3.7 SEISMIC DESIGN

3.7.1 SEISMIC INPUT

3.7.1.1 Design Response Spectra

3.7.1.1.1 Horizontal Design Response Spectra

The horizontal design response spectra were developed using synthetic earthquake records for a maximum acceleration of 0.10 g for the safe shutdown earthquake (SSE) and a 0.05 g for the operating basis earthquake (OBE) for two percent and five percent damping, as shown on Figures 3.7-1 through 3.7-4. In simulating the earthquakes a maximum duration of 20 seconds was used in the model, of which zero to two seconds was used for the rising period, two to seven seconds for the constant maximum acceleration period, and seven to 20 seconds for the receding period. These durations were selected based on the available data of the duration of the earthquakes in this region. The SSE spectra was normalized to a 0.05 g acceleration for OBE.

The design response spectra used in the plant design differ from the design response spectra recommended in NRC Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants, Revision I December 1973. The regulatory guide response spectra have slightly higher values in general. Use of Regulatory Guide 1.60 permits utilization of damping values indicated in Regulatory Guide 1.61, Damping Values for Seismic Design of Nuclear Power Plants, October 1973. These damping values are equal to or greater than the values utilized for Waterford 3 plant design (refer to Subsection 3.7.1.3). By utilizing lower damping values in the Waterford 3 design, as compared to the damping values of Regulatory Guide 1.61, the analysis and design of Waterford 3 compensates for any diffences.

The design response spectra used in the plant design produce an amplification factor of 3.5 at two percent damping (for OBE) in the period range 0.15 to 0.5 seconds. The use of Regulatory Guide 1.60 produces an amplification range of 3.54 to 4.25 for the above damping and period range; however, the use of higher damping value allowed by Regulatory Guide 1.61 (four percent for reinforced concrete structures in OBE) results in an amplification factor range of 2.92 to 3.50 for the above period. A similar correlation exists for the SSE. Thus the design response spectra and the damping factors used for the plant design provide an adequate and approximately equivalent basis for seismic design.

The horizontal design response spectra for the SSE and OBE are applied at the bottom of the foundation of the common mat of Nuclear Plant Island Structure, at the top of the Pleistocene formation in the free field (refer to Appendix 3.7A, Site Amplification Studies).

3.7.1.1.2 Vertical Design Response Spectra

There are no data in the region relating to horizontal and vertical acceleration for strong motion earthquakes. A vertical acceleration equal to two-thirds of the horizontal acceleration was used in developing the vertical design response spectra, which is illustrated in Figures 3.7-5 through 3.7-8.

3.7.1.2 Design Time History

A synthetic accelerogram (earthquake time history), whose spectra envelopes the design response spectra shown in Figures 3.7-1 through 3.7-8 was developed by Law Engineering Testing Company, Marietta, Georgia, using computer techniques. These techniques permitted the generation of a synthetic accelerogram to represent specified design peak acceleration, duration, variation in intensity with time and frequency content. For the horizontal component of the SSE, the design peak acceleration is 0.10 g, the duration of strong motion is 20 seconds and the frequency content is as specified by the design response spectra shown in Figures 3.7-1 through 3.7-4.

Sinusoidal waves of many frequencies are generated and superimposed to form the artificial accelerogram. The phase angles of these waves are selected in the interval 0 to 27i using a random number generator. The relative amplitudes of the sinusoidal waves are determined from a power spectral density function, $G(\omega)$, which expresses the relative values of the energy content of each frequency. The first estimate of the shape and ordinates of the function $G_1(\omega)$ is derived from the desired pseudo-velocity design spectrum, S_V. The response of a single degree of freedom system to this artificial motion is then evaluated and the ordinates S_{vn} are computed. The values S_{vn} normally are considerably different from the desired values S_V of the design response spectrum. A new input spectral density function $G_2(\omega)$, is then obtained by multiplying the initial choice $G_1(\omega)$, by the ratio S_V/S_{vn}, and used to modify the artificial accelerogram. The process is then continued until the value of S_{vn} (obtained after n adjustments) are sufficiently close to the desired values S_v of the design response spectrum.

The artificial accelerogram, whose spectrum envelopes the horizontal design spectrum for the OBE at the top of Pleistocene formation in the free field using damping ratios of two percent and five percent is shown in Figures 3.7-3 and 3.7-4, respectively. A comparison of the spectral values of the design response spectra and the corresponding time history response spectra was made for 100 period points over the following range:

Period Range (seconds)	No. of Periods
0.03 to 0.1	25
0.1 to 0.5	50
0.5 to 3.0	25
	100

The above procedure employed approximately equal period increments within a given period range. The following tabulation presents the number of frequencies considered, employing an equal frequency increment.

Frequency Increment (hertz)	Increment (hertz)	No. of Frequencies Used
0.2 - 3.0	0.10	37
3.0 - 3.6	0.15	7
3.6 - 5.0	0.20	10
5.0 - 8.0	0.25	14
8.0 - 15.0	0.50	16
15.0 - 18.0	1.00	3
18.0 - 22.0	2.00	4
22.0 - 34.0	3.00	9
		100

Similar design response spectra and time history spectra were made utilizing 200 computed period points within the above frequency range, which verified the above results.

3.7.1.3 Critical Damping Values

The damping ratios, expressed as percent of critical damping, which are used in the analysis of seismic Category I systems and components are presented in Table 3.7-1. These damping values both for the SSE and OBE are equal to or more conservative than the values recommended by NRC Regulatory Guide 1.61. Damping values utilized by the NSSS are given in Subsection 3.7.3.1.2.

The damping value for the soils at the site are selected on a conservative basis from the strains induced by the earthquakes. Individual damping versus strain curves are presented in Subsection 2.5.4.

Since damping values are strain-dependent, the single values used in design were compatible with the actual strains developed during earthquakes. An equivalent linear variable-damping lumped-mass solution, similar to that developed by Idriss and Seed , was utilized. In this analysis, damping and shear moduli values were assumed and were a portion of the input to the computer. The output included a profile of calculated shear strain versus depth. On the first run, the calculated shear strain value did not correspond to the initially assumed value. The shear modulus was adjusted accordingly using Figures 2.5-77 and 2.5-78 and successive iterations made until the calculated shear strain and the assumed strain converged. The point of convergence occurred at 0.04 percent strain for the Recent alluvium and 0.08 percent strain for the upper Pleistocene sediments. Therefore, the following design values were utilized:

	DAMPING percent
Recent Alluvium (+13 to -40 ft. MSL)	8
Pleistocene Sediments (-40 to -317 ft. MSL)	7.5

3.7.1.4 Supporting Media For Seismic Category I Structures

All seismic Category I structures are founded at elevation - 47 ft. MSL on a one ft. thick compacted shell filter blanket on top of the Pleistocene clay. The Reactor Building, Reactor Auxiliary Building, Fuel Handling Building and the Component Cooling Water System structures are supported on a common foundation mat, 267 ft. wide and 380 ft. long, which is embedded 64.5 ft. below finished plant grade, in the stiff gray and tan clays.

Table 3.7-2 provides a tabulation of the foundation elevation and total structural height of the seismic Category I structures supported on common foundation mat. The plant grade elevation is +17.5 ft. MSL.

The depth of soil over the bedrock is approximately 36,000 ft. The soil layering characteristics and soil properties are discussed in Subsection 2.5.4.

3.7.2 SEISMIC SYSTEM ANALYSIS

This subsection includes discussion of seismic analysis of all seismic Category I structures. Seismic analysis of seismic Category I piping systems and components including the Reactor Coolant System is discussed in Subsection 3.7-3.

3.7.2.1 Seismic Analysis Methods

The seismic analyses of all seismic Category I structures were performed using either the normal mode time history technique or the response spectrum technique.

In the case of seismic Category I structures, the seismic response was determined by the response spectra developed for the OBE (0.05 g) and the SSE
(0.10 g), as described in Subsection 3.7.1.1.

3.7.2.1.1 Seismic Category 1 Structures

3.7.2.1.1.1 Mathematical Model

As all seismic Category I structures are founded on a common foundation mat, described in Section 3.8, the mathematical modeling involves construction of a single composite model for each directional seismic analysis.

The model comprises five individual cantilevers, representing the Reactor Building, the containment vessel, the reactor internal structure, the Reactor Auxiliary Building and the Fuel Handling Building. The Component Cooling Water System is not separately identified and is included in the Reactor Auxiliary Building and Fuel Handling Building cantilevers. The five cantilevers are founded on the same base, which is in turn supported by foundation springs. For each cantilever, the distributed masses of the structure are lumped at certain select points and connected by weightless elastic bars representing the stiffness of the structure between the lumped masses. In determining the stiffnesses, the deformation due to bending, shear and joint rotation are considered throughout.

Typical mathematical models for horizontal and vertical excitation analysis are shown on Figures 3.7-9 and 3.7-10, respectively. The input data used for these models for seismic analyses are summarized in Tables 3.7-3 and 3.7-4.

Equivalent soil springs, as described in Subsection 3.7.2.4, and damping values, as described in Subsection 3.7.1.3, are used in the analysis.

Every mass point of the two dimensional horizontal model is allowed two degrees of freedom, namely, translation and rotation. For the vertical model, only one translational degree of freedom is considered. A mathematical model for torsional effects is described in Subsection 3.7.2-11.

3.7.2.1.1.2 Equations of Motion

Once the mathematical model is established, the motion of each lumped mass under any external excitation may be written in the matrix form as follows:

$$[M] = \{\ddot{\Delta}\} + [C]\{\dot{\Delta}\} + [K]\{\Delta\} = \{F\} \qquad (1)$$

where: [M] = square mass matrix

[K] = square matrix of stiffness coefficients including the shear and bending deformations

$\{\ddot{\Delta}\}$ = column matrix of acceleration vectors

$\{\dot{\Delta}\}$ = column matrix of velocity vectors

$\{\Delta\}$ = column matrix of lateral displacement and joint rotation vectors

{F} = column matrix of external load vectors

[c] = damping matrix

The stiffness matrix [K] is formulated by computing the stiffness coefficients for each joint of the original structure and assembling them in the proper sequence to form the complete square matrix. In the computation of the stiffness matrix, it is assumed that all joints at the same level have the same displacements (i.e., translations and rotations).

→(DRN 00-1032)

The cantilever connecting two lumped masses is considered as a beam element and the effects of bending and shear deformation are included in computing the stiffness coefficients. The effects of equivalent soil springs are also included in the formation of the stiffness matrix [K]. As shown in Figure 3.7-9, there are three soil springs, two translational and one rocking being considered for horizontal excitations. The first translational spring Kx represents the shear effect between the common foundation mat and the soil and it is applied at the bottom of the mat, while the second translational spring Kxx represents the bearing effect between the mat and the soil and it is applied at the mid height of the mat side surface. The rocking spring K∅ is considered acting at the rotation center of the mat. The method used to account for torsional response is discussed in Subsection 3.7.2.11.

←(DRN 00-1032)

The effect due to relative displacement between interconnected mass points are also considered. The connecting members between mass points are modeled as beams and springs and their effects to the structural response are incorporated in the stiffness matrix. In the design of seismic Category I systems and components, the maximum relative displacement time histories of supports obtained from structural responses are utilized.

3.7.2.1.1.3 Natural Frequencies and Mode Shapes

In calculating the natural frequencies and the mode shapes, the damping term [c] $\{\dot{\Delta}\}$ is ignored and the external load vector in equation (1) is set to zero, the displacement vector $\{\Delta\}$ is assumed to take the form of simple harmonic motion:

$$\{\Delta\} = \{\phi\} \text{ Sin wt} \qquad (2)$$

where: $\{\phi\}$ = Relative amplitude of mode shape vector
ω = Natural frequency of vibration

After substituting into equation (1) and simplifying, the equations of motion are reduced to the following form:

$$[K]^{-1}[M]\{\Phi\} = \frac{1}{\omega 2}\{\Phi\} \qquad (3)$$

Solution to this eigenvalue problem exists only for particular values which correspond to the natural frequencies of vibration of the structure. Equation (3) is solved by the Jacobi method to obtain values of natural frequency of vibration (ω) and their corresponding mode shape vectors $\{\Phi\}$

3.7.2.1.1.4 Modal Analysis

After all natural frequencies and their mode shapes are determined, the method of modal analysis is employed to calculate the structural responses. This method actually simplifies the analysis of a multidegree of freedom system into an analysis of several equivalent single degree systems, one corresponding to each normal mode. The governing equation of motion is shown in the following:

→(DRN 00-1032, R11-A; 06-913, R15)

$$\ddot{A}_n + 2\beta_n A_n + \omega\frac{2}{n} A_n = \frac{-\ddot{Y}_{so} f_a(t) \sum_{x=1}^{m} M_x \alpha \phi_{xn}}{\sum_{x=1}^{N} M_x \phi 2_{xn}^{\;2}} \qquad (4)$$

←(DRN 00-1032, R11-A; 06-913, R15)

where: A_n = displacement of any one arbitrarily selected mass (usually the topmost mass) for the nth mode

β_n = damping coefficient = $\lambda_n \omega_n$

λ_n	=	Percentage of critical damping of the nth mode
ω_n	=	natural frequency of the nth mode
$\ddot{Y}_{so}$	=	maximum ground acceleration
→(DRN 00-1032, R11-A)		
$f_a(t)$	=	time function of ground motion
←(DRN 00-1032, R11-A)		
m_x	=	mass at the xth level
m	=	number of masses subjected to inertia $M_x \ddot{Y}_{so} f(t)$
ϕ_{xn}	=	normalized displacement of the maps Ms of the nth mode
N	=	total number of degrees of freedom

If the two summations on the right-hand side of the equation (4) are denoted by P_n which is defined as the modal participation factor of the nth mode, then

$$\ddot{A}_n + 2\beta_n \dot{A}_n + \omega_n A_n = -P_n \ddot{Y}_{so} f_a(t) \tag{5}$$

Since the values of B_n,ω_n and P_n are already known for each normal mode, equation (5), which is actually "n" independent equations, can be solved separately using the method developed by NC Nigen and PC Jennings.(1)

The total displacement is the summation of the displacement of each normal mode, that is:

→(DRN 00-1032, R11-A)

$$Y_x(t)_{\max} = \sum_{n=1}^{N} P_n \phi_{xn} A_n \tag{6}$$

←(DRN 00-1032, R11-A)

In spectral analysis. A_n's are spectral values from the design spectral curves. The algebraic sum of equation (6) gives the upper limit of the displacement of any mass. However, all the maximum displacements of all normal modes do not necessarily occur at the same time. For the purpose of design, the root-mean-square method is adopted from the statistical point of view:

→(DRN 06-913, R15)

$$Y_{x\max} = \left[\sum_{n=1}^{N} (P_n \phi_{xn} A_n)^2 \right]^{1/2} \tag{7}$$

←(DRN 06-913, R15)

3.7.2.2 Natural Frequencies and Response-Loads

A summary of natural frequencies for significant modes is presented in Table 3.7-5. A summary of structural responses determined by the seismic analysis for major seismic Category I structures is presented in Tables 3.7-6 through 3.7-9.

