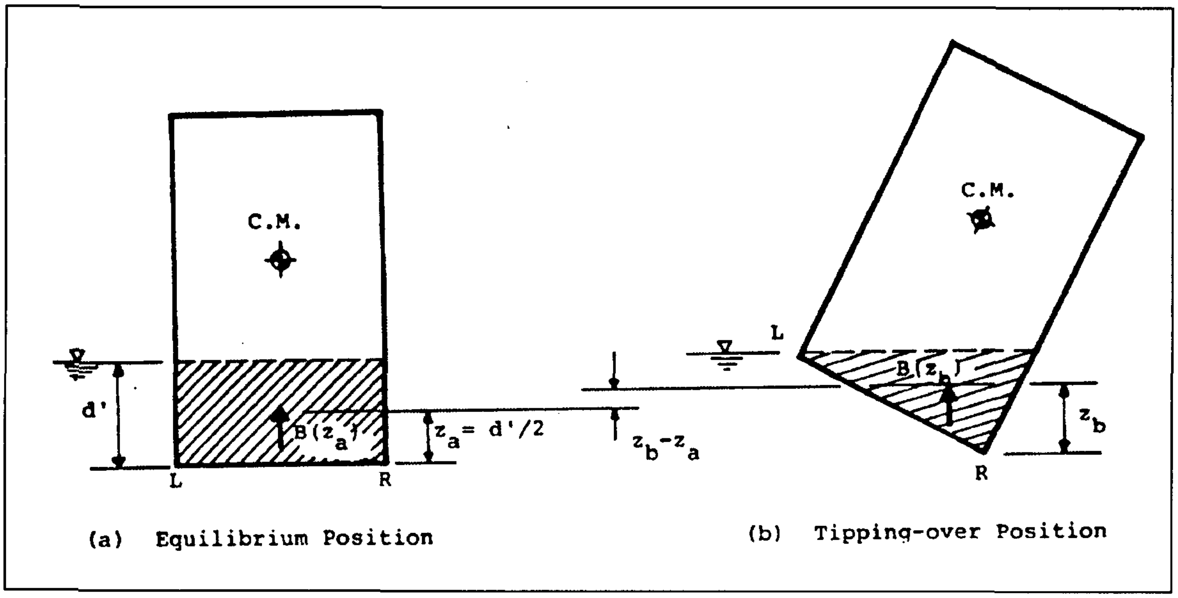



**Figure 3.7-48 (2) Energy Components for Buoyancy**

**NRC RAI 3.7-48, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

1. *Provide the corrected Equation 4-17 found in BC-TOP-4-A, Rev.3, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, November 1974, Bechtel Power Corporation.*
2. *Provide the technical basis for using the SRSS method to combine the contribution from peak values of ground velocity and relative velocity.*

**GE Response**

1. GE found from its independent derivation that there is an error in Equation 4-17 in the above document. It should be corrected as follows:

   (Original Equation 4-17) $W_b = (z_b + z_a)[B(z_b) - B(z_a)]/2 + B(z_a)(z_b - z_a)$

   (Corrected Equation) $W_b = (z_b - z_a)[B(z_b) - B(z_a)]/2 + B(z_a)(z_b - z_a)$

2. The peak values of the horizontal ground velocity $(V_h)_g$ and the relative lateral velocity $(V_x)_i$ do not occur simultaneously. Similarly, the peak values of the vertical ground velocity $(V_v)_g$ and the relative vertical velocity $(V_z)_i$ do not occur simultaneously. Therefore, they are combined by the SRSS method as shown in DCD Tier 2 Equation 3.7-21.

No DCD change will be made in response to this RAI Supplement.

**NRC RAI 3.7-49**

*The applicant is requested to provide the following information needed for the staff to perform its confirmatory analyses:*

1. *Detailed finite element (FE) RB/FB model (including figures showing mesh plots, node numbering, etc.) used for the development of the lumped-mass stick model.*
2. *Detailed fixed-base (fixed at the top of the foundation mat) lumped-mass stick model used in GE's SSI analyses.*
3. *Large-size structural design drawings of the RB/FB. Specifically, drawings showing the detailed foundation mat and embedded side walls are needed.*
4. *Soil information used to develop soil springs and soil damping for the SSI analyses of the RB/FB supported by the soft soil condition.*
5. *Description of the computer code "DAC3N" used by GE for the SSI analyses.*
6. *Input ground motion time history text files in digitized form.*
7. *Description of the SSI analytical formulation and digitized response computation results.*

**GE Response**

1. As stated in the response to RAI 3.7-6, a finite element model was not used for the development of the lumped-mass stick model.

2. Detailed fixed-base (fixed at the top of the foundation mat) lumped-mass stick model used in GE's SSI analyses are shown in Table 3.7-49 (1) through (14) and were provided in MFN 06-135.

3. Please see the response to RAI 3.7-4.

4. Soil information is shown in DCD Tier 2 Table 3A.3-1.

5. Computer code "DAC3N" is described in DCD Tier 2 Appendix 3C.

6. The digitized data of input ground motion time histories compatible to RG 1.60 were provided in Attachment 3.7-49-A1 and in electronic format in a CD transmitted by MFN 06-135.

7. The SSI analytical formulation is described as follows. The digitized response computation results of RB/FB floor response spectra shown in DCD Tier 2 Figures 3A.8-1 through 3A.8-3 for the fixed-base case were provided in Attachment 3.7-49-A2 and in electronic format in a CD transmitted by MFN 06-135.

   As stated in the response to RAI 3.7-17, the SSI analyses for the RB/FB and CB were performed by the direct integration method in the time domain. The response

of a multi-degree-of-freedom linear system subjected to external forces and/or uniform support excitations is represented by the differential equations of motion in the matrix form in DCD Tier 2 Equation (3.7-1).

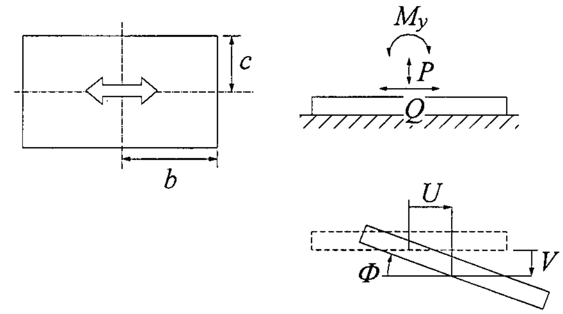
The viscous damping matrix consists of structure damping and soil radiation damping. As stated in the response to RAI 3.7-46, the structure damping matrix is generated using the DCD Tier 2 Equations (3.7-14) and (3.7-17).

As stated in DCD Tier 2 Section 3A.5, the soil is modeled with sway-rocking springs. The base spring is evaluated from vibration admittance theory, based on three dimensional wave propagation theory for uniform half space soil. The assumptions used for the evaluation are as follows.