3.7.2.3 Procedure Used for Modeling

→(DRN 00-1032)

Major seismic Category I structures that are considered in conjunction with foundation media in forming a soil-structure interaction model are defined as "seismic systems." Other seismic Category I structures, systems, and components that are not designated as "seismic systems" are considered as "seismic subsystems."

←(DRN 00-1032)

The procedure used to calculate the lumped masses at designated floor levels consisted of combining the floor weights, equipment weights and one-half of the wall and column weights from the adjacent upper and lower floors. In solving the mathematical model for vertical excitation, similar lumping of masses was used.

3.7.2.4 Soil-Structure Interaction

The free-field motion of the site, during a seismic event, is locally affected by the presence of the buildings. The effects of dynamic interaction between soil and buildings can be such that the free-field response of the soil is either amplified or attenuated in some portions of the frequency range of interest. To evaluate the modifying effect of soil-structure interaction on the free-field motion (at the foundation level), a simplified lumped-mass soil spring analysis has been performed. The rationale of using lumped-mass spring method instead of finite element method for the interaction study is as follows:

a) The soil conditions, immediately underneath the plant foundations are fairly uniform and a hard rock boundary is not present in the immediate vicinity. Both these conditions dictate the use of a simplified approach for conservatism.

b) The effects of variations in soil shear modulus with strain have been considered and effective values were established from strains induced by both the static and dynamic considerations. Statistical methods of analysis were utilized to determine the participation of shear modulus throughout the time history analysis. A range of soil moduli was studied to establish the responses of soil-structure system (see Appendix 3.7-A).

c) All seismic Category I structures are located on a single common mat foundation. By virtue of this arrangement, the effects of adjacent structures on the soil-structure interaction response are automatically eliminated, leading to a simplified analysis.

The soil-structure interaction model for vertical and horizontal excitations consisted of a two dimensional lumped-mass spring system, representing the seismic Category I Nuclear Plant Island Structure and typical site geology. A three dimensional lumped-mass spring system was used for torsional response analysis. The basis for selection of a simplified soil spring approach is discussed in Appendix 3.7A. The foundation springs for horizontal excitation consisted of one rotational spring and two translational springs as shown on Figure 3.7-9. The foundation springs for vertical excitation are shown in Figure 3.7-10. The rotational and translational spring and springs were calculated using the following formulae by Whitman and Richart[2], and Barkan[3]:

Rotation (or rocking) $$K_\phi = \frac{G}{1-\mu} \beta_o \, BL^2$$

Sliding (or shear)

$$K_x = 2\ (1+\mu)\ G\ \beta_x\ \sqrt{BL}$$

→(DRN 00-1032)

Bearing (or compression)

$$K_{xx} = \frac{G\beta_z}{1-\mu}\ \sqrt{A}$$

←(DRN 00-1032)

where:

G = shear modulus of soil

μ = Poisson's ratio of soil

B = width of rectangular foundation

L = length of rectangular foundation

A = bearing area

→(DRN 00-1032)

β_o, β_x and β_z = site constants dependent on B/L ratio

←(DRN 00-1032)

The values of shear modulus and Poisson's ratio were obtained from laboratory testing and field geophysical analysis (see Subsection 2.5.4.2).

Since shear moduli are strain-dependent, the single values used in design were compatible with the actual strains developed during earthquakes. An equivalent linear variable-damping lumped-mass solution, similar to that developed by Idriss and Seed[18], was utilized. In this analysis, damping and shear moduli values were assumed and were a portion of the input to the computer. The output included a profile of calculated shear strain versus depth. On the first run, the calculated shear strain value did not correspond to the initially assumed value. The shear modulus was adjusted accordingly using Figure 2.5-77 and 2.5-78 and successive iterations made until the calculated shear strain and the assumed strain converged. The point of convergence occurred at 0.04 percent strain for the Recent alluvium and 0.08 percent strain for the upper Pleistocene sediments. Therefore the following design conservative values were utilized:

	SHEAR MODULUS psi
Recent Alluvium (+13 to -40 ft. MSL)	3400 (490 KSF)
Pleistocene Sediments (-40 to -317 ft. MSL)	5800 (830 KSF)

Refer to Appendix 3.7A for the results of a parametric study of shear modulus where it was varied from 5800 psi to 16,050 psi.

3.7.2.5 Development of Floor Response Spectra

A time history method of analysis is used to develop floor response spectra, as described in detail in Subsection 3.7.2.1.

3.7.2.6 Three Components of Earthquake Motion

→(EC-18996, R304)

The original seismic analysis of seismic Category I structures, systems or components does not consider simultaneous action of three components of design earthquake nor the calculation of responses by square root of the sum of the square of corresponding maximum values of the response as recommended in Regulatory Guide 1.92, Combination of Modes and Spatial Components in Seismic Response Analysis, December 1974. Instead the maximum value of response in each element is determined by considering each horizontal and vertical component of an earthquake separately.

→(DRN 00-1032, R11-A)

For each structural element, the two responses related to one horizontal and one vertical earthquake components are combined using the absolute sum method. The comparisons of the maximum response used in the plant structural design and that obtained using square root of the sum of the squares (SRSS) are provided in Tables 3.7-18 to 20. They are made for three randomly selected elements of the Reactor Shield Building at elevations +184.0, +61.0 and 0.0 ft. MSL, respectively. They indicate that the maximum response used is larger than the maximum response obtained using SRSS. Thus, the design approach in obtaining the maximum earthquake is equivalent to that obtained in accordance with Regulatory Guide 1.92. ASME Code N-411 is approved for application in WF3 piping analyses. The envelope response spectrum analysis method with Regulatory Guide 1.92 modal and directional combinations shall be used.

←(DRN 00-1032, R11-A; EC-18996, R304)

3.7.2.7 Combination of Modal Responses

When the spectrum method of modal analysis is used, the modes are combined by the square root of the sum of the squares (SRSS), without taking into consideration the effect of spacing of modes, as recommended by Regulatory Guide 1.92 (refer to Subsection 3.7.2.6).

3.7.2.8 Interaction of Noncategory I Structures With Seismic Category I Structures

The structural frames of nonseismic structures are designed to withstand seismic motion such that nonseismic structures will not collapse and impair the integrity of seismic Category I structures or components.

3.7.2.9 Effects of Parametric Variation on Floor Response Spectra

The following conservative assumptions are included in the calculation of the floor response spectra:

a) The expected actual earthquake time histories are enveloped by a smooth ground response spectrum for design use. This has conservative effects on modal analysis because it treats the modes in the maximum acceleration range as though they all had the same amplification factor as the most strongly amplified mode.

b) The time history used to calculate the floor response spectra produces a ground response spectrum which envelopes the design ground response spectra. In order to do this, it has spectral peaks which are substantially higher than the design spectra.

c) The building and soil damping values used in the analysis are near the lower bound of the available damping data. The actual values of damping are expected to be much higher than the values used in the analysis.

d) The yield strengths used in the analysis are based on the minimum values and are considerably lower than expected values.

e) The additional strength and damping that are available when materials are stressed beyond yield are neglected when using linear elastic analytical methods.

In order to maintain the consistent conservative design objective, parametric studies of foundation stiffness were also performed using a range of shear modulus from 5,800 psi to 16,050 psi. As a result of these studies, conservative design envelopes for all mass points and levels within the seismic Category I structures were developed for the design floor responses.

Figures 3.7-11 through 3.7-20 show the variation in floor responses (SSE with one percent damping) for shear modulus values of 5,800, 8,000 and 16,050 psi and the design envelope for related mass points and levels. Each design envelope encompasses all the spectral peaks occurring within the above range of soil shear modules and results in extremely conservative equipment and piping design at respective floor levels.

3.7.2.10 Use of Constant Vertical Load Factors

A vertical seismic system multi-mass dynamic analysis is used to account for vertical response loads (refer to Subsection 3.7.2-1.1-1).

3.7.2.11 Method Used to Account for Torsional Effects

The effects of torsional modes of vibration are analyzed by a three dimensional lumped-mass system using the MRI/Stardyne computer program (refer to Subsection 3.8.3.4). Each mass point of the system is given two orthogonal horizontal degrees of freedom and a third rotational degree of freedom in the same plane, as shown in Figure 3.7-21. The mass points are then idealized as a rigid diaphragm with three degrees of freedom, two translational and one rotational. In this analysis, torsional effect results from the translational seismic inputs because of the eccentricity between the mass center and the shear center of each floor (mass polar moment of inertia).

Soil structure interaction is considered by including translational and rotational springs at the base of the lumped-mass mathematical model as discussed in Subsection 3.7.2.4. In addition, a torsional spring is also considered.

The maximum increase in acceleration due to torsional modes of vibration is found to be less than five percent from a case without torsional mode of vibration, as shown in Table 3.7-10. The structural design takes into account the torsional effect. An additional 5 percent to or a subtraction of 5 percent from actual eccentricity has been found to have a negligible additional effect on structural acceleration responses.

3.7.2.12 Comparison of Responses

In order to provide a check on the seismic analysis of seismic Category I structures, an analysis using both the modal analysis response spectrum method and time history method has been conducted. Tables 3.7-6 through 3.7-9 give the response at selected points for major seismic Category I structures using both these methods. These responses illustrate approximate equivalency between the two methods.

3.7.2.13 Methods for Seismic Analysis of Dams

There are no seismic Category I dams associated with Waterford 3.

3.7.2.14 Methods to Determine Category I Structure Overturning Moments

The seismically induced overturning moments in the seismic Category I structures are obtained from the seismic dynamic analysis discussed in Subsection 3.7.2.1.

The bearing pressures arising from two horizontal orthogonal components of seismic motion, are combined algebraically and further combined with buoyancy and other applicable loads in accordance with the load combinations discussed in Subsection 3.8.4.3.

In calculating factors of safety against overturning, the moments due to two horizontal orthogonal components of seismic motion are combined by the SRSS method. The factor of safety against overturning for the Nuclear Plant Island Structure is 2.77 as shown in Figure 3.7-22.

3.7.2.15 Analysis Procedures for Damping

The structural and foundation material damping ratios considered in the seismic analyses are those specified in Subsection 3.7.1.3.

Composite damping in the mathematical models is determined by first evaluating the mode shapes of the system and identifying the relative participation of all portions of the system for each of these modes. Where the response participation is primarily from a single material type, the assumed damping is appropriate to that material. Where no single material can be identified as primary to the response, the damping is computed as a weighted average of the different material damping ratios based on the relative participation of each material in the mode shape. Using this procedure, modal damping ratios representing the composite damping characteristics are determined for each mode of response for use in the normal mode time history technique.

The procedure used to find the equivalent modal damping ratios for the natural modes of a structure having composite materials or substructures with different damping ratios is as follows:

→(DRN 00-1032)

$$D_n = \frac{\sum_{i=1}^{m} d_i s_{ni}}{S_n}$$

←(DRN 00-1032)

where: D^n = percentage of critical damping ratio for the n^{th} mode

d_i = percentage of material damping ratio for the i^{th} structural component

→(DRN 00-1032)

S_{ni} = strain energy of the i^{th} structural component in the n^{th} mode

$= \Sigma_i \Sigma_j \phi_{In} K_{ij} \phi_{jn}$ where i and j are limited to the component only.

S_n = total strain energy of structure in the n^{th} mode $= \Sigma_i \Sigma_j \phi_{In} K_{ij} \phi_{jn}$ where i and j are covered for the whole structure.

←(DRN 00-1032)

m = number of structural components.

3.7.3 SEISMIC SUBSYSTEM ANALYSIS

3.7.3.1 <u>Seismic Analysis Methods</u>

3.7.3.1.1 Non-NSSS Seismic Category I Piping

All seismic Category I Piping 1/2 inch or larger, other than the reactor coolant loop piping is seismically analyzed as follows:

a) All the Code Class 1 piping systems are analyzed by the method described in Subsection 3.7.3.1.1.2.

b) All the Code Class 2 and 3 piping systems except as described in (c) below using either: (b1) the method described in Subsection 3.7.3.1.1.1, or (b2) the method described in Subsection 3.7.3.1.1.2.

c) All Code Class 3 chilled water piping is analyzed by Chart Method. The lines listed in Table 3.7-21 are analyzed by Chart Method as described in Subsection 3.7.3.5.1. In all cases the design temperature is less than 275F.