- Uniform half space soil
- Rectangular shape foundation
- Uniform stress distribution for horizontal and vertical spring
- Triangle stress distribution for rocking and torsional spring
- Evaluation by load-weighted average displacement

The base spring value is represented by the following dynamic ground compliance in frequency domain.

$\nu$ : Poisson's ratio
$\mu$ : Shear modulus
$V_S$ : Shear wave velocity
$h$ : Damping factor
$i$ : Imaginary unit
$t$ : Time
$\omega$ : Cyclic frequency
$\lambda = c / b$
$a_0 = \omega b / V_S$
$n^2 = (1-2\nu)/2(1-\nu)$




**Vertical**

$$\frac{V}{P \cdot e^{i\omega t}} = \frac{1-\nu}{\mu \cdot \pi \cdot b \cdot (1+2hi)} \cdot A_V \cdot \left\{ 1 - \frac{a_0}{A_V} \cdot I_{RV} \right\} \tag{1}$$

where,

$$I_{RV} = \int_0^{\infty} \int_0^{\pi/2} \left\{ \frac{\xi \cdot \sqrt{\xi^2 - n^2/(1+2hi)}}{F(\xi)} + (1+2hi) \cdot (1-\nu) \right\} \cdot \{S(a_0, \xi, \theta)\}^2 \, d\theta d\xi$$

$$A_V = \frac{1}{6}\left\{3\log\left(\frac{\sqrt{\lambda^2+1}+1}{\lambda}\right)+\frac{3}{\lambda}\log\left(\sqrt{\lambda^2+1}+\lambda\right)-\left(\sqrt{\lambda^2+1}-\lambda\right)+\frac{1-\sqrt{\lambda^2+1}}{\lambda^2}\right\}$$

$$F(\xi)=\left\{2\xi^2-1/(1+2hi)\right\}^2-4\xi^2\sqrt{\xi^2-n^2/(1+2hi)}\sqrt{\xi^2-1/(1+2hi)}$$

$$S(a_0,\xi,\theta)=\frac{\sin(a_0\cdot\xi\cdot\cos\theta)\cdot\sin(\lambda\cdot a_0\cdot\xi\cdot\sin\theta)}{a_0\cdot\xi\cdot\cos\theta\cdot\lambda\cdot a_0\cdot\xi\cdot\sin\theta}$$

**Horizontal**

$$\frac{U}{Q\cdot e^{i\omega t}}=\frac{1}{\mu\cdot\pi\cdot b\cdot(1+2hi)}\left[\left\{A_{H1}+(1-\nu)\cdot A_{H2}\right\}+\frac{a_0}{\pi}\left\{I_{H1}-I_{H2}\right\}\right] \tag{2}$$

where,

$$I_{H1}=\int_0^{\infty}\int_0^{\pi/2}\left\{\frac{\xi}{\sqrt{\xi^2-1/(1+2hi)}}-1\right\}\cdot\left\{S(a_0,\xi,\theta)\cdot\sin\theta\right\}^2 d\theta d\xi$$

$$I_{H2}=\int_0^{\infty}\int_0^{\pi/2}\left\{\frac{\xi\cdot\sqrt{\xi^2-1/(1+2hi)}}{(1+2hi)\cdot F(\xi)}+(1-\nu)\right\}\cdot\left\{S(a_0,\xi,\theta)\cdot\cos\theta\right\}^2 d\theta d\xi$$

$$A_{H1}=\frac{1}{6}\left\{3\log\left(\frac{\sqrt{\lambda^2+1}+1}{\lambda}\right)-\sqrt{\lambda^2+1}-\frac{1}{\lambda^2}+2\lambda-\frac{\lambda^2}{\sqrt{\lambda^2+1}}+\frac{1}{\lambda^2\cdot\sqrt{\lambda^2+1}}\right\}$$

$$A_{H2}=\frac{1}{6}\left\{\frac{3}{\lambda}\log\left(\sqrt{\lambda^2+1}+\lambda\right)-\lambda+\frac{\lambda^2}{\sqrt{\lambda^2+1}}-\frac{1}{\lambda^2\cdot\sqrt{\lambda^2+1}}-\frac{\sqrt{\lambda^2+1}}{\lambda^2}+\frac{2}{\lambda^2}\right\}$$

**Rotational**

$$\frac{\Phi}{M_y\cdot e^{i\omega t}}=\frac{1-\nu}{\mu\cdot\pi\cdot b^3\cdot(1+2hi)}\cdot A_R\cdot\left\{1-\frac{9a_0}{\pi\cdot(1-\nu)\cdot(1+2hi)\cdot A_R}\cdot I_{RR}\right\} \tag{3}$$

where,

$$I_{RR} = \int_0^{\infty}\int_0^{\pi/2}\left\{\frac{\xi\cdot\sqrt{\xi^2-\pi^2/(1+2hi)}}{F(\xi)}+(1+2hi)\cdot(1-\nu)\right\}\cdot\left\{\frac{\sin(\lambda\cdot a_0\cdot\xi\cdot\sin\theta)}{a_0\cdot\xi\cdot\cos\theta\cdot\lambda\cdot a_0\cdot\xi\cdot\sin\theta}\cdot N(a_0,\xi,\cos\theta)\right\}^2 d\theta d\xi$$

$$N(a_0,\xi,\cos\theta)=\frac{\sin(a_0\cdot\xi\cdot\cos\theta)}{a_0\cdot\xi\cdot\cos\theta}-\cos(a_0\cdot\xi\cdot\cos\theta)$$

$$A_R=\frac{3}{10}\left[5\cdot\left\{log\left(\frac{\sqrt{\lambda^2+1}+1}{\lambda}\right)-\left(\sqrt{\lambda^2+1}-1\right)\right\}+\frac{\sqrt{\lambda^2+1}-1}{\lambda^2}+\frac{\sqrt{\lambda^2+1}}{3}\left(1-2\lambda^2\right)+\frac{2}{3}\lambda^3\right]$$

As shown in Figure 3.7-49(1), the base spring value derived from the vibration admittance theory is a function of frequency $\omega$, and can be represented by complex stiffness ${}_RK(\omega) + i_IK(\omega)$ (where ${}_RK$ is the real number portion, and ${}_IK$ is the imaginary portion). However, since the expression (${}_RK(\omega) + i_IK(\omega)$) is complicated to be used directly in response analysis, the spring values are simplified and replaced with frequency independent soil spring and damping coefficient, respectively, for the time history analysis solved in time domain. The approximate method is described below.

The horizontal and rotational components of the soil springs ($\overline{K}_S, \overline{K}_R$), are represented by the static theoretical solutions of the elastic wave theory with frequency $\omega = 0$.