The following is a list of piping systems which are analyzed by the methods as specified in item (a), (bl), (b2), and (c).

CLASS 1 PORTION OF		CLASS 2 and 3 PORTION OF
Chemical and Volume Control (a)		Blowdown (b1)
Reactor Coolant (a)		Boron Management (b1)
Safety Injection (a)		Chilled Water (c)
		Essential Cooling Water (b1)
		Chemical and Volume Control (b1)
		Condensate (b1)
		Containment Spray (b1)
	*	Demineralized Water (b1)
		Emergency Diesel (b1)
		Feedwater (b1), (b2)
		Fuel Pool Cooling (b1)
	*	Instrument Air (b1)
		Main Steam (b1), (b2)
	*	Nitrogen Gas (b1)
		Reactor Coolant (b1)
		Safety Injection (b1)
	*	Service Air (b1)
	*	Sampling System (b1)
		Waste Management (b1)

* Only the portion between the containment isolation valves were analyzed.

3.7.3.1.1.1 Equivalent Static Load Method (ESLM)

For subsystems which would normally be analyzed by the modal response spectra method (Subsection 3.7.3.1.1.2), if the first mode period of the piping is 70 percent or less of the first mode period of the structure (i.e., peak of the floor response spectra), a modal response spectra method is not performed. Equivalent static load method is used as specified in standard Review Plan Section 3.7.2.

In all cases the stiffness matrix method of natural mode analysis is employed to determine first natural period. The preset value for the maximum allowable period is 0.20 seconds which is not greater than 70 percent of the first mode period of the structure.

The equivalent static load method is made directly, using an acceleration value of 1.5 times the maximum value of the floor response spectra in the period range equal to 0.20 seconds or less.

The acceleration value that is multiplied by 1.5 is taken from Floor Response Spectra similar to Figures 3.7-11 through 3.7-20 envelopes at a period of 0.20 seconds.

3.7.3.1.1.2 Modal Response Spectra Method

The adequacy of the seismic design of the Reactor Coolant System components other than the main loop is determined by modal analysis. The mathematical models employed in the analysis is in sufficient detail to reflect the dynamic response of all significant modes. All modes with natural frequencies in the range of 33 Hz and below are considered significant.

Dynamic loads of a piping system are calculated using the acceleration values of the floor response spectra with an appropriate damping factor. These loads are then used in an elastic analysis to calculate stresses.

The method of dynamic analysis by modal response spectra method is as follows:

a) Basic Assumptions

1) The system is linearly elastic.

2) Masses are lumped at discrete intervals and are connected by weightless elastic members. The maximum spacing between mass points may not exceed one half the distance for which the frequency of a simple support beam would be 20 cps.

3) Each mass point has up to six degrees of freedom except for points indicated as restrained in a given direction.

4) The system is anchored at two or more positions and these anchor points are assumed fixed for the determination of natural frequencies and mode shapes.

→(EC-18996, R304)

5) Dynamic loadings in the three coordinate directions are determined separately and combined on the basis of excitation occurring in the vertical and one maximum horizontal direction at the same time. When using Code Case N-411, response spectrum method will use Regulatory Guide 1.92 modal and directional combinations.

←(EC-18996, R304)

6) The mass polar moment of inertia, i.e., the mass component involved in rotation, is negligible.

7) Damping is viscous and assumed constant for all modes.

8) Increased flexibility due to pipe bends is included in the analysis, pressure is included per the ASME Section III Code.

b) Method of Analysis

1) Frequency Analysis

The stiffness matrix method of natural mode analysis is employed to determine natural frequencies and associated mode shapes.

The equations of motion for the piping system may be written as:

$$[M]\{\ddot{\Delta}\}[\mathrm{K}]\{\Delta\}=\{\mathrm{F}\} \qquad (8)$$

→(DRN 00-1032, R11-A)

[M] = diagonal matrix of lumped masses, the rows and columns of which are arranged to correspond to the components of the stiffness matrix: The masses effective in the three coordinate directions are taken to be equal to the total mass assumed lumped at the point under study.

←(DRN 00-1032, R11-A)

[K] = square, symmetric matrix of stiffness coefficients including the effects of axial deformation, bending and torsional shear in the three coordinate directions.

$\{\Delta\}$ = column matrix of displacement

$\{\ddot{\Delta}\}$ = column matrix of acceleration

{F} = column matrix of external loads

The stiffness matrix [K] is assembled as follows: (each pipe section has the following properties)

E = modulus of elasticity

μ = poissons ratio

I = moment of inertia

A = cross-sectional area

L = length

From these properties the characteristics of the section are computed:

$$G = \frac{E}{2\,(1+\mu)}; \; \mathrm{GJ} = \frac{\mathrm{EI}}{1+\mu}; \; \alpha = \beta = \frac{2(1+\mu)}{\mathrm{AE}}$$

$$\varepsilon = \eta = \frac{1}{\mathrm{EI}}; \gamma = \frac{1}{\mathrm{AE}}; \lambda = \frac{1}{\mathrm{GJ}} = \frac{1+\mu}{\mathrm{EI}}$$

The end flexibility of the section is contained in the six by six matrix ϕ:

$$
\begin{matrix}
\alpha L + \dfrac{\varepsilon L^3}{3} & 0 & 0 & 0 & \dfrac{\varepsilon L^2}{2} & 0 \\
0 & \beta L + \dfrac{\eta L^3}{3} & 0 & \dfrac{-\eta L^2}{2} & 0 & 0 \\
0 & 0 & \gamma L & 0 & 0 & 0 \\
0 & \dfrac{-\eta L^2}{2} & 0 & \eta L & 0 & 0 \\
\dfrac{\varepsilon L^2}{2} & 0 & 0 & 0 & \varepsilon L & 0 \\
0 & 0 & 0 & 0 & 0 & \lambda L
\end{matrix}
$$

A rotation matrix [R] is established to bring the pipe section into the general coordinate system. This matrix is based on the orientation and location of the section in the overall system. The flexibility [ϕ_G] in the generalized coordinate system is:

→(DRN 00-1032)

$$[\phi_G] = [R]\ [\phi]\ [R]^T$$

The flexibilities are accumulated for each element and the stiffness coefficients are computed as $K_A = [\phi_G]^{-1}$ and assembled into the overall stiffness matrix. For the determination of natural frequencies and mode shapes, equation (8) is solved by first setting the external loads {F1} equal to zero and the displacement vector {Δ} equal to {δ} sin wt.

←(DRN 00-1032)

Then:

$$\{\ddot{\Delta}\} = -\{\delta\}\omega^2 \sin \omega t$$

Equation (8) becomes:

$$[K]\ \{\delta\} = \omega^2 [M]\{\delta\}$$

This generalized eigenvalue equation is solved by iterative techniques to determine the natural frequencies and mode shape vectors {δn} of the system.

This generalized procedure permits the analysis of multiple fixed branched and looped systems with multiple lumped masses as well as simple single branch systems.

2) Modal Analysis

The response of each mode of vibration considered is computed as:

→(DRN 00-1032, R11-A)

$$R_{nd} = \sum_{i=1}^{N} M_i \quad \delta_{idn}$$

←(DRN 00-1032, R11-A)

where:

N = total number of lumped masses

M_i = mass at i

d = direction X, Y, or Z

n = mode number

→(DRN 00-1032, R11-A)

δ_{idn} = component of nth mode shape in direction d at i

$$M_n = \sum_{d = d_1}^{d_3} \sum_{i=1}^{N} M_i \, (\delta_{idn})^2 \ = \text{effective mass for mode n}$$

←(DRN 00-1032, R11-A)

The disturbance factor D_n for the earthquake is defined as:

→(DRN 06-913, R15)

$$D_n = \left[\left(PF_{nd1} S_{and1}\right)^2 + \left(PF_{nd2} S_{and2}\right)^2 + \left(PF_{nds} S_{and3}\right)^2 \right]^{1/2}$$

←(DRN 06-913, R15)

where: d_1, d_2 and d_3 indication the three directions of motion

$PF_{nd} = R_{nd}/M_n$ = participation factor for nth mode in d direction

S_{and} = floor response spectral acceleration in the nth mode.

The modal inertia forces for each mode of vibration are then computed as:

$$F_{idn} = M_i \quad \delta_{idn} \ D_n$$

All significant modes (frequency less than 33 Hz) are included in the analysis.

3) Stress and Displacement Analysis

The modal inertia forces F_{idn} are utilized as response loads in a static analysis to generate modal internal forces F^*_{idn}, moments M^*_{idn} and displacement Δ_{idn}. The final stresses resulting from the earthquake disturbance are computed as the maximum resulting from combining the modal stress by the square root of the sum of the squares (SRSS) method. The final inertia of shear forces, moments and displacements to be used for design are determined by combining the results of the modes considered on the same basis, i.e.,

→(DRN 06-913, R15)

$$F^*{}_{id} = \left(\sum_{n} F^*{}_{idn}{}^{2} \right)^{1/2}$$

$$M^*{}_{id} = \left(\sum_{n} M^*{}_{idn}{}^{2} \right)^{1/2}$$

$$\delta_{id} = \left(\sum_{n} \Delta^*{}_{idn}{}^{2} \right)^{1/2}$$

$$\sigma_i = \frac{\left(\sum_{d=d_1}^{d3} M^*{}_{id}{}^{2} \right)^{1/2}}{Z}$$

←(DRN 06-913, R15)

where:

z = section modulus of pipe cross section

The computer program (PIPESTRESS 2010) used for the simplified dynamic analysis utilizes the same stiffness matrix method as that described for the modal response spectra method. The program automatically determines forces, moments and deflections in the three coordinate directions and the stresses at selected points in the piping system. The intensification factor is applied to both bending moment and torsional moment. The computer program PIPESTRESS 2010 is discussed in Subsection 3.9.1.2.1.1.

3.7.3.1.2 Reactor Coolant System

3.7.3.1.2.1 Seismic Analysis Methods

The adequacy of seismic loadings used for the design of the major components of the Reactor Coolant System were confirmed by the methods of dynamic analysis employing time-history and response spectrum techniques. The major components are the reactor vessel, the steam generators, the reactor coolant pumps, the reactor coolant piping and the pressurizer.

→(EC-8458, R307)

The original seismic loads prior to RSG were calculated by performing a Nuclear Island building seismic time history analysis with a simplified representation of the RCS (Reactor Coolant System) consisting of the mass of the RCS. The earthquake excitation consisted of ground motion acceleration time histories for OBE and SSE applied to the base of the Nuclear Island buildings. The seismic response of the coupled building/RCS model was computed. The time histories at the RCS major component supports to the building produced by these analyses were then applied to a separate and more detailed RCS model to obtain seismic loads for the RCS major components and supports.

The original analysis was performed using the STRUDL computer code. The RSG coupled Nuclear Island/RCS seismic analysis used the same earthquake excitation and the same Nuclear Island model as the original analysis, with the model converted to the ANSYS computer code and enhanced as follows. The original coupled analysis used a simple RCS model consisting of mass only. The RCS portion of the RSG coupled analysis was represented as a detailed model that accounted for both mass and stiffness. By using this approach, the intermediate step in the original analysis of saving the support time histories

←(EC-8458, R307)

→(EC-8458, R307)
and applying them as input to the more detailed RCS model was eliminated. Consequently, for the RSG seismic analysis, the ground motion acceleration time histories were applied at the base of the coupled building/RCS model, and the reactor coolant system loads were determined directly from this analysis.

The seismic analysis methodology in the RSG analysis is equivalent to the original analysis. As discussed above, the only difference is the inclusion of the detailed RCS model directly into the RSG coupled building/RCS model, which eliminated an intermediate step in the analysis process. The RSG analyses were performed for SSE and OBE using the ANSYS computer code.
←(EC-8458, R307)
The analysis of the pressurizer and surge line employed separate mathematical models and utilized response spectrum techniques. A representation of the coupled components, of sufficient detail to account for possible dynamic interaction effects from the containment internal support structure to the Reactor Coolant System components, was supplied for consideration in performing the analysis of the containment internal support structure. The results of the analysis of the containment internal support structure included the time-history forcing functions for use in the separate analysis of the detailed model of the coupled components of the Reactor Coolant system. Response spectra were developed from the containment internal support structure analysis for use in the pressurizer dynamic analysis.

For time history analyses, dynamic responses to vertical seismic excitation were found for both the case of initial support displacement upward and the case of initial support displacement downward. The responses were combined to determine the most severe combinations produced by the effects of seismic excitations in each of the horizontal directions applied simultaneously with seismic excitation in either vertical direction.

→(EC-8458, R307)
←(EC-8458, R307)

The sum of the absolute values method was used to combine the modal responses for the response spectrum modal analysis of the pressurizer.

In general, the damping factors used in the seismic analyses of the major components of the Reactor Coolant System were selected from Table 3.7-11. Modal damping factors of two percent of critical and one percent of critical, for the SSE and OBE respectively, were used in the seismic analyses of the major components of the Reactor Coolant System to avoid under estimating the amplitude of vibrations or dynamic stresses (Regulatory Guide 1.61 October 1973, paragraph c.3).