The damping constants ($h_{S1}$, $h_{R1}$) of the horizontal and rotational components of the soil springs corresponding to the fundamental frequency ($\omega_1$) of the soil/building coupled system are calculated using Equation (4).

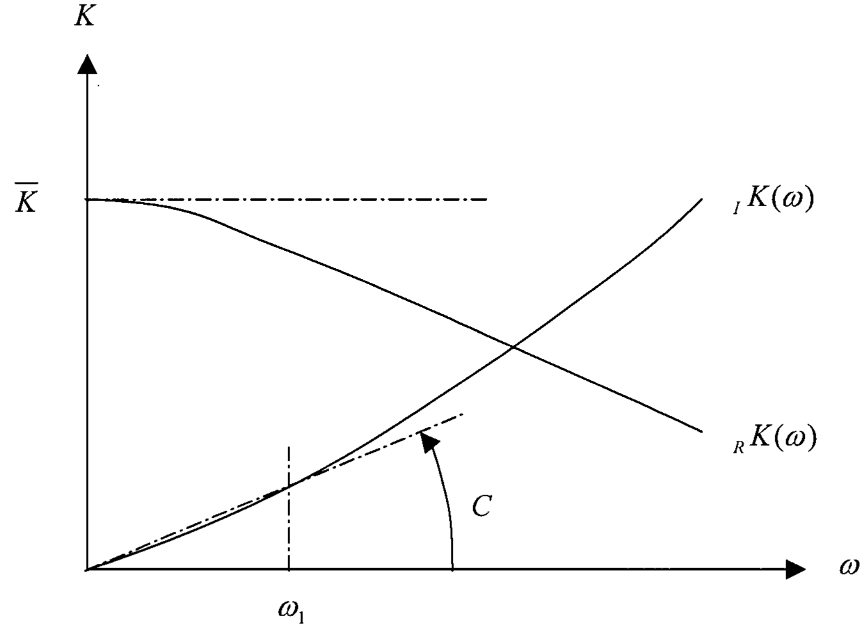
$$\begin{aligned} h_{S1} &= {}_IK_S(\omega_1)/2{}_RK_S(\omega_1) \\ h_{R1} &= {}_IK_R(\omega_1)/2{}_RK_R(\omega_1) \end{aligned} \qquad (4)$$

For the damping constants ($h_S$, $h_R$) of the soil spring, Equation (5) is used as a linear approximation.

$$\begin{aligned} h_S(\omega) &= \frac{h_{S1}}{\omega_1}\omega \\ h_R(\omega) &= \frac{h_{R1}}{\omega_1}\omega \end{aligned} \qquad (5)$$

Then, the viscous damping coefficient is derived using Equation (6).

$$\begin{aligned} C_S &= \frac{2h_{S1}}{\omega_1}\overline{K}_S \\ C_R &= \frac{2h_{R1}}{\omega_1}\overline{K}_R \end{aligned} \qquad (6)$$




Note: $\omega_1$ is the fundamental frequency of soil/building coupled system.

**Figure 3.7-49 (1) Approximate Method of Soil Spring**

**NRC RAI 3.7-49, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Response was acceptable; however, GE provides clarification of the response as shown below.*

**GE Response**

1. No change.
2. Detailed fixed-base (fixed at the top of the foundation mat) lumped-mass stick model used in GE's SSI analyses are provided by the following report, provided to the NRC by MFN 06-278:

   - SER-ESB-023, *Revised Reactor/Fuel Complex Building Stick Model, Rev. 1*, August 9, 2006

3. No change.
4. No change.
5. No change.
6. No change.
7. The SSI analytical formulation is described as follows. The digitized response computation results of RB/FB floor response spectra shown in DCD Tier 2 Figures 3A.8-1 through 3A.8-3 for the fixed-base case were provided in Attachment 3.7-49-A2 and in electronic format in a CD transmitted by MFN 06-135.

   As stated in the response to RAI 3.7-17, the SSI analyses for the RB/FB and CB were performed by the direct integration method in the time domain. The response of a multi-degree-of-freedom linear system subjected to external forces and/or uniform support excitations is represented by the differential equations of motion in the matrix form in DCD Tier 2 Equation (3.7-1).

   The viscous damping matrix consists of structure damping and soil radiation damping. As stated in the response to RAI 3.7-46, the structure damping matrix is generated using the DCD Tier 2 Equations (3.7-14) and (3.7-17).

   As stated in DCD Tier 2 Section 3A.5, the soil is modeled with sway-rocking springs. The base spring is evaluated from vibration admittance theory, based on three dimensional wave propagation theory for uniform half space soil. The assumptions used for the evaluation are as follows:

   - Uniform half space soil
   - Rectangular shape foundation

- Uniform stress distribution for horizontal and vertical spring
- Triangle stress distribution for rocking spring
- Evaluation by load-weighted average displacement

The detailed analytical formulation of the base spring value is described in the following report that is available for NRC audit:

- 26A6647, *Seismic Analysis of Reactor/Fuel Building Complex, Rev. 2*

As shown in Figure 3.7-49S1 (1), the base spring value derived from the vibration admittance theory is a function of frequency $\omega$, and can be represented by complex stiffness ${}_RK(\omega) + i_IK(\omega)$ (where ${}_RK$ is the real number portion, and ${}_IK$ is the imaginary portion). However, since the expression (${}_RK(\omega) + i_IK(\omega)$) is complicated to be used directly in response analysis, the spring values are simplified and replaced with frequency independent soil spring and damping coefficient, respectively, for the time history analysis solved in time domain. The method used to obtain the equivalent frequency independent soil stiffness and damping is described below:

The horizontal and rotational components of the soil springs ($\overline{K}_S, \overline{K}_R$), are represented by the static theoretical solutions of the elastic wave theory with frequency $\omega = 0$.

The damping constants ($h_{S1}$, $h_{R1}$) of the horizontal and rotational components of the soil springs corresponding to the fundamental frequency ($\omega_1$) of the soil/building coupled system are calculated using Equation (1).