3.7.3.1.2.2 Mathematical Models

In the description of the mathematical models which follow, the spatial orientations are defined by the set of orthogonal axes where Y is in the vertical direction, and X and Z are in the horizontal plane, in the directions indicated on the appropriate figure. The mathematical representation of the section properties of the structural elements employs a 12 by 12 stiffness matrix for the three-dimensional space frame models, and employs a six by six stiffness matrix for the two-dimensional plane frame model. Elbows in piping runs include the in-plane/out-of-plane bending flexibility factors as specified in the ASME B&PV Code, Section III.
→(EC-8458, R307)
a) Reactor Coolant System - Coupled Components

The original Reactor Coolant System (RCS) seismic model with the original steam generators (OSG) was developed with the ICES STRUDL-II computer Code. This model was converted to the ANSYS computer code for the analysis of the replacement steam generators (RSG). A comparison of the natural frequencies of the STRUDL RCS model with OSG to the ANSYS RCS model with OSG is shown in Table 3.7-12. The changes in frequency are negligible, confirming that the ANSYS model is benchmarked to the original analysis.
←(EC-8458, R307)

➔(EC-8458, R307)

The Coupled Building/RCS seismic model with RSG was developed in ANSYS. The RCS portion of the model for RSG is basically the same as the pipe break RCS model developed for EPU, with the OSG replaced by the RSG, plus some additional modifications. These include removal of nonlinearities to make it consistent with the linear seismic model in the original analysis. The other changes include the replacement RV closure head, a separate representation of the RSG sliding base, separate snubber/lever systems for RSG#1 and RSG#2, and the RSG shear key representation. The analysis included NGF fuel, but since the weight change for NGF fuel is negligible, no changes in the model were required. The representation of the reactor internals is formulated in conjunction with the analysis of the reactor internals discussed in Subsection 3.7.3.14 and is designed to simulate the dynamic characteristics of the models used in that anlaysis.

🡐(EC-8458, R307)

The mathematical model provides a three-dimensional representation of the dynamic response of the coupled components to seismic excitations in both the horizontal and vertical in both the horizontal and vertical directions. The mass is distributed at the selected mass points and corresponding translational degrees of freedom are retained to include rotary inertial effects of the components. The total mass of the entire coupled system is dynamically active in each of the three coordinate directions.

b) Pressurizer

The mathematical model employed in the analysis of the pressurizer is shown schematically in Figure 3.7-24. This lumped parameter, planar model provides a multi-mass representation of the axially symmetric pressurizer and includes two mass points with a total of five dynamic degrees of freedom.

➔(EC-8458, R307)

c) Surge Line Analysis

A lumped mass mathematical model is employed in the analysis of the surge line for Replacement Steam Generators. The surge line is modeled as a three-dimensional piping run with end points anchored at the attachments of the pressurizer and the reactor vessel outlet piping. In the definition of the mathematical model, 36 mass points with a total of 108 dynamic degrees of freedom were selected to provide a complete three-dimensional representation of the dynamic response of the surge line. All supports and restraints defined for the surge line assembly are included in the mathematical model. The total mass of the surge line is dynamically active in each of the three coordinate directions.

🡐(EC-8458, R307)

➔(EC-8458, R307)

🡐(EC-8458, R307)

3.7.3.1.3 Results

➔(EC-8458, R307)

The reactions (forces and moments) at all design points in the system, obtained from the dynamic seismic analysis with the replacement steam generator are compared with seismic loads in each component design specification.

🡐(EC-8458, R307)

The maximum seismic loads calculated by the time history techniques are the result of a search and comparison over the entire time domain of each individual component of load due to the simultaneous application of the vertical and either horizontal excitation. The maximum calculated components of load for each design location do not in general occur at the same time, nor for the same combination of horizontal and vertical excitation, and therefore results in a conservative worst case.

The maximum seismic loads calculated by the response spectrum techniques are the result of combining the modal reactions due to the horizontal and the vertical excitation.

3.7.3.1.4 Conclusion

All seismic loads calculated by the dynamic seismic analyses are less than the corresponding loads in the component design specifications. These analyses were performed for the OBE excitation and the results are compared with the OBE design specification loads. The design specifications conservatively use twice the OBE loads for the SSE, therefore the conclusion for the analysis and comparison for OBE loads is the same for the SSE.

→(EC-8458, R307)

All seismic loads calculated for the reactor coolant system with replacement steam generator are acceptable and all components and supports are structurally adequate.

←(EC-8458, R307)

3.7.3.2 Determination of Number of Earthquake Cycles

The procedure used to account for the fatigue effect of cyclic motion associated with the OBE recognizes that the actual motion experienced during a seismic event consists of a single maximum or peak motion, and some number of cycles of lesser magnitude. The total or cumulative fatigue effect of all cycles of different magnitude will result in an equivalent cumulative usage factor. The equivalent cumulative usage factor can also be specified in terms of a finite number of cycles of the maximum or peak motion. Based on this consideration, seismic Category I subsystems, components, and equipment are designed for a total of 200 full-load cycles about a mean value of zero and with an amplitude equal to the maximum response produced during the entire OBE event.

3.7.3.3 Procedure Used For Modeling

The mathematical model used in all seismic Category I piping subsystems not analyzed by the chart method includes sufficient mass points and corresponding degrees of freedom to provide a three dimensional representation of the dynamic characteristics of the subsystem. The distribution of mass and the selected location of mass points account for torsional effects of valves and other eccentric masses.

Rigid valves are modeled as outlined in Subsection 3.7.3.11.

Non-rigid valves (less than 33 Hz) are modeled with the piping. The model of the valve is supplied by the valve manufacturer.

All non-rigid equipment is modeled with the piping subsystem with sufficient detail to reflect the dynamic response of all significant modes. All modes with natural frequencies in the range of 33 Hz and below are considered significant.

Modeling of reactor internals, fuel assemblies and control element drive mechanisms is described in Subsection 3.7.3.14.

3.7.3.4 Basis for Selection of Forcing Frequencies

Where feasible, systems and components are arranged to avoid the resonant frequency regions. The shifting of the resonant region of subsystems are accomplished by modifying their mass-stiffness characteristics.

If the first mode period of the piping is over 70 percent of the first mode period of the structures, a multi-mode response analysis is performed.

For ASME Section III Code Class I piping systems where the first mode period of the piping is 70 percent or less of the first mode period of the structures, procedures as outlined in Subsection 3.7.3.1 are followed.

The basis for acceptability of the seismic design of equipment and subsystems for the Reactor Coolant System is that the stresses and deformations produced by vibratory motion of the postulated seismic events, in combination with other coincident loadings, be within the established limits.

Within practical limitations, the seismic design of the Reactor Coolant System is accomplished in a manner to maintain the resonant frequencies well above the range which is significantly excited by the forcing frequencies. Based upon the results of analysis of the preliminary design, the stiffness of the restraint and support system is modified as required to maintain the fundamental frequencies of equipment and subsystems sufficiently removed from the resonant range and, thereby, maintain the seismic response within the established limits.

Structural fundamental frequencies ("forcing frequencies") for reactor internals are not selected but calculated as described with Subsection 3.7.3.14.

3.7.3.5 Use of the Chart Method and Equivalent Static Load Method

3.7.3.5.1 Use of the Chart Method

The chart method of analysis consists of locating restraints such that the period of the first mode of vibration will not exceed the preset value of 70 percent of the first mode period of the supporting structure. This method involves the use of appropriate and comprehensive charts and tabulations that include correction factors for the effects of concentrated loads, branch connections and other effects. The piping system is studied for loading effects in each of the three coordinate directions to assure that it is adequately restrained in all directions. An additional analysis is performed to evaluate the thermal effects of the restraints of the system. This is done by means of charts that define the minimum distance required for placing restraints adjacent to any expanding leg in order to stay within allowable stress limits. The lines which are analyzed by this method are listed in Table 3.7-21.

3.7.3.5.2 Use of Equivalent Static Load Method (ESLM)

To justify the ESLM analysis (Subsection 3.7.3.1.1.1) procedure for piping, three sample problems (see Figures 3.7-26 through 3.7-29 are presented using both ESLM analysis and modal response spectra methods (Subsection 3.7.3.1.1.2). The static analyses used 1.0 g horizontal and 0.666 g vertical accelerations. The dynamic analysis utilized seven modes for sample one and five modes for samples two and three. For all modes the horizontal acceleration was taken as 1.0 g and vertical acceleration 0.666 g. The periods for the analyzed modes of all systems were between 0.20 seconds and 0.08 seconds.

In all cases the maximum computed stress was higher for the ESLM analysis than for the dynamic analysis.

Table 3.7-14 shows the computed stress values at the point of maximum stress for each problem and method. The maximum stress is at the same point for both methods in all three problems.

➔(DRN 00-1032, R11-A)
Since the ESLM yields higher stresses than the dynamic analysis when using the same acceleration, the use of 1.5 times the peak of the floor response spectra for piping periods less than 70 percent of the first period of the structure is conservative by at least a 1.5 factor for the three systems analyzed by both methods.
🡐(DRN 00-1032, R11-A)

3.7.3.6 Three Components of Earthquake Motion

Subsection 3.7.3.14 discusses the components of earthquake motion used in the analysis of reactor internals, fuel assemblies and control element drive mechanisms. For all other seismic category I subsystems, refer to Subsection 3.7.2.6.

3.7.3.7 Combination of Modal Responses

3 7.3.7.1 Subsystems Other Than The NSSS

The procedure for combining modal responses for seismic Category I subsystems other than NSSS supplied components is discussed in Subsection 3.7.3.1.1 with the following modification. In the analysis of complex systems where closely spaced modal frequencies are encountered, the responses of the closely spaced modes are combined by the summation of the absolute values method and, in turn, combined with the responses of the remaining significant modes by the square root of the sum of the squares method. Modal frequencies are considered closely spaced whithin their difference is less than $\pm$ 10 percent of the lower frequency.

→(DRN 00-1121, R11-A)

3.7.3.7.2 NSSS

←(DRN 00-1121, R11-A)

The SRSS method is the procedure normally used for the NSSS to combine the modal responses, when the modal analysis response spectrum method of analysis is employed. The procedure is modified only in two cases:

a) In the analysis of simple systems, where three or less dynamic degrees-of-freedom are involved, the modal responses are combined by the summation of the absolute values method.

b) In the analysis of complex systems, where closely spaced modal frequencies are encountered, the responses of the closely spaced modes are combined by the summation of the absolute values method and, in turn, combined with the responses of the remaining significant modes by the SRSS method. Modal frequencies are considered closely spaced when their difference is less than 10 percent of the lower frequency.

3.7.3.8 Analytical Procedures for Piping

For all ASME Code Class I piping, the loadings and, in turn, the primary stresses produced by inertial effects are determined by applying the modal responses, mode-by-mode, to the piping system with the supports/restraints maintained "fixed". The modal results are then combined as discussed in Subsection 3.7.3.7. The loadings and, in turn, the secondary stresses produced by the relative displacements of the piping supports/restraints are determined by imposing the relative displacements on the piping system. The displacements are imposed in a manner to produce maximum primary plus secondary stresses in the piping when the total inertial effects are added to the effects resulting from the imposed relative displacements. There is no Reactor Coolant System piping that is routed between buildings.

For all ASME Code Class 2 and 3 piping differential displacement between buildings is taken into account in the seismic analysis but that displacements at different support points within a structure are not considered, because they are negligible. This is based on a review of all ASME Code Class 1 calculations which indicated that the maximum relative displacement between the two extremes of any calculation.

3.7.3.9 Multiple Supported Equipment Components With Distinct Inputs

When the response spectrum method is used, the effect of relative displacements between support points are considered in the following situations:

a) piping passing between structures and anchored at such structural penetrations or by equipment mounted on such structures, which have a common foundation mat where significantly different structural deflection modes can be expected under seismic disturbances, and

b) piping or portions of piping wholly contained within a structure and anchored at equipments or intermediate walls supported on a common structural foundation, when such equipments or intermediate walls exhibit deflection modes markedly different from the enveloping structure under seismic excitations. As an example, the absolute sum of displacements between the top of the pressurizer and the containment penetration anchor is considered.

The implementation of relative displacement considerations* are as follows:

a) First a static analysis is performed by considering the maximum relative displacement between the end anchor points. Such design displacement is obtained by adding absolutely the maximum anchor movements.

b) Second, a dynamic analysis is performed assuming no relative displacement between anchor points, but using the worst floor response spectrum when the anchor points are in the same structure or the enveloped floor response spectrum if necessary when the support points are in separate structures.

Results from steps a and b are combined by SRSS method with other stresses to satisfy ASME Section III Code equations.

3.7.3.10 Use of Constant Vertical Static Load Factors

A single constant seismic vertical load factor is not used for the seismic design of seismic Category I structures, components, and equipment (refer to Subsection 3.7.2.1.1.1).

3.7.3.11 Torsional Effects of Eccentric Masses

Safety Class 1

Torsional effects of motor and air operated valves and other eccentric masses are included in the analysis of safety class 1 systems by taking into account the mass and eccentricity in the mathematical model (refer to Subsection 3.7.3.3).

The mathematical models used in seismic analysis of Reactor Coolant System other than the main loop include sufficient mass points and corresponding dynamic degrees-of-freedom to provide a three dimensional representation of the dynamic characteristics of the system. The distribution of mass and the selected location of mass points account for torsional effects of valves and other eccentric masses.