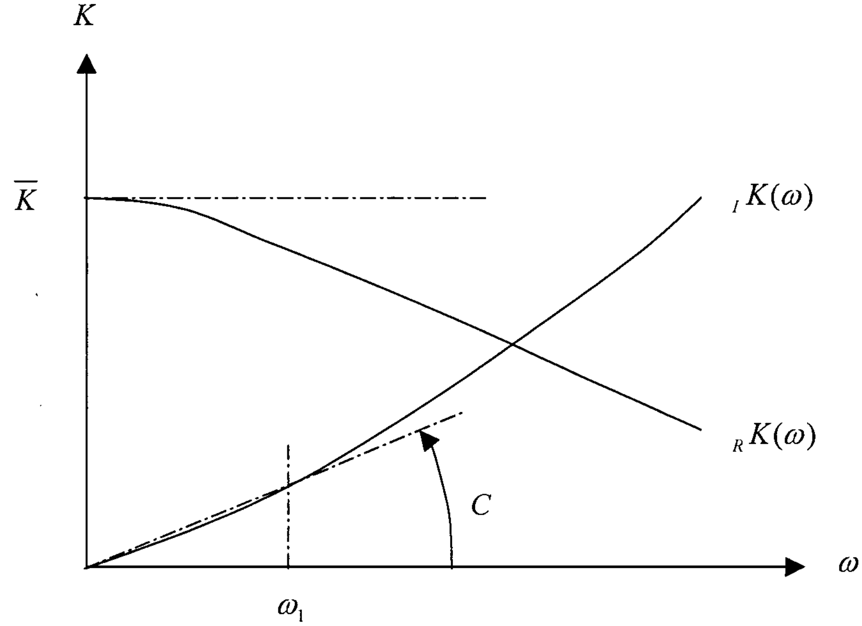
$$h_{S1} = {}_IK_S(\omega_1)/2_RK_S(\omega_1)$$
$$h_{R1} = {}_IK_R(\omega_1)/2_RK_R(\omega_1) \qquad (1)$$

For the damping constants ($h_S$, $h_R$) of the soil spring, Equation (2) is used as a linear approximation.

$$h_S(\omega) = \frac{h_{S1}}{\omega_1}\omega$$
$$h_R(\omega) = \frac{h_{R1}}{\omega_1}\omega \qquad (2)$$

Then, the viscous damping coefficient is derived using Equation (3).

$$C_S = \frac{2h_{S1}}{\omega_1}\overline{K}_S$$
$$C_R = \frac{2h_{R1}}{\omega_1}\overline{K}_R \qquad (3)$$




Note: $\omega_1$ is the fundamental frequency of soil/building coupled system.

**Figure 3.7-49S1 (1) Method for Frequency-Independent Soil Properties**

**NRC RAI 3.7-51**

*DCD Section 3.7.3.3.2 provides the approach and method for modeling the subsystems. The staff identified the need for the following additional information:*

*(a) The alternate criterion in DCD Section 3.7.3.3.2 for ensuring a sufficient number of mass degrees of freedom relies on determination of the "cutoff frequency" for the analysis; DCD Section 3.7.2.1.1 is referenced. The staff's review of DCD Section 3.7.2.1.1 noted that only the missing mass method is considered acceptable for capturing the high frequency response contribution (above $f_{zpa}$). (The staff's position for the consideration of missing mass in the seismic analysis is stated in RAI 3.7-17.) Consequently, there is no acceptable basis in DCD Section 3.7.2.1.1 for determining the "cutoff frequency." The staff requests the applicant to define "cutoff frequency", as it relates to ensuring a sufficient number of mass degrees of freedom, and explain in detail how it is determined for structures, systems, and components.*

*(b) The staff also requests the applicant to clarify its criterion in DCD Section 3.7.3.3.2 related to location of lumped masses, in order to ensure conservative dynamic loads. It appears that the goal would be to drive the natural frequency of the equipment mathematical model toward the peak of the response spectrum. However, the criterion appears to be aimed at lowering the natural frequency.*

**GE Response**

(a) The cutoff frequency for the modal superposition analysis of subsystems for seismic and non-seismic building dynamic loads is 100 Hz or the rigid frequency defined as f2 in DG-1127 (see response to RA 3.12-20). All modes with frequencies up to the cutoff frequency are included in the modal superposition and the residual rigid response due to the missing mass associated with the truncated higher frequency modes is accounted for in accordance with the methods described in DCD Tier 2 Subsection 3.7.2.7. For further clarity, DCD Tier 2 Subsection 3.7.2.1.1, 5$^{th}$ paragraph, last sentence "Alternatively, the cutoff frequency may be selected to ensure that the number of modes included is sufficient such that inclusion of all truncated modes does not result in more that a 10% increase in total response" will be deleted.

(b) The fourth bullet in DCD Tier 2 Section 3.7.3.3.2 will be revised to read as follows:

- When an equipment mass is concentrated between two supports, the concentrated mass is located at a point between the two supports where the maximum displacement of the concentrated mass will occur. This will tend to lower the natural frequencies of the equipment system model. Because the equipment fundamental frequency is typically in the higher frequency, lower amplification range of the support input motion response spectra, lowering the natural frequencies of the equipment will move them into the higher amplification region of the excitation and thereby conservatively increase the equipment response level.

Similarly, in the case of live loads (mobile) and variable support stiffness, the location of the load and the magnitude of the support stiffness are chosen to lower the system natural frequencies. Similar to above discussion, this ensures conservative dynamic responses because the lowered equipment frequencies tend to be shifted to the higher amplification range of the input motion spectra. If not, the model is adjusted to give more conservative responses.

Markups of DCD Tier 2 Sections 3.7.2.1.1 and 3.7.3.3.2 were provided in MFN 06-135.

**NRC RAI 3.7-51, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Revise DCD Section 3.7.3.3.2 to more clearly describe the approach for ensuring that a conservative response is obtained for equipment.*

**GE Response**

DCD Tier 2 Section 3.7.3.3.2 will be clarified in the next update as noted in the attached markup.

## NRC RAI 3.7-53

*In DCD Section 3.7.3.15, the applicant described the important elements to consider in the seismic analysis of above-ground tanks. However, several items in the analysis method for the aboveground tanks need to be clarified:*

*(a) DCD Section 3.7.3.15 indicates that the beneficial effects of soil-structure interaction (SSI) may be considered in this evaluation. The applicant is requested to confirm that if SSI effects are important (i.e., may lead to higher responses) then they will (not may) be considered as well. This should be included in the DCD description. In addition, provide a description or reference to an appropriate SSI method of analysis (comparable to those identified in SRP 3.7.3(II)(14)) that is used for the tank analysis.*

*(b) Describe how the damping values for the impulsive mode are determined and whether the values are in accordance with those specified in NUREG/CR-1161. If not, provide the justification for any alternative method.*

## GE Response

(a) DCD Tier 2 Section 3.7.3.15, 6th bullet, 3rd sentence will be revised to read "If the effects of soil-structure interaction results in higher response then an appropriate SSI method of analysis comparable to Reference 3.7-16 is used." In DCD Tier 2 Section 3.7.6, the following will be added: Reference 3.7-16 Brookhaven National Laboratory, BNL 52361, "Seismic Design and Evaluation Guidelines for the Department of Energy High-Level Waste Storage Tanks and Appurtenances." October 1995.

(b) The damping value for the impulsive mode is the same as the tank shell material in accordance with NUREG/CR-1161. DCD Tier 2 Section 3.7.3.15, 2nd bullet, 3rd sentence will be clarified.

Markups of DCD Tier 2 Sections 3.7.3.15 and 3.7.6 were provided in MFN 06-135.

**NRC RAI 3.7-53, Supplement 1**

**NRC Assessment Following the June 8, 2006 Audit**

*Revise the DCD to address damping when SSI effects are included in the tank analysis.*

**GE Response**

DCD Tier 2 will be revised to further clarify how the soil damping is determined and utilized in the analysis when SSI is included in the tank analysis.

DCD Tier 2 Section 3.7.3.15 will be revised in the next update as noted in the attached markup.

accordance with industry practices. Seismic Category II (C-II) items are those corresponding to positions C.2 and C.4 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 Seismic Design Parameters

As discussed in Standard Review Plan (SRP) 3.7.1, structures that are important to safety and that must withstand the effects of earthquakes are designed to the relevant requirements of GDC 2 and comply with Appendix A to 10 CFR 100 concerning natural phenomena. Standardized plants envelop the most severe earthquakes that affected a great number of sites where a nuclear plant may be located, with sufficient margin considering limited accuracy, quantity and period of time in which historical data have been accumulated. Seismic design parameters considered for ESBWR comprise two site conditions, generic sites and early site permit (ESP) sites. Three sites, North Anna (Reference 3.7-2), Clinton (Reference 3.7-3) and Grand Gulf (Reference 3.7-4) are currently in the process of ESP application to the NRC. A review of the three site conditions reveals that Clinton and Grand Gulf are bounded by the envelope of generic site and North Anna conditions. North Anna ESP site is therefore selected for further consideration in conjunction with generic sites for site enveloping seismic design of the ESBWR Standard Plant. COL Applicant will confirm that site-specific seismic design parameters do not exceed the site envelope parameters discussed in Subsection 3.7.5.1.

### *3.7.1.1 Design Ground Motion*

The ESBWR standard plant SSE design ground motion is rich in both low and high frequencies. The low-frequency ground motion follows RG 1.60 ground spectra anchored to 0.3g. The high-frequency ground motion matches the North Anna ESP site-specific spectra as representative of most severe rock sites in the Eastern US. These two ground motions are considered separately in the basic design. To verify the basic design the two separate inputs are further enveloped to form a single ground motion as the design basis ground motion for ESBWR. The single envelope design ground response spectra are shown in Figures 2.5-1 and 2.5-2 for horizontal and vertical direction, respectively. They are defined as free-field outcrop spectra at the foundation level (bottom of the base slab). Application of design ground motion at the foundation level is a conservative approach for deeply embedded foundations as compared to the compatible free-field motion deconvoluted from the free ground surface motion at the finished grade. The ESBWR Reactor Building (RB) and Control Building (CB) foundations are embedded at depth of 20 m (66 ft) and 14.9 m (49 ft), respectively. The Fuel Building (FB) shares a common foundation mat with the RB. The development of design ground motion is delineated in the following subsections.

#### 3.7.1.1.1 Low-Frequency Ground Motion

The ground response spectra for low-frequency ground motion are developed in accordance with Regulatory Guide 1.60 anchored to 0.3g and specified at the foundation level in the free field for

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 or pocket lock construction and is 4% for welded construction.

For ASME Section III, Division 1 Class 1, 2, and 3, and ASME/ANSI B31.1 piping systems, alternative damping values specified in Figure 3.7-37 may be 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 EBAS buildings). 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:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{P\} \qquad (3.7\text{-}1)$$

where,

| | | |
|---|---|---|
| [M] | = | mass matrix |
| [C] | = | damping matrix |

The timewise solution of Equation 3.7-8 can be obtained easily by using the standard normal mode solution technique. After obtaining the displacement response of the active degrees of freedom ($U_a$), Equation 3.7-7 can then be used to solve the support point reaction forces ($F_s$). Analysis can be performed using either the time history method or response spectrum method. Additional considerations associated with the ISM response spectrum method of analysis are given in Subsection 3.7.3.9.

The response spectrum method is not used for seismic response analysis of primary building structures.

#### 3.7.2.1.3 Static Coefficient Method

This is an alternative method of analysis that allows a simpler technique in return for added conservatism. This method does not require determination of natural frequencies. The response loads are determined statically by multiplying the mass value by a static coefficient equal to 1.5 times the maximum spectral acceleration at appropriate damping value of the input response spectrum. A static coefficient of 1.5 is intended to account for the effect of both multi-frequency excitation and multi-mode response for linear frame-type structures, such as members physically similar to beams and columns, which can be represented by a simple model similar to those shown to produce conservative results (References 3.7-13 and 3.7-14). A factor of less than 1.5 may be used if justified. If the fundamental frequency of the structure is known, the spectral acceleration value at this frequency can be multiplied by a factor of 1.5 to determine the response. A factor of 1.0 instead of 1.5 can be used if the component is simple enough such that it behaves essentially as a single-degree-of-freedom system. When the component is rigid, it is analyzed statically using the Zero Period Acceleration (ZPA) as input. Structures, systems, and components are considered rigid when the fundamental frequency is equal to or greater than the frequency at which the input response spectrum returns to approximately the ZPA. Relative displacements between points of support are also considered and the resulting response is combined with the response calculated using the equivalent static method. The static coefficient method is not used for primary building structures.

### *3.7.2.2 Natural Frequencies and Responses*

Natural frequencies and SSE responses of Category I buildings are presented in Appendix 3A.

### *3.7.2.3 Procedures Used for Analytical Modeling*

The mathematical model of the structural system is constructed as a stick model for seismic response analysis of primary building structures. The details of the model are determined by the complexity of the actual systems and the information required from the analysis. In constructing the primary structural system model, the following subsystem decoupling criteria are applicable:

- If $R_m < 0.01$, decoupling can be done for any $R_f$.
- If $0.01 \le R_m \le 0.1$, decoupling can be done if $R_f \le 0.8$ or $R_f \ge 1.25$.
- If $R_m > 0.1$, a subsystem model should be included in the primary system model

where $R_m$ (mass ratio) and $R_f$ (frequency ratio) are defined as:

$R_m$ = total mass of the supported subsystem/total mass of the supporting system

$R_f$ = fundamental frequency of the supported subsystem/dominant frequency of the support motion.

If the subsystem is comparatively rigid in relation to the supporting system, and also is rigidly connected to the supporting system, it is sufficient to include only the mass of the subsystem at the support point in the primary system model. On the other hand, in case of a subsystem supported by very flexible connections (e.g., pipe supported by hangers), the subsystem need not be included in the primary model. In most cases, the equipment and components, which come under the definition of subsystems, are analyzed (or tested) as a decoupled system from the primary structure and the dynamic input for the former is obtained by the analysis of the latter. One important exception to this procedure is the reactor pressure vessel (RPV), which is considered as a subsystem but is analyzed using a coupled model of the RPV and primary structure.