Safety Class 2 and 3

For valves that are rigid, a study performed for the H B Robinson Nuclear Unit No. 2 (Docket No. 50261) demonstrated that the additional pipe stresses due to the offset weights of valve operators were insignificant except for the case where large operators were installed on two in. and smaller pipes. The weight of valves and operators are lumped at the center line of

*Where the relative displacement considerations apply, piping stress analysis is performed under OBE conditions.

the pipe in the mathematical model except for lines two in. or less in diameter. When analysis using this procedure indicates stresses close to allowable code values in the vicinity of the valve, the stress analysis is performed considering the eccentricity of the masses.

Non-rigid valves (< 33 Hz) are either modeled with the piping using the model supplied by the manufacturer (see Subsection 3.7.3.3) or the valves are made rigid by the provision of additional supports.

3.7.3.12 Buried Seismic Category I Piping Systems and Tunnels

There is no buried seismic Category I piping at Waterford 3.

3.7.3-13 Interaction of Other Piping with Seismic Category I Piping

Where seismic Category I piping systems are in close proximity to nonseismic systems, the excessive movement of the nonseismic system due to seismic induced effects is restrained so that no failure of the seismic Category I system occurs.

Where seismic Category I piping is directly connected to nonseismic piping, the seismic effects of the nonseismic piping is prevented from being transferred to the seismic Category I piping by use of anchors or combinations of restraints. If this is not practical, the interactive effects of the unrestrained portion of the nonseismic piping is included in the analysis of the seismic Category I piping.

Dynamic coupling, or interaction effects, between seismic Category I RCS piping other than the main loop and piping systems which are nonseismic is accounted for in the seismic design of seismic Category I piping. Where practical, this is accomplished by providing a system of supports and restraints at the interfaces between the seismic Category I and the nonseismic piping system to dynamically decouple the two systems. Otherwise, the mathematical model used for seismic analysis of the seismic Category I piping is extended to incorporate the pertinent features of the nonseismic piping system.

3.7.3.14 Seismic Analysis of Reactor Internals, Core, and Control Element Drive Mechanisms (CEDM)

Dynamic analyses of the reactor internals, core and CEDM's are conducted to determine their response to horizontal and vertical seismic excitation, and to verify the adequacy of their seismic design. The seismic analyses of the internals and core include the use of model analysis techniques for linear conditions, and step-by-step integration of the equations of motion for nonlinear impact conditions, such as exist when the gaps between components close. These analyses are conducted in conjunction with the analyses of the Reactor Coolant System. The applicable seismic load, stress and deformation criteria are presented in Subsections 3.9.4, 3.9.5, and Section 4.2.

3.7.3.14.1 Reactor Internals and Core

The seismic analyses of the reactor internals and core consist of two phases. In the first phase, linear lumped-parameter models are formulated, natural frequencies and mode shapes for the models are determined, and the response is obtained utilizing the modal analyses response spectrum method. The response spectra used are based upon the acceleration of the reactor-vessel supports. The response spectra are modified by a conservative factor to account for possible response amplification between the vessel supports and the reactor internals support. The response spectrum analysis is used to obtain preliminary design seismic loads and displacements.

In the second phase, because the linear horizontal analysis showed that the relative displacements between the core and core shroud and between the core-support barrel and pressure-vessel snubbers are sufficiently large to close the gaps that exist between these components, a nonlinear horizontal analysis is required. The horizontal nonlinear analysis is divided into two parts. In the first part, the internals and core are analyzed to obtain the internals response and the proper dynamic input for the reactor core model. In the second part, the core plate motion from the first part is applied to a more detailed nonlinear model of the reactor core. Because the linear vertical analysis indicates that the response of the core is not sufficiently large to cause it to lift off the core support plate, a vertical nonlinear analysis of the internals is unnecessary.

In these analyses, the horizontal and vertical components of the seismic excitation are considered separately and the maximum absolute responses for either horizontal direction are added to the maximum absolute vertical responses.

3.7.3.14.1.1 Mathematical Models

Equivalent multi-mass mathematical models are developed to represent the reactor internals and core. The linear mathematical models of the internals are constructed in terms of lumped masses and elastic-beam elements. At appropriate locations within the internals and core, points (nodes) are chosen to lump the weights of the structure. A sketch of the internals and core showing the relative node locations for the horizontal model is presented on Figure 3.7-30.

Figures 3.7-31 and 3.7-32 show the idealized linear horizontal and vertical models. The criterion for choosing the number and location of mass concentration is to provide for accurate representation of the dynamically significant modes of vibration of each of the internals components. Between the nodes, properties are calculated for moments of inertia, cross-sectional areas, effective shear areas, and lengths. Since the seismic excitation of the internals is input at the vessel/internals interface, only the internals and core are included in the models. Separate horizontal and vertical models of the internals and core are formulated to more efficiently account for structural differences in these directions. Consequently, the horizontal and vertical response are treated as uncoupled.

The horizontal nonlinear lumped mass representation of the internals, shown on Figure 3.7-33 is essentially identical to the linear model except for the gap and springs between the core support barrel and reactor vessel, and between the core shroud guide lugs and the fuel alignment plate. The internals model is excited by applying the earthquake time-history obtained from the reactor coolant system analysis, to the reactor vessel, which is assumed to be a rigid body. The input excitation consists of the horizontal and rotational (rocking) motion of the reactor vessel.

The horizontal nonlinear reactor core model consisting of one row of 17 individual fuel assemblies is depicted on Figure 3.7-34. The distribution of mass and stiffness values is based upon experimentally determined fuel-assembly vibration characteristics. To simulate the gaps in the core nonlinear spring couplings are used to connect corresponding nodes on adjacent fuel assemblies and the core shroud. The impact stiffness and impact damping (coefficient of restitution) parameters for the gap elements are derived from the impact tests described in Reference 19. The spacer grid impact representation used for the analysis is capable of representing two types of fuel assembly impact situations. In the first type, only one side of the spacer grid is loaded. This type of impact occurs when the peripheral fuel assembly hits the core shroud, or when two fuel assemblies strike one another. The second type of impact loading occurs typically when the fuel assemblies pile up on one side of the core. In this case, the spacer grids are subjected to a through-grid compressive loading.

The damping factors used in the seismic analysis of the reactor internals are in accordance with the values in Table 3.7-11. Damping values used for fuel assemblies are based on the results of the full-scale structural tests.

Additional salient details of the internals and core models are discussed in the following paragraphs.

a) The dynamic analysis of reactor internals presents some special problems due to their immersion in a confined fluid. It has been shown both analytically and experimentally[8] that immersion of a body in a dense fluid medium lowers its natural frequency and significantly alters its vibratory response as compared to that in air. The effect is more pronounced where the confining boundaries of the fluid are in close proximity to the vibrating body as is the case for the reactor internals.

   For the linear analysis, the method of accounting for the effects of a surrounding fluid on a vibrating system is to ascribe to the system additional or "hydrodynamic mass". This hydrodynamic mass decreases the frequencies of the system, but is not directly included in the inertia force effects. Hydrodynamic mass effects for moving cylinders in a water annulus are discussed in References 8 and 9. The results of these references are applied to the internals structures to obtain the total (structural plus hydrodynamic) mass matrix that is used in the evaluation of the natural frequencies and mode shapes for the model.

   For the nonlinear analysis, hydrodynamic coupling between the reactor vessel and core support barrel and between the core support barrel and core shroud is also included. This is done following the method of Reference 10, by including off-diagonal terms in the mass matrix.

b) Core Support Barrel

   The core support barrel is modeled as a beam with shear deformation. It has been shown that the use of beam theory for cylindrical shells gives sufficiently accurate results when shear deformation is included[11][12].

c) Fuel Assemblies

   The fuel assemblies are modeled as uniform beams with rotational springs at each end to represent the proper end conditions. The member properties for the beam elements representing the fuel assemblies are derived from the fuel-assembly load deflection characteristics and fundamental natural frequency.

d) Support-Barrel Flanges

To obtain accurate lateral and vertical stiffnesses of the upper and lower core-support-barrel flanges and the upper guide structure support-barrel upper flange, finite-element analyses of these regions are performed. As shown in Figure 3.7-35, these areas are modeled with quadrilateral and triangular ring elements. Asymmetric loads, equivalent to lateral shear loads and bending moments, and symmetric axial loads are applied and the resulting displacements calculated. These results are then used to derive the equivalent member properties for the flanges.

e) Control Element Assembly (CEA) Shrouds

For the horizontal model, the CEA shrouds are treated as vibrating in unison and are modeled as guided cantilever beams in parallel. Since the stiffness of the upper guide structure support plate is large compared to that of the shrouds, the CEA shrouds are assumed to be rigidly connected to the upper guide structure support plate.

f) Upper Guide Structure Support Plate and Lower Support Structure Grid Beams

These grid beam structures are modeled as plane grids. Displacements due to vertical (out-of-plane) loads applied at the beam junctions are calculated through the use of the STRUDL computer program.[5] Average stiffness values based on these results yield an equivalent member cross-section area for the vertical model.

3.7.3.14.1.2 Analytical Techniques

a) Natural Frequencies and Mode Shapes

The mass- and beam-element properties of the models are utilized in the STAR program computer from the MRI/STARDYNE Analysis System programs [13][14] to obtain the natural frequencies and mode shapes. The system utilizes the stiffness-matrix method of structural analysis. The natural frequencies and mode shapes are extracted from the system of equations:

→(DRN 00-1032, R11-A)

$$(\underline{K} - W\frac{2}{n}\underline{M})\,\phi_n = 0 \qquad (10)$$

←(DRN 00-1032, R11-A)

where:

$\underline{K}$ = model stiffness matrix

M = model mass matrix

W_n = natural circular frequency for the n^{th} mode

Φ_n = normal mode shape matrix for n^{th} mode

The mass matrix, $\underline{M}$, includes the hydrodynamic and structural masses.

b) Response Calculations Methods

1) Response Spectra Method

The response spectrum analysis is performed using the modal extraction data and the following relationships for each mode:

(a) Nodal Accelerations

→(DRN 00-1032, R11-A)

$$\ddot{X}_{in} = \gamma_n A_n \phi_{in} \quad (11)$$

←(DRN 00-1032, R11-A)

where:

$\ddot{X}_{in}$ = absolute acceleration at node "i" for mode "n"

γ_n = modal participation factor

A_n = modal acceleration from response spectrum

ϕ_{in} = mode shape factor at node "i" for mode "n"

(b) Nodal Displacement

→(DRN 00-1032, R11-A)

$$Y_{in} = \frac{\ddot{X}_{in}}{W_n^2} \quad (12)$$

←(DRN 00-1032, R11-A)

where:

Y_{in} = displacement at node "i" for mode "n" relative to base

w_n = natural circular frequency for n^{th} mode

(c) Member Forces and Moments

→(DRN 06-913, R15)

$$F_n = \frac{(\gamma_n A_n)}{W_n^2} \overline{F}_n \quad (13)$$

←(DRN 06-913, R15)

where:

→(DRN 00-1032, R11-A)

F_n = actual member force for mode "n"

←(DRN 00-1032, R11-A)

$\overline{F}_n$ = modal member force for mode "n"

The effect of the fluid environment is accounted for by defining the modal participation as follows:

←(DRN 00-1032, R11-A)

→(DRN 06-913, R15)

$$\gamma_n = \frac{\sum_{i=1}^{M} W_{si}\,\phi_{in} 2}{\sum^{M} W_{Ti}\,\phi_{in} 2} \qquad (14)$$

←(DRN 00-1032, R11-A; 06-913, R15)

where:

W_{si} = structural weight of node "i"

W_{Ti} = structural plus hydrodynamic weight of node "i"

M = number of masses.

The SRSS method is normally used to combine the modal responses. Where modal frequencies are closely spaced, the responses of these modes are combined by the sum of their absolute values. The modal damping factors are obtained by the method of "mass mode weighting," which gives:

$$\beta_n = \frac{\sum \mathrm{M}_{\mathrm{i}} \left|\phi_{in}\right| \beta_i}{\sum \mathrm{M}_{\mathrm{i}} \left|\phi_{in}\right|} \qquad (15)$$

where:.

β_n = modal damping factor

M_i = structural mass of mass node i"

$|\phi_{in}|$ = absolute value of the mode shape at mass mode "i"

β_i = damping associated with mass point "i".

2) Nonlinear Analysis

The nonlinear seismic response and impact forces for the internals are determined using the CESHOCK computer program(15). This computer program provides the numerical solution to transient dynamic problems by step-by-step integration of the differential equations of motion. The input excitation for the model is the time-history accelogram of the reactor vessel.

Input to the CESHOCK computer program consists of initial conditions, nodal lumped masses, linear-spring coefficients, mass moments of inertia, nonlinear spring curves, and the time-history. The output from the CESHOCK computer program consists of displacements, translational and angular accelerations, impact forces, shears, and moments.

A brief description of the general methods used in the CESHOCK computer program to solve transient dynamic problems can be found in Subsection 3.9.1.2.2.2.5.

3.7.3.14.1.3 Results

Reactor internals and fuel response due to vertical and horizontal OBE and SSE excitations are calculated using the models and techniques described above. The resulting seismic loads for the reactor internals are combined with other loads as described in Subsection 3.9.5 and the resulting stresses and deformations are compared to criteria presented in Subsection 3.9.5. The resulting loads, stresses and deformations of the fuel are compared to the criteria presented in Section 4.2.

3.7.3.14.2 Control Element Drive Mechanisms

The pressure retaining components of the CEDM are designed to the appropriate stress criteria of ASME Code Section III for all loadings specified in the mechanisms design specification. The structural integrity of the CEDM when subjected to seismic loadings is verified by a combination of test and analysis. Methods of modal dynamic analysis employing response spectrum techniques are supported with experimentally obtained information.

Dynamic responses to horizontal and vertical seismic excitation in the tests and analyses are considered separately in obtaining the final results.

The reed switch position transmitter (RSPT), as required in the design specification, is a Class 1E electrical component. As such, the capability of the RSPT to perform adequately during and following seismic events of the specified intensities is demonstrated.

→ (DRN 03-2056, R14)

3.7.3.14.2.1 Input Excitation Data

←(DRN 03-2056, R14)

For the dynamic analyses, a response spectra definition of the excitation at the base of the CEDM Nozzle is obtained from the seismic analysis of the RCS.

→ (DRN 00-1032, R11-A)

For the verification tests of the mathematical model, a continuous sine wave input motion of sufficient amplitude at the base of the nozzle is used.

←(DRN 00-1032, R11-A)

→ (DRN 03-2056, R14)

Response spectra output from RCS seismic and BLPB analyses were applied to the base of the CEDMs and the base of the closure head lift rig (CHLR) for the power uprate evaluations.

←(DRN 03-2056, R14)

For the seismic qualification testing of the RSPT, random, biaxial multifrequency input motion of intensities greater than the required response spectra is used.

3.7.3.14.2.2 Model Description

→(DRN 03-2056, R14)

For the original analysis a mathematical model consisting of lumped masses and weightless structural members was used in the dynamic analysis of the CEDM design. The lumped mass nodal points and member stiffness properties were defined to provide an accurate representation of the dynamically significant modes of vibration within the seismic frequency range. A schematic representation of the model is shown in Figure 3.7-36. Due to symmetry of the CEDM structure about its longitudinal axis, a two-dimensional representation of the CEDM dynamic response was sufficient. The effect of different CEDM nozzle lengths on the dynamic response characteristics of the CEDM was accounted for in the analysis.

→(EC-2800, R307; EC-8458, R307)

For the evaluation of the CEDMs for power uprate to 3716 MWt and the Replacement Steam Generators, the CHLR, which under the current configuration provides an additional support near the top of each CEDM, was added to the CEDM model. The coupled CEDM/CHLR model is defined using the ANSYS computer code. The same analytical model was used for the replacement CEDMs.

←(EC-2800, R307; EC-8458, R307)

3.7.3.14.2.3 Analysis

For the original evaluation, the dynamic analysis of the mathematical structural model was performed using the ICES STRUDL II computer program. Natural frequencies and mode shapes of the composite mathematical model were calculated first. These values were verified through test. Where necessary, the model was modified to reflect the test results. Using the mode shapes and natural frequencies, a modal response analysis using a response spectrum technique was performed. The response at each node was calculated and compared with design criteria. The analysis was based on a critical damping ratio of one percent. Tests have shown damping to be at least twice as high. The CEDM nozzle loads were also verified against design allowables.

→(EC-2800, R307)

For the evaluation of the CEDMs for power uprate 3716 MWt, and for the replacement CEDMs, a dynamic analysis of the coupled CEDM/CHLR structural model is performed using the ANSYS computer code. Natural frequencies and mode shapes for the CHLR and the CEDMs are first calculated separately and compared to verified values.

←(EC-2800, R307)

Using the mode shapes and natural frequencies, a modal response analysis using a response spectrum technique is performed, and the response at each node is calculated and compared to acceptance criteria given in Section 3.9.4.

Damping values used in the analysis are provided below:

	Dynamic Event	
Location	OBE	SSE & BLPB
CHLR	4%	7%
CEDM	2%	4%
Boot	10%	10%

←(DRN 03-2056, R14)

3.7.3.14.2.4 Tests

A prototype mechanism and nozzle is tested to verify the adequacy of the calculation. The testing consists essentially of experimentally determining the mechanism natural frequency and damping factor at both room and operating temperatures. In addition to these pluck tests, forced vibration tests are conducted in order to verify the mathematical model for higher order resonance modes and to demonstrate applicability of the selected damping parameters for the entire excitation range. A functional test of the mechanism is also performed to demonstrate that the mechanism can operate and allows scramming of the control element assembly when subjected to the deflections that occur during a seismic event.

A prototype RSPT is installed inside the CEDM shroud assembly of a full CEDM which is assembled in the seismic shaker fixture. The mounting configuration simulates in-service conditions. The complete test assembly is excited with a sufficient number of random, biaxial multi-frequency input motions of intensities greater than the required response spectra. The electrical performance of the RSPT is verified during and following the test cycles.

3.7.3.14.2.5 Results

The responses (forces and moments) at all design points in the system, obtained from tests and/or analysis are compared with the seismic load and deflection restrictions in the design specification.

→(DRN 03-2056, R14)
The following is a summary of significant results for the original seismic analysis.
←(DRN 03-2056, R14)

a) Damping was verified to be at least twice as high as the value used for the analysis.

b) Adequacy of the mathematical model was verified by test.

c) The CEDM was analyzed with 17 different nozzle lengths and all results were enveloped. The maximum computed nozzle bending moment (in-lbs) compares with the allowables as follows:

Type of Loading	Analysis	Allowable
Operational & OBE	87,303	108,600
Operational & SSE	126,269	163,800

→(EC-2800, R307)
d) A stress analysis of the CEDM pressure containing components demonstrated the stresses to be well below the material allowables. The analysis was in compliance with the guidelines of the ASME code Section III.
←(EC-2800, R307)

e) The CEDM will allow scramming a CEA when subjected to maximum expected deflections during the specified earthquake disturbances.

f) The RSPT was seismically qualified for the intensities of the specified seismic events.

→(DRN 03-2056, R14; EC-2800, R307; EC-1020, R307; EC-8458, R307))
The following is a summary of the significant results of the evaluation for the power uprate to 3716 MWt, including loading conditions attributed to the replacement steam generators, replacement CEDMs and replacement reactor vessel closure head:
←(EC-2800, R307; EC-1020, R307; EC-8458, R307))

a) Damping was verified to be at least as high as the value used in the analysis.

b) Adequacy of the mathematical model was verified by comparison to verified results.

→(EC-2800, R307; EC-8458, R307)

c) The CEDM was analyzed with three (3) limiting nozzle lengths, and all results were enveloped.

←(EC-2800, R307; EC-8458, R307))

→(EC-1020, R307; EC-8458, R307))
The stress analysis results for the CEDM nozzles and J-weld region are within Code allowable limits for these conditions.
←(EC-1020, R307; EC-8458, R307))

←(DRN 03-2056, R14)

3.7.3-15 Analysis Procedure for Damping

The analysis procedure for damping of seismic Category I subsystems is given in Subsections 3.7.3.14 and 3.7.2.15.

3.7.4 SEISMIC INSTRUMENTATION

3.7.4.1 Comparison with Regulatory Guide 1.12

The seismic instrumentation program for Waterford 3 is designed to meet the guidance specified by Regulatory Guide 1.12, Instrumentation for Earthquakes, Revision 1, April 1974. A comprehensive seismic instrumentation program is provided to record any seismic disturbances at the site. The discussions and descriptions below provides comments and clarifications to the regulatory position of Regulatory Guide 1.12. Waterford 3 is not a multiunit site. The safe shutdown earthquake maximum ground acceleration is 0.1g.

Three triaxial time-history accelerometers (T/A's) and two starter units (S/U's) are provided to actuate and operate time-history accelerograph system. One pair of T/A and S/U is located on the containment mat at elevation -34 ft. MSL adjacent to the Reactor Building while a similar pair is located at elevation +50 ft. MSL on the Reactor Building wall.

The T/A's together with S/U's transmit signals to control unit and triaxial time-history accelerograph magnetic tape recorder (M/TR) in the main control room, which provides a record of frequency, amplitude, and phase relationship data in the event of a seismic disturbance.

The two T/A's are vertically separated by an 84 ft. distance. The T/A's are located approximately directly above each other and are oriented along the same axes. The T/A's are located for easy access for maintenance inspection and are mounted rigidly on structures directly connected to the containment structure, such that the accelerograph records are related to the containment structure movement.

One triaxial time-history accelerometer (T/A) is located in free field on its own ground level mat in the yard area near the make-up demineralizer. This T/A will transmit signals to a M/TR recorder in the main control room whenever one of the two S/U's in the Reactor Building triggers.

Two triaxial peak acceleration recorders (P/A's) are located in the Reactor Building. One is located on the reactor coolant pipe connecting the reactor vessel with steam generator No. 2 (elevation +8'2" MSL) and the other is located on safety injection tank 1B at elevation +56 ft. One passive triaxial peak shock recorder (PP/SR) is located on the containment structure at elevation +50 ft. MSL.

One active triaxial peak shock recorder (AP/SR) is located adjacent to the Reactor Building at elevation -34 ft. MSL. The AP/SR permanently records peak accelerations at a number of discrete frequencies.

One triaxial seismic switch trigger unit (S/S) is located adjacent to the Reactor Building at elevation -34 ft. MSL.

The seismic Category I structures, other than the Reactor Building, are not expected to have a significantly different response to an earthquake because they are all founded on a common mat.

One triaxial peak acceleration recorder (P/A) is located at elevation +22 ft. MSL in the Reactor Auxiliary Building. Two passive triaxial peak shock recorders (PP/SR) systems are also located at elevation -34 ft. MSL and elevation +22 ft. MSL.

Adequate indication, recording and annunciation is provided in the seismic monitoring system panel in the main control room, as detailed in Subsection 3.7.4.3.

The instrumentation is designed to perform its function over the range of expected environmental conditions associated with its location at the site.

A plan for the timely utilization of the data obtained from the seismic instrumentation is provided, as discussed in Subsection 3.7.4.4.

3.7.4.2 Location and Description of the Instrumentation

Regulatory Guide 1.12 provides the bases for selection and location of the seismic instrumentation. Locations of the seismic monitoring system instruments are given in Table 3.7-17. The four seismic instrumentation systems are described below.

Triaxial Time-History Accelerograph System

The following are the components of the Triaxial Time History Accelerograph System:

a) Triaxial time - history accelerometer (T/A):

Each T/A continuously senses triaxial acceleration greater than O.Olg from the instant (<100 msec) the S/U activates the system.

b) Triaxial time - history accelerograph system starter unit (S/U):

→(DRN 00-1032, R11-A)

Each S/U is Provided with vertical and horizontal triggers. The S/U actuates the triaxial time - history accelerograph and the magnetic tape recorder upon a p-wave acceleration exceeding a present threshold of O.Olg and will continue to run until approximately eight seconds (adjustable) after the last acceleration above the threshold.

←(DRN 00-1032, R11-A)

c) Triaxial time - history accelerograph magnetic tape recorder (M/TR):

The M/TR, when actuated by the S/U, records the damped response spectra inputs and initiates control room annunciation when the OBE containment foundation design values have been exceeded at any of the frequencies monitored. The recorders are mounted in the control room.

d) Triaxial time - history accelerograph control unit (C/U):

The C/U supplies power to the S/U and has provisions for in-place testing and calibration as a permanent part of the acquired record.

e) Triaxial time - history accelorograph magnetic tape/strip recorder playback unit (P/SR):

This unit is designed to transcribe signals which are received from the T/A's and processed by the M/TR into graphic form.

Triaxial Peak Accelerograph System

The P/A is a self-contained passive device requiring no internal or external power or control connections. The P/A records triaxial peak accelerations. Each peak acceleration axis record is scratched permanently on the recording plate which is removed after the seismic disturbance for data deduction and evaluation.

Triaxial Response Spectrum Recorder System

The following are the components of the Triaxial Response Spectrum Recorder System:

a) Triaxial peak shock recorder (passive) PP/SR:

The PP/SR is a passive device, requiring no internal or external power, which records peak accelerations at a number of discrete frequencies. After the seismic event, the data records from the PP/SR are used to develop a response shock spectrum.

b) Triaxial peak shock recorder (active) AP/SR:

The AP/SR is an active device which permanently records peak accelerations at a number of discrete frequencies. The AP/SR also provides control room alarm contacts for a select number of predetermined acceleration limits which have been exceeded at certain frequencies. After the seismic event, the data records from the AP/SR are used to develop a response shock spectrum.

c) Triaxial peak shock annunciator (P/SA):

The P/SA, located in the main control room, annunciates a preset number of triaxial acceleration limits which are related to its discrete reed frequencies. The P/SA monitors three orthogonal axes utilizing an amber light to indicate when approaching 70 percent of the OBE design limit and a red light to indicate exceeded design limits. The P/SA provides power to the AP/SR.

Triaxial Seismic Switch System

The following are the components of the Triaxial Seismic Switch System:

a) Triaxial seismic switch trigger unit (S/S):

The S/S is of triaxial trigger configuration and provides control room annunciation whenever the SSE zero period acceleration on at least one axis has been exceeded.

b) Triaxial seismic switch control unit (SS/CU):

The SS/CU, located in the control room, houses continuously rechargeable batteries, battery tester, and an ac supply indicator.

3.7.4.3 Main Control Room Operator Notification

The seismic monitoring system panel is located in the main control room. The following instruments are mounted on the panel:

Description	Type	Tag No.
Magnetic Tape Recorder	M/TR	YR-SM-6000
Accelerograph Control Unit	C/U	YZ-SM-6000
Recorder Playback Unit	P/SR	YR-SM-6001
Peak Shock Annunciator	P/SA	YZ-SM-6045
Seismic Switch Control Unit	SS/CU	YZ-SM-6060

A description of each of the above devices is given in Subsection 3.7.4.2.