In general, three-dimensional models are used with six degrees of freedom assigned to each mass (node) point (i.e., three translational and three rotational). Some dynamic degrees of freedom, such as rotary inertia, may be neglected, since their contribution to the total kinetic energy of the system is small compared to the contribution from translational inertia. A two- or one-dimensional model is used if the directional coupling effect is negligible. Coupling between two horizontal motions occurs when the center of mass, the centroid, and the centroid of rigidity do not coincide. The degree of coupling depends on the amount of eccentricity and the ratio of uncoupled torsional frequency to the uncoupled lateral frequency. Structures are generally designed to keep eccentricities as small as practical to minimize lateral/torsional coupling and torsional response.

Nodal points are generally selected to coincide with the locations of large masses, such as floors or at heavy equipment supports, at all points where significant changes in physical geometry occur, and locations where the responses are of interest. The mass properties in the model include all contributions expected to be present at the time of dynamic excitation, such as dead weight, fluid weight, attached piping and equipment weight, and appropriate part (25% of floor live load or minimum 75% of roof snow load, as applicable) of the live load. For design, 100% of roof snow load is used. The hydrodynamic effects of any significant fluid mass interacting with the structure are considered in modeling of the mass properties. Masses are lumped to node points. Alternatively, the consistent mass formulation may be used. The number of masses or dynamic degrees of freedom is considered adequate when additional degrees of freedom do not result in more than a 10% increase in response. Alternatively, the number of dynamic degrees of freedom is no less than twice the number of modes below the cutoff frequency in Subsection 3.7.2.1.1. For the stick models of the primary building structures, the number of dynamic degrees of freedom is no less than twice the number of modes below 50 Hz.

The RPV, including its major internal components, is analyzed together with the primary structure using a coupled RPV and supporting structural model. The RPV model is constructed following the general modeling procedures described above for the primary structures. The RPV model includes major internal components such as the fuel assemblies, control rod (CR) guide

### *3.7.2.13 Analysis Procedure for Damping*

When the modal superposition method of analysis (either time history or response spectrum) is used for models that consist of elements with different damping properties, the composite modal damping ratio can be obtained either as stiffness-weighted:

$$\lambda_k = \frac{\{\phi\}^T[\bar{K}]\{\phi\}}{K^*} \tag{3.7-14}$$

or as mass-weighted:

$$\lambda_k = \{\phi\}^T[\bar{M}]\{\phi\} \tag{3.7-15}$$

where:

| | | |
|---|---|---|
| $\lambda_k$ | = | equivalent modal damping for the kth mode |
| $K^*$ | = | $\{\phi\}^T [K] \{\phi\}$ |
| $[K]$ | = | assembled stiffness matrix |
| $[\bar{K}]$, $[\bar{M}]$ | = | modified stiffness or mass matrix constructed from element matrices formed by the product of the damping ratio for the element and its stiffness or mass matrix |
| $\{\phi\}$ | = | kth normalized modal vector. |

The composite modal damping calculated by either Equation 3.7-14 or 3.7-15 is limited to 20%. For models that take SSI into account by the lumped soil spring approach, the method defined by Equation 3.7-14 is acceptable. For fixed base model, either Equation 3.7-14 or 3.7-15 may be used.

In the seismic response analysis of primary building structures described in Appendix 3A using the complex response method in the frequency domain, material damping is included in the formulation of the complex stiffness matrix:

$$[k_j^*] = [k_j](1 + 2i\lambda_j) \tag{3.7-16}$$

where

| | | |
|---|---|---|
| $[k_j^*]$ | = | complex stiffness matrix of element j |
| $[k_j]$ | = | stiffness matrix of element j |
| $\lambda_j$ | = | material damping ratio of element j |
| $i$ | = | $\sqrt{-1}$ . |

In the seismic response analysis of primary building structures described in Appendix 3A using the time history method solved by direct integration, the damping matrix is formed by the following procedure:

(1) First, the stiffness-weighted modal damping $\lambda_k$ is calculated in accordance with Equation 3.7-14

(2) The damping matrix that fits the relationships between the frequencies and modal damping constants above can be calculated using the following formula. (Reference 3.7-9)

$$[C]=[M][\Phi][\Lambda][\Phi]^T[M] \tag{3.7-17}$$

where,

$[M]$: mass matrix

$[\Phi]$: undamped characteristic mode matrix

$[\Lambda]$: $\begin{bmatrix} \Lambda_1 & & & & \\ & \ddots & & & \\ & & \Lambda_k & & \\ & & & \ddots & \\ & & & & \Lambda_n \end{bmatrix}$

$$\Lambda_k = \frac{2\lambda_k \omega_k}{m_k}$$

$\lambda_k$: k-th damping constant

$\omega_k$: k-th undamped circular frequency

$m_k$: k-th equivalent mass

$n$: maximum mode number

In the dynamic response analysis of containment loads described in Appendix 3F using the direct integration time history method, the damping matrix is formed by a linear combination of the mass and stiffness matrices,

$$[C]=\alpha[M]+\beta[K] \tag{3.7-18}$$

where $\alpha$ and $\beta$ are constants. They are determined to give the required damping value as a function of the circular frequency $\omega$, i.e.,

$$\lambda = \frac{\alpha}{2\omega}+\frac{\beta\omega}{2} \tag{3.7-19}$$

### *3.7.2.14 Determination of Seismic Category I Structure Overturning Moments*

When the combined effect of earthquake ground motion and structural response is strong enough, the structure undergoes a rocking motion pivoting about either edge of the base. When the amplitude of rocking motion becomes so large that the center of structural mass reaches a

The stiffness of the supporting structures is included in the analysis, unless the supporting structure can be shown to be rigid.

#### 3.7.3.3.2 Equipment

For dynamic analysis, equipment is represented by lumped-mass system, which consists of discrete masses connected by zero-mass elements. The criteria used to lump masses are as follows:

- The number of modes of a dynamic system is controlled by the number of masses used; therefore, the number of masses is chosen so that all significant modes are included. The number of masses or dynamic degrees of freedom is considered adequate when additional degrees of freedom do not result in more than a 10% increase in response. Alternatively, the number of dynamic degrees of freedom is no less than twice the number of modes below the cutoff frequency of Subsection 3.7.2.1.1.
- Mass is lumped at any point where a significant concentrated weight is located. Examples are the motor in the analysis of a pump stand, and the impeller in the analysis of a pump shaft.
- If the equipment has free-end overhang span whose flexibility is significant compared to the center span, a mass is lumped at the overhang span.
- When equipment is concentrated between two existing nodes located between two supports in a finite element model, a new node is created at that location. Alternatively, the equipment mass can be concentrated at the nearest node to either side which tends to shift the natural frequency to the higher amplification region of the input motion response spectrum. When the approximate location of the equipment mass is shifted toward the mid-span between the supports the natural frequency is lowered and when the approximate location is shifted toward either support the natural frequency is increased. Moving the natural frequencies of the equipment into the higher amplification region of the excitation thereby conservatively increases the equipment response level.

  Similarly, in the case of live loads (mobile) and variable support stiffness, the location of the load and the magnitude of the support stiffness are chosen to lower the system natural frequencies. Similar to the above discussion, this ensures conservative dynamic responses because the lowered equipment frequencies tend to be shifted to the higher amplification range of the input motion spectra. If not, the model is adjusted to give more conservative responses.

#### 3.7.3.3.3 Modeling of Special Engineered Pipe Supports

Modifications to the normal linear-elastic piping analysis methodology used with conventional pipe supports are required to calculate the loads acting on the supports and on the piping components when the special engineered supports, described in Subsection 3.9.3.7.1 (6), are used. These modifications are needed to account for greater damping of the energy absorbers and the non-linear behavior of the limit stops. If these special devices are used, the modeling and analytical methodology shall be in accordance with methodology accepted by the regulatory agency at the time of certification or at the time of application, per the discretion of the applicant.

- Relative deformations imposed by seismic waves traveling through the surrounding soil or by differential deformations between the soil and anchor points.
- Lateral earthquake pressures and ground-water effects acting on structures.

- The effects of static resistance of the surrounding soil on piping deformations or displacements, differential movements of piping anchors or equipment, and bent geometry and curvature changes, etc., are considered. When applicable, procedures using the principles of the theory of structures on elastic foundations can be used.
- When applicable, the effects caused by local soil settlements, soil arching, etc., are considered in the analysis.

### *3.7.3.14 Methods for Seismic Analysis of Seismic Category I Concrete Dams*

For Seismic Category C-I concrete dams, if applicable to the site, the seismic analysis takes into consideration the dynamic nature of forces (due to both horizontal and vertical earthquake loadings), the behavior of the dam material under earthquake loadings, soil-structure interaction effects, and nonlinear stress-strain relations for the soil. FEM is the usual analytical tool used.

### *3.7.3.15 Methods for Seismic Analysis of Above-Ground Tanks*

The seismic analysis of C-I above-ground tanks considers the following items:

- At least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration are included in the analysis. The horizontal response analysis includes at least one impulsive mode in which the response of the tank shell and roof is coupled together with the portion of the fluid contents that move in unison with the shell, and the fundamental sloshing (convective) mode.
- The fundamental natural horizontal impulsive mode of vibration of the fluid-tank system is estimated giving due consideration to the flexibility of the supporting medium and to any uplifting tendencies for the tank. The rigid tank assumption is not made unless it can be justified. The horizontal impulsive-mode spectral acceleration, $S_{a1}$, is then determined using this frequency and damping value for the impulsive mode. This is the same as that for the tank shell material in accordance with NUREG/CR-1161. Alternatively, the maximum spectral acceleration corresponding to the relevant damping may be used.
- Damping values used to determine the spectral acceleration in the impulsive mode are based upon the system damping associated with the tank shell material as well as with the soil-structure interaction (SSI). The SSI system damping takes into account soil damping in the form of stiffness-weighted damping in accordance with Equation 3.7-14 or complex stiffness matrix in accordance with Equation 3.7-16.
- In determining the spectral acceleration in the horizontal convective mode, $S_{a2}$, the fluid damping ratio is 0.5% of critical damping unless a higher value can be substantiated by experimental results.
- The maximum overturning moment, $M_o$, at the base of the tank is obtained by the modal and spatial combination methods discussed in Subsections 3.7.2.7 and 3.7.2.6, respectively. The uplift tension resulting from $M_o$ is resisted either by tying the tank to the foundation with anchor bolts, etc., or by mobilizing enough fluid weight on a

## Table 3.7-1
## Damping Values for SSE Dynamic Analysis

| Components | Percent of Critical Damping |
|---|---|
| Reinforced concrete structures | 7.0 |
| Welded and friction bolted steel assemblies/structures | 4.0 |
| Bearing bolted steel assemblies/structures | 7.0 |
| Equipment | 3.0 |
| Piping systems [1] | |
| - diameter greater than 305 mm (12 in) | 3.0 |
| - diameter less than or equal to 305 mm (12 in) | 2.0 |
| RPV, skirt, shroud, chimney, and separators | 4.0 |
| Control rod guide tubes and CRD housings | 2.0 |
| Fuel assemblies | 6.0 |
| Cable Trays | 10 max [2] |
| Conduits | 7.0 |
| HVAC ductwork | |
| - companion angle | 7.0 |
| - pocket lock | 7.0 |
| - welded | 4.0 |