The 70 percent of the OBE design limit annunciation in the main control room is used by the main control room operator as a warning for the potential extent of the seismic event. At this point, the operator would prepare for a shutdown of the plant.

3.7.4.4 Comparison of Measured and Predicted Responses

The plant operators are provided with procedures and criteria to review the accelerations recorded at the plant site. These criteria consider system design and dynamic analyses in establishing the acceptable levels for continued operation.

Should a seismic event be realized during plant operation and subsequent data analysis reveals the predicted responses are not within ± 20 percent of that measured, action will be taken to re-evaluate the dynamic models utilized for Waterford 3.

For earthquakes which have equaled or exceeded the OBE spectrum, the event will be reported to the NRC for evaluation of the required procedures prior to restart of the plant, as required in 10CFR100, Appendix A.

SECTION 3.7 REFERENCES

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6. Deleted

7. Deleted

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Triaxial Response Spectrum Recorder System

The following are the components of the Triaxial Response Spectrum Recorder System:

a) Triaxial peak shock recorder (passive) PP/SR:

The PP/SR is a passive device, requiring no internal or external power, which records peak accelerations at a number of discrete frequencies. After the seismic event, the data records from the PP/SR are used to develop a response shock spectrum.

b) Triaxial peak shock recorder (active) AP/SR:

The AP/SR is an active device which permanently records peak accelerations at a number of discrete frequencies. The AP/SR also provides control room alarm contacts for a select number of predetermined acceleration limits which have been exceeded at certain frequencies. After the seismic event, the data records from the AP/SR are used to develop a response shock spectrum.

c) Triaxial peak shock annunciator (P/SA):

The P/SA, located in the main control room, annunciates a preset number of triaxial acceleration limits which are related to its discrete reed frequencies. The P/SA monitors three orthogonal axes utilizing an amber light to indicate when approaching 70 percent of the OBE design limit and a red light to indicate exceeded design limits. The P/SA provides power to the AP/SR.

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The following are the components of the Triaxial Seismic Switch System:

a) Triaxial seismic switch trigger unit (S/S):

The S/S is of triaxial trigger configuration and provides control room annunciation whenever the SSE zero period acceleration on at least one axis has been exceeded.

b) Triaxial seismic switch control unit (SS/CU):

The SS/CU, located in the control room, houses continuously rechargeable batteries, battery tester, and an ac supply indicator.

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Accelerograph Control Unit	C/U	YZ-SM-6000
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Peak Shock Annunciator	P/SA	YZ-SM-6045
Seismic Switch Control Unit	SS/CU	YZ-SM-6060

A description of each of the above devices is given in Subsection 3.7.4.2.

The 70 percent of the OBE design limit annunciation in the main control room is used by the main control room operator as a warning for the potential extent of the seismic event. At this point, the operator would prepare for a shutdown of the plant.

3.7.4.4 Comparison of Measured and Predicted Responses

The plant operators are provided with procedures and criteria to review the accelerations recorded at the plant site. These criteria consider system design and dynamic analyses in establishing the acceptable levels for continued operation.

Should a seismic event be realized during plant operation and subsequent data analysis reveals the predicted responses are not within ± 20 percent of that measured, action will be taken to re-evaluate the dynamic models utilized for Waterford 3.

For earthquakes which have equaled or exceeded the OBE spectrum, the event will be reported to the NRC for evaluation of the required procedures prior to restart of the plant, as required in 10CFR100, Appendix A.

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TABLE 3.7-1

DAMPIMG VALUES (PERCENT OF CRITICAL DAMPING)

		Operating Basis Earthquake (OBE)		Safe Shutdown Earthquake (SSE)	
		Waterford	RG1.61	Waterford	RG1.61
Welded Steel Plate Assemblies		1	2	1	4
Steel Containment Vessel		2	2	2	4
Welded Steel Frame Structures		2	2	2	4
Bolted or Riveted Steel Framed Structures		2.5	4	2.5	7
Reinforced Concrete Equipment Supports		2	4	5	7
Reinforced Concrete Frames and Buildings		2	4	5	7
Steel Piping	(12" or less)	0.5	1	1.0	2
	(>12")	0.5	2	1.0	3
Soils (Pleistocene Deposit)		7.5		7.5	

TABLE 3.7-2

SOIL SUPPORTED SEISMIC CATEGORY I STRUCTURES

	Foundation Elevation in ft. MSL*	Top of Structure Elevation in ft. MSL	Total Structural Height in ft.
Reactor Shield Building	-35	214.5	249.5
Reactor Auxiliary Building	-35	46 to 106.5	81 to 141.5
Fuel Handling Building	-35	94	129
Component Cooling Water structures	-35	30	65

* Mat thickness is 12 ft.

TABLE 3.7-3 (Sheet 1 of 2)

INPUT DATA FOR SEISMIC ANALYSIS
HORIZONTAL EXCITATIONS

	Mass Point	Length (ft.)	Area Moment of Inertia ($ft.^4$) N-S	Area Moment of Inertia ($ft.^4$) E-W	Effective Area ($ft.^2$) N-S	Effective Area ($ft.^2$) E-W	Weight (Kip)
Shield Building	1	27.73	2,554,000		401		7,010
	2	21.7	4,058,000		711		4,959
	3	19.7	4,058,000		711		4,318
	4	20.0	4,058,000		711		4,104
	5	25.0	4,058,000		711		4,446
	6	25.0	4,058,000		711		6,242
	7	20.0	4,058,000		711		4,446
	8	22.0	4,058,000		711		4,104
	9	19.0	4,058,000		711		5,301
	10	18.0	4,058,000		711		2,822
	11	17.0	11,782,470		2,262		10,173
Containment Vessel	12	21.5	257,500		98		354
	13	22	527,500		129		376
	14	22	1,031,000		213		376
	15	22	1,420,000		287		668
	16	22	1,723,000		416		1,735
	17	22	1,420,000		287		755
	18	22	1,723,000		287		755
	19	22	1,420,000		287		755
	20	22	1,420,000		287		755
	21	11	1,420,000		287		755
Reactor Bldg. Internal Structure	22	7.3	540,000	190,600	962	494	1,295
	23	7	540,000	190,600	962	494	2,167
	24	11	1,770,000	1,317,000	1,519	670	8,060
	25	12	1,770,000	1,317,000	1,519	670	5,782
	26	14.5	1,876,000	1,353,000	1,737	1,105	9,538
	27	12.5	2,095,820	1,364,900	2,102	2,070	8,855
	28	7	2,080,000	1,607,000	2,096	2,580	7,802
Fuel Handling Bldg.	29	44.5	764,130	1,561,810	292	524	6,853
	30	24.5	1,118,940	2,512,750	725	1,373	10,240
	31	20.0	12, 545,150	45,558,660	2,110	2,160	25,010
	32	36.0	15,630,050	53,700,752	2,262	2,676	33,670
Reactor Auxiliary Building	33	15.5	42,650	10,400	164	68	428
	34	15.5	158,800	16,050	270	68	1,029
	35	23.0	4,009,200	10,607,934	531	660	17,637
	36	25.0	14,056,450	24,867,658	1,017	1,472	34,965
	37	25.0	27,605,870	50,543,260	3,177	3,055	49,093
	38	31.0	38,109,290	71,336,276	3,832	3,973	59,499

TABLE 3.7-3 (Sheet 2 of 2)

INPUT DATA FOR SEISMIC ANALYSIS
HORIZONTAL EXCITATIONS

Foundation Mat

Shape	Length (ft.)	Width (ft.)	Thickness (ft.)	Weight (Kips)	Mass Moment of Inertia (K-ft^2) N-S	Mass Moment of Inertia (K-ft^2) E-W
Rectangular	380	267	12	293,100	3.4440×10^9	1.6244×10^9

Soil Spring Constants

K_{H2} Bearing Spring Const (K/ft.) N-S	K_{H2} Bearing Spring Const (K/ft.) E-W	K_{H1} Sliding Spring Const (K/ft.) N-S	K_{H1} Sliding Spring Const (K/ft.) E-W	Rocking Spring Const (ft.-K/radian) N-S	Rocking Spring Const (ft.-K/radian) E-W	(K^E/ft.^{2})	μ
127,500	156,500	865,000	881,000	38.4×10^9	24×10^9	2764.8	0.5

E: Young's Modulus of Soil

μ: Poisson's Ratio of Soil

K_{H1}: Horizontal or translational spring constant for soils below base mat

K_{H2}: Horizontal or translational spring constant for soils against side faces of base mat**

** By including K_{H2}, the natural period of the structure decreased approximately 7.5%, thereby moving toward the peak response region of the response spectrum. Therefore, it is conservative to include this spring constant in the analysis.

Physical Propeties for Structural Materials

A. Concrete

Modulus of Elasticity:

$$E_c = W^{1.5} 33 \sqrt{f'_c} = 5.11 \times 10^5\ KSF$$

where W = 140 lb./ft.^{3}, f'_c= 4,000 psi

$G_c = E_c / 2(1+\mu) = 2.16 \times 10^5$ KSF

where = $\sqrt{f'_c} / 350 = \sqrt{4{,}000} / 350 = 0.18$

B. Soil

Modulus of Elasticity:

Pleistocene Sediments:

μ= 0.5, G_1 = 6,4000 psi = 921.6 KSF

E_1 = 1.5 x 2 x 921.6 = 2,764.8 KSF

Recent Alluvium:

μ= 0.5, G_2 = 2,300 psi = 331.2 KSF

TABLE 3.7-4 (Sheet 1 of 4)

INPUT DATA FOR SEISMIC ANALYSIS
VERTICAL EXCITATIONS

	Mass No.	Cross-Sectional Area (ft.^{2})	Weight (Kips)	Member Length (ft.)	Floor Stiffness (k/ft.)	Floor Mass Point No.
Shield	1	802	7,010	27.73		
Building	2	1,423	4,959	21.7		
	3	1,423	4,318	19.7		
	4	1,423	4,104	20.0		
	5	1,423	4,446	25.0		
	6	1,423	6,242	25.0		
	7	1,423	4,446	20.0		
	8	1,423	4,104	22.0		
	9	1,423	5,301	19.0		
	10	1,423	2,822	18.0		
	11	4,524	10,173	17.0		
Containment						
Vessel	12	195	354	21.5		
	13	259	376	22.0		
	14	426	376	22.0		
	15	575	668	22.0		
	16	832	1,735	22.0		
	17	575	755	22.0		
	18	575	755	22.0		
	19	575	755	22.0		
	20	575	755	22.0		
	21	575	755	11.0		
Reactor Building	22	1,250	1,295	7.3		
Interal Structures	23	1,250	2,167	7.0		
	24	2,111	7,973	11.0		
	25	2,111	5,682	12.0		
	26	2,623	9,438	14.5	20.6 x 106	29
	27	3,945	8,855	12.5		
	28	3,353	7,802	7.0		
Fuel Handling						
Building	30	840	6,853	44.5		
	31	2,357	10,240	24.5		
	32	2,441	25,010	20.0		
	33	2,408	33,670	36.0		
Reactor Auxiliary						
Building	34	232	428	15.5		
	35	338	1,029	15.5		
	36	1,191	17,637	23.0		
	37	2,489	34,965	25.0		
	38	4,247	49,093	25.0		
	39	6,380	59,499	31.0		

TABLE 3.7-4 (Sheet 2 of 4)

INPUT DATA FOR SEISMIC ANALYSIS
VERTICAL EXCITATIONS

Foundation Mat	Mass No.	Weight (Kips)	Vertical Spring K_z (K/FT)
	40	291,110	1.5076×10^6

Soil Spring Constants

The vertical spring constant considered in the present Waterford 3 studies consists of two parts: one due to normal stress over the base area; another due to shear stress around the side areas.

a) Bearing Spring Constant: K_{z1} (Vertical spring constant for soils below base mat)

$$K_{z1} = \frac{G}{1-\mu} \sqrt{BL}$$

G = 6,400 psi - 921.6 KSF
μ = 0.5
L = 380', B = 267

Shear modulus and Poisson's ratio for pleistocene sediments

L/B = 380/267 = 1.43
β_z = 2.15

$$K_{z_1} = \frac{921.6}{0.5} \times 2.15 \times \sqrt{380 \times 267}$$

= 1,260,988

= 1.260988×10^6 K/ft.

(Reference: "Design Procedures for Dynamically Loaded Foundations," R.V. Whitman and F.E. Richart, Jr. Journal of the Soil Mechanics and Foundation Division, 1967)

b) Sliding Spring Constant: Kx (Vertical spring constant for soils against side faces of base mat)**

$$K_x = 2(1 + \mu)\, G\beta_x \sqrt{BL}$$

G = 2,3000 psi = 331.2 KSF for recent alluvium

μ = 0.5

L is the length of rectangular foundation in the direction of acting force; for side effects L is eqaual to the thickness of the mat.

L = 12', B_1 = 380', B_2 = 267'

L/B1 = 12'/380' = 0.0316 $\beta_{x_1} = 1.0$

** See Table 3.7-3 for the similar reasons to include K_x in the analysis.

TABLE 3.7-4 (Sheet 3 of 4)

INPUT DATA FOR SEISMIC ANALYSIS
VERTICAL EXCITATIONS

L/B_2 = 12'/267' = 0.045 $\beta_{x_1} = 1.0$

K_x = 2 [2(1 + 0.5) x 331.2 x $\sqrt{12 \times 380}$ + 2(1 + 0.5) x 331.2 x $\sqrt{12 \times 267}$]

= 6(331.2 x 67.5 + 331.2 x 56.6)

= 6 x 41,100 = 246,610 K/ft.