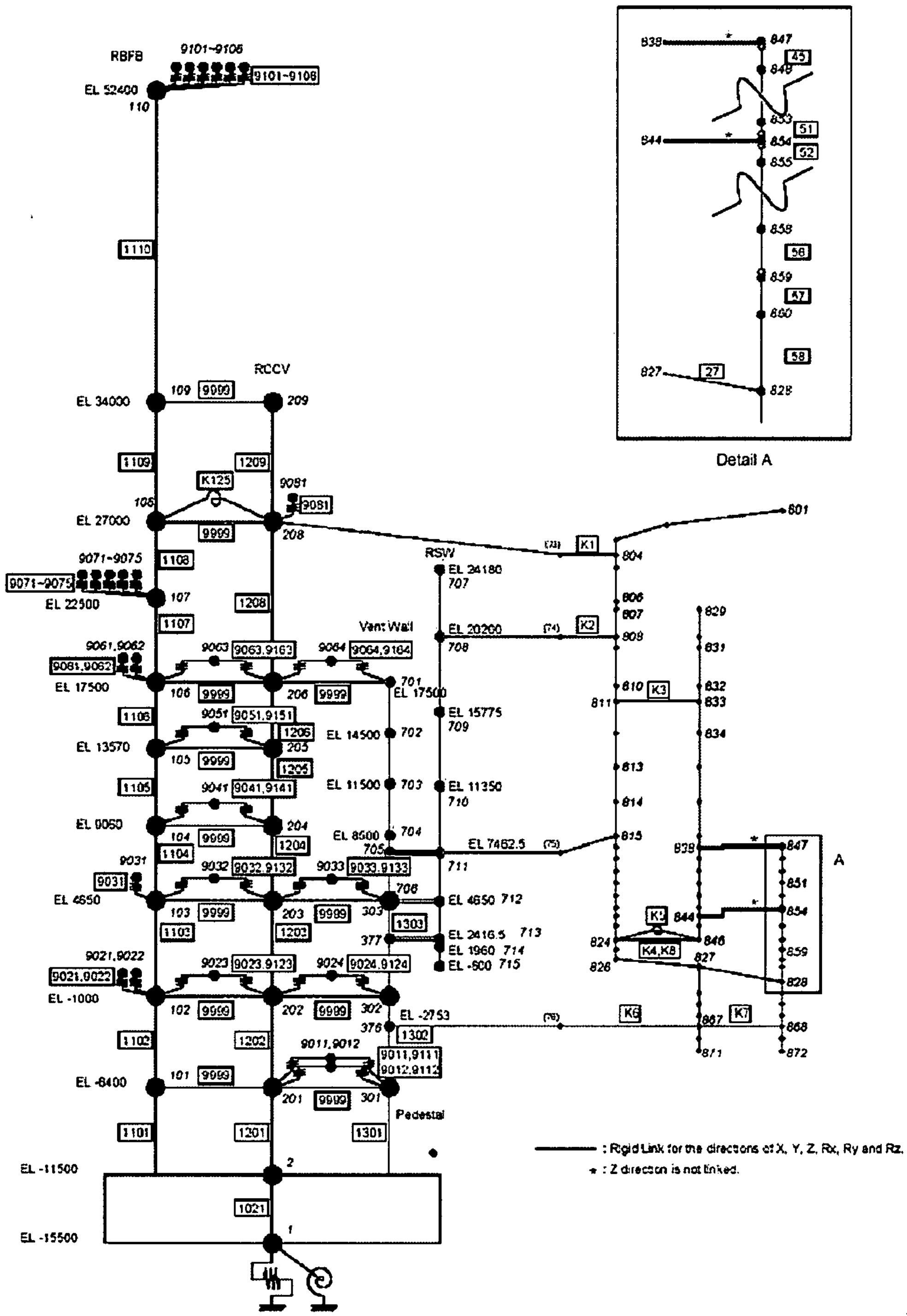
[1] See Figure 3.7-37 for alternative damping values for response spectra analysis of ASME Section III, Division 1 Class 1, 2, and 3, and ASME/ANSI B31.1 piping systems.

[2] a. If the cables are restrained by spray-on fire protection materials, the damping is limited to 7% for cable trays on welded or bolted steel supports.
b. Maximum damping on welded steel tray systems shall be 10%.
c. Cable trays shall be at least one-third full with cable ties spacing not less than 6 ft (on average), and cable tray system stability shall be assured.
d. If the condition (c) cannot be met, the cable tray shall be treated as a steel assembly.

## Table 3.7-3

## Summary of Methods of Seismic Analysis for Primary Building Structures

| Building Structure | Site Condition | SSI Model | Analysis Method | Three Components Combination | Modal Combination | Computer Program | Use of Analysis Output |
|---|---|---|---|---|---|---|---|
| Reactor Building including containment and containment internal structures | Uniform Sites | 3D lumped mass stick coupled with soil springs | Direct integration in the time domain | Algebraic Sum | n/a | DAC3N | max. forces, moments, acceleration, floor response spectra and max. relative displacements |
| Reactor Building including containment and containment internal structures | Layered Sites | 3D lumped mass stick coupled with soil finite elements | Frequency response in the frequency domain. | Algebraic Sum | n/a | SASSI | acceleration, floor response spectra and soil pressure |
| Fuel Building | Uniform Sites | Integrated with the Reactor Building models | Direct integration in the time domain | Algebraic Sum | n/a | DAC3N | max. forces, moments, acceleration, floor response spectra and max. relative displacements |
| Fuel Building | Layered Sites | Integrated with the Reactor Building models | Frequency response in the frequency domain. | Algebraic Sum | n/a | SASSI | acceleration, floor response spectra and soil pressure |
| Control Building | Uniform Sites | 3D lumped mass stick coupled with soil springs | Direct integration in the time domain | Algebraic Sum | n/a | DAC3N | max. forces, moments, acceleration, floor response spectra and max. relative displacements |
| Control Building | Layered Sites | 3D lumped mass stick coupled with soil finite elements | Frequency response in the frequency domain. | Algebraic Sum | n/a | SASSI | acceleration, floor response spectra and soil pressure |




Note: Refer to Figures 3A.7-1 through 3A.7-3 for eccentricities of individual sticks.

**Figure 3A.7-4. ESBWR RBFB Complex Seismic Model**

On the basis of geometry input data, and thermohydraulic initial conditions (pressure, temperature, fluid), the code integrates fluid equations in order to calculate time-histories of pressure, temperature, forces in the different nodes and sections of the piping network.

#### *3D.4.4.4 Subcompartment Pressurization - Contain*

The CONTAIN 2.0 code is an analysis tool for predicting the physical, chemical, and radiological conditions inside the containment and connected buildings of a nuclear reactor in the event of an accident. CONTAIN 2.0 was developed at Sandia National Laboratories under the sponsorship of the US Nuclear Regulatory Commission (USNRC) for analyzing containment phenomena under severe accident and design basis accident conditions. It is designed and has capability to predict the thermal-hydraulic response inside the containment in the event of an accident.

CONTAIN 2.0 is a highly flexible and modular code that can run both everything from quite simple to highly complex problems.

### 3D.4.5 Integral Attachment - LUGST

The computer program LUGST evaluates the stress in the pipe wall that is produced by loads applied to the integral attachments. The program is based on Welding Research Council Bulletin 198.

### 3D.4.6 Response Spectra Generation

#### *3D.4.6.1 ERSIN Computer Program*

ERSIN is a computer code used to generate response spectra for pipe-mounted and floor-mounted equipment. ERSIN provides direct generation of local or global acceleration response spectra.

Equipment Control Panels, Local Equipment Racks, Main Steam Isolation Valves (MSIVs), Safety Relief Valves (SRVs) and Hydraulic Control Units (HCUs) are some of the components that are analyzed using ERSIN computer code.

#### *3D.4.6.2 RINEX Computer Program*

RINEX is a computer code used to interpolate and extrapolate amplified response spectra used in the response spectrum method of dynamic analysis. RINEX is also used to generate response spectra with nonconstant model damping. The non-constant model damping analysis option can calculate spectral acceleration at the discrete eigenvalues of a dynamic system using either the strain energy weighted modal damping or the ASME Code Class N-411-1 damping values.

#### *3D.4.6.3 CALESPW Computer Program*

CALESPW is used to calculate the response spectra from time histories, for the degrees of freedom selected in a model. The program solves the second order differential equation through Nigam-Jennings method for an established set of frequencies.

#### *3D.4.6.4 SFT Computer Program*

SFT is a computer code used to calculate Fourier Transform and Power Density Spectrum (PDS) of an acceleration time history. The program also allows adjusting the input PDS to the PDS defined in Standard Review Plan (SRP 3.7.1) and calculating its corresponding time history.