Vertical Soil Spring Constant:

K_z = 1,261,000 + 246,600

= 1,507,600

= 1.5076 x 10^6 K/ft.

Lumped Mass Weight of Foundation Mat

W = 297.110^K

Consider Mat as a one degree of freedom structure, the natural period is:

$$\xi = 2\pi \sqrt{\frac{297{,}110}{32.2 \times 1.5076 \times 10^6}} = 0.492 \text{ sec.}$$

Consider the whole mathematical model as a one degree of freedom structure, the natural period for W = 645.930 = 200.60 x 10^2k - sec.^{2}/ft. is:

$$\xi = \frac{2\pi}{100} \sqrt{\frac{200.60}{1.5076}} = 0.724 \text{ sec.}$$

If the shear modulus G increases to 3G, 5G, then becomes

$$\xi = \frac{0.722}{\sqrt{3}} = 0.418 \text{ sec. (for 3G)}$$

$$\xi = \frac{0.722}{\sqrt{5}} = 0.324 \text{ sec. (for 5G)}$$

TABLE 3.7-4 (Sheet 4 of 4)

INPUT DATA FOR SEISMIC ANALYSIS
VERTICAL EXCITATIONS

Pressurizer:

Floor Stiffness:

$K = 870\ E\ I_a/a^2$ a/b = 1 pg. 167, Norris

I_a is the moment of inertia per unit width.

$$I_a = \frac{h^3}{12} = \frac{5^3}{12} = \frac{125}{12}\ ,\ a = 15$$

$$K = 870 \times 511{,}000 \times \frac{125}{12} \times \frac{1}{15^2} = 2.06 \times 10^7\ K/ft.$$

$W = 287^K$

Reference: Structure Design for Dynamic Loads, Charles H. Norris

TABLE 3.7-5

NATURAL FREQUENCIES IN CYCLES PER SECOND (CPS)

	E-W Earthquake		N-S Earthquake		Vertical Earthquake	
Mode No.	G = 16050 psi	G = 6400 psi	G = 16050 psi	G = 6400 psi	G = 16050 psi	G = 6400 psi
1	1.68	1.08	1.68	1.08	2.09	1.33
2	3.35	2.47	3.42	2.51	11.12	11.12
3	5.22	4.54	4.86	4.26	16.03	16.10
4	7.51	7.49	7.42	7.43	19.23	19.29
5	9.10	9.18	8.01	8.40	20.88	20.90
6	11.12	11.15	9.81	9.72	31.10	31.10
7	11.99	11.99	12.07	12.07	32.20	32.22
8	14.37	14.24	13.77	14.16	33.54	33.60
9	16.46	16.57	14.74	15.16	35.61	35.70
10	17.84	17.92	17.20	17.24	48.75	48.76

TABLE 3.7-6

COMPARISON OF STRUCTURAL RESPONSES FOR SEISMIC CATEGORY I STRUCTURES
USING RESPONSE SPECTRA AND TIME HISTORY METHODS

N-S EARTHQUAKE MOTION, SSE
SOIL SHEAR MODULUS = 16050 psi

			Response Spectrum Method (5%)			Time History Method		
	Mass No.	Elevation (Ft.)	Max. Disp (Ft.)	Max Shear (K)	Max. Moment (K-Ft)	Max. Disp (Ft)	Max. Shear (K)	Max. Moment (K-Ft)
Shield Bldg.	1	200.13	0.111	3,035	84,167	0.101	3,245	89,981
	11	-18.0	0.063	17,733	2,439,758	0.056	16,876	2,446,829
Containment Vessel	12	197.50	0.089	111	2,395	0.078	114	2,456
	21	0.0	0.064	1,910	199,028	0.057	1,795	236,807
Reactor Bldg. Internals	22	60.3	0.071	318	2,322	0.063	285	2,080
	28	-4.0	0.063	10,069	355,523	0.057	9,196	548,131
FHB	29	90.0	0.077	1,833	81,575	0.068	1,695	75,414
	32	1.0	0.065	17,820	1,033,242	0.058	16,447	950,300
RAB	33	100.0	0.079	117	1,821	0.070	110	1,699
	38	-4.0	0.065	38,784	2,305,161	0.058	35,818	2,121,269
Mat.	39	-37.24	0.060	147,560	7,723,643	0.054	117,016	9,135,883

TABLE 3.7-7

COMPARISON OF STRUCTURAL RESPONSES FOR SEISMIC CATEGORY I STRUCTURES
USING RESPONSE SPECTRA AND TIME HISTORY METHODS

E-W EARTHQUAKE MOTION, SSE
SOIL SHEAR MODULUS = 16050 psi

			Response Spectrum Method (5%)			Time History Method		
	Mass No.	Elevation (Ft.)	Max. Disp (Ft.)	Max Shear (K)	Max. Moment (K-Ft)	Max. Disp (Ft)	Max. Shear (K)	Max. Moment (K-Ft)
Shield Bldg.	1	200.13	0.123	3,469	96,194	0.119	4,070	112,870
	11	-18.0	0.061	18,925	2,692,084	0.057	19,308	2,967,545
Containment Vessel	12	197.50	0.099	127	2,740	0.093	147	3,153
	21	0.0	0.063	2,024	215,930	0.059	2,123	285,590
Reactor Bldg. Internals	22	60.3	0.074	331	2,414	0.069	323	2,360
	28	-4.0	0.062	10,121	361,124	0.059	9,878	585,990
FHB	29	90.0	0.079	1,885	83,886	0.073	1,915	85,224
	32	1.0	0.064	17,705	1,033,657	0.060	17,392	1,004,382
RAB	33	100.0	0.083	124	1,924	0.077	130	2,012
	38	-4.0	0.063	38,702	2,316,784	0.060	38,171	2,266,303
Mat.	39	-37.24	0.057	145,118	7,642,172	0.054	119,063	9,293,276

TABLE 3.7-8

COMPARISON OF STRUCTURAL RESPONSES FOR SEISMIC CATEGORY I STRUCTURES
USING RESPONSE SPECTRA AND TIME HISTORY METHODS

VERTICAL EARTHQUAKE MOTION, SSE
SOIL SHEAR MODULUS = 16050 psi

			Response Spectrum Method (5%)		Time History Method	
	Mass No.	Elevation (Ft.)	Max. Disp (Ft.)	Max. Force (K)	Max. Disp (Ft)	Max. Force (K)
Shield Bldg.	1	200.13	0.031	1,268	0.032	1,225
	11	-18.0	0.029	1,751	0.028	1,663
Containment Vessel	12	197.50	0.030	62	0.028	59
	21	0.0	0.029	130	0.028	123
Reactor Bldg. Internals	22	60.3	0.029	224	0.028	212
	28	-4.0	0.029	1,341	0.028	1,274
FHB	30	90.0	0.030	1,205	0.029	1,140
RAB	33	1.0	0.030	5,858	0.028	5,548
	34	100.0	0.030	76	0.029	71
	39	-4.0	0.030	10.329	0.028	9,782
Mat.	40	-37.24	0.029	49,989	0.028	48,465

TABLE 3.7-9

COMPARISON OF ACCELERATION FOR SEISMIC CATEGORY I STRUCTURES
USING RESPONSE SPECTRA AND TIME HISTORY METHODS

SSE
SOIL SHEAR MODULUS = 16050 psi

			Response Spectrum Method (5%)			Time History Method		
	Mass No.	Elevation (Ft)	E-W Accel (G)	N-S Accel (G)	Vert Accel (G)	E-W Accel (G)	N-S Accel (G)	Vert Accel (G)
Shield Bldg.	1	200.13	0.498	0.432	0.180	0.546	0.448	0.175
Containment Vessel	12	197.50	0.362	0.314	0.173	0.387	0.320	0.168
Reactor Bldg. Internals	22	60.3	0.256	0.245	0.172	0.235	0.217	0.168
FHB	29	90.0	0.276	0.267	0.176	0.262	0.245	0.167
RAB	33	100.0	0.291	0.274	0.177	0.284	0.254	0.170
Mat.	39	-37.24	0.200	0.210	0.171	0.197	0.197	0.167

TABLE 3.7-10

COMPARISON OF ACCELERATIONOF DYNAMIC ANALYSIS
WITH AND WITHOUT TORSIONAL DEGREE OF FREEDOM

G = 6400 psi, sse, Spectrum Method

	Mass No.	E-W Direction Case I	E-W Direction Case II**	E-W Direction Difference (%)	N-S Direction Case I	N-S Direction Case II**	N-S Direction Difference (%)
Shield Bldg.	1	.29	0.27	-7	.24	0.23	-4
	2	.26	0.25	-4	.22	0.22	0
	3	.25	0.23	-8	.21	0.21	0
	4	.23	0.22	-4	.20	0.20	0
	5	.21	0.21	0	.19	0.19	0
	6	.19	0.19	0	.18	0.18	0
	7	.18	0.17	-5	.17	0.17	0
	8	.17	0.16	-6	.16	0.16	0
	9	.15	0.15	0	.15	0.15	0
	10	.14	0.14	0	.15	0.15	0
	11	.14	0.14	0	.14	0.14	0
Containment	12	.24	0.23	-4	.20	0.20	0
Vessel	13	.23	0.22	-4	.20	0.19	-5
	14	.21	0.21	0	.19	0.18	-5
	15	.20	0.19	-5	.18	0.18	0
	16	.19	0.18	-5	.17	0.17	0
	17	.18	0.17	-6	.17	0.17	0
	18	.17	0.16	-6	.16	0.16	0
	19	.16	0.16	0	.16	0.16	0
	20	.15	0.15	0	.15	0.15	0
	21	.14	0.14	0	.15	0.15	0
Reactor Bldg.	22	.16	0.16	0	.16	0.16	0
Internal	23	.16	0.16	0	.16	0.16	0
Structures	24	.16	0.16	0	.16	0.16	0
	25	.16	0.15	-6	.15	0.15	0
	26	.15	0.15	0	.15	0.15	0
	27	.15	0.14	-6	.15	0.15	0
	28	.14	0.14	0	.1445	0.1468	+1.6
	29	.18	0.16	-10	.17	0.17	0
FHB	30	.16	0.15	-6	.16	0.16	0
	31	.15	0.15	0	.15	0.15	0

* Without torsional degree of freedom

** With torsional degree of freedom

TABLE 3.7-11

MAXIMUM DAMPING RATIOS USED IN ANALYSIS OF NSSS CATEGORY I STRUCTURES, SYSTEMS AND COMPONENTS

Item	Damping ratio, percent of critical viscous damping: Operational Basis Earthquake	Damping ratio, percent of critical viscous damping: Safe Shutdown Earthquake
Equipment and large diameter piping systems, pipe diameter greater than 12 in.	2	3
Small diameter piping system, diameter less than or equal to 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7

→(EC-8458, R307)

TABLE 3.7-12 (Revision 307 07/13)

Benchmark of RCS Model Natural Frequencies – STRUDL (Original Steam Generators) VS. ANSYS (Original Steam Generators)

STRUDL Results from [1]				ANSYS Results from Computer Run No. 3				Comparison	
Mode	Frequency (Hz)	Joint	Direction	Mode	Frequency (Hz)	Node	Direction	Percent Difference	Location
1	2.63	9932	Z	1	2.63	9932	Z	0.0%	Reactor Internals
2	2.63	9932	X	2	2.63	9932	X	0.0%	Reactor Internals
3	7.96	9902	Z	3	7.99	9901	X	-0.4%	Reactor Internals
4	8	9902	X	4	8.17	9901	Z	-2.1%	Reactor Internals
5	13.56	3404	X	5	13.53	3101	X	0.2%	SG 1
6	13.56	404	X	6	13.54	101	X	0.1%	SG 2
8	15.41	4103	X	7	14.73	2105	X	4.4%	Pump 1B and 2B
9	16.05	412	Z	8	15.97	34121	Z	0.5%	SG 1
10	16.06	3412	Z	9	15.98	41201	Z	0.5%	SG 2
11	16.34	2103	X	10	16.34	2105	X	0.0%	Pump 1B and 2B
12	16.55	1103	X	12	16.59	1105	X	-0.2%	Pump 1A and 2A
13	17.2	4103	X	11	16.44	4105	X	4.4%	Pump 1B and 2B
14	18.38	1103	X	14	20.1	5105	X	-9.4%	Pump 1A and 2A
15	21.74	9995	X	15	21.74	9998	X	0.0%	RV lower Mass Pt.
16	22.97	408	X	16	22.85	408	X	0.5%	SG 1 and 2
17	23.34	404	Z	17	23.18	101	Z	0.7%	SG 1
18	23.35	3404	Z	18	23.19	3101	Z	0.7%	SG 2
19	24.85	404	Y	19	24.81	3222	Y	0.2%	SG 1
20	24.85	3404	Y	20	24.82	101	Y	0.1%	SG 2
21	25.68	2103	Z	21	25.64	2105	Z	0.2%	Pump 1B
22	25.68	4103	Z	22	25.64	4105	Z	0.2%	Pump 2B
23	25.7	5103	Z	23	25.66	5105	X	0.2%	Pump 2A
24	25.71	1103	Z	24	25.67	1105	Z	0.2%	Pump 1A
25	29.82	9995	Z	25	29.58	9998	Z	0.8%	RV lower Mass Point
26	31.4	9932	Y	26	30.84	9932	Y	1.8%	Reactor Internals
27	31.84	4580	X	27	33.48	4586	Z	-5.2%	Inlet Piping Pump 2B
28	33.43	2580	Z	28	33.5	2586	Z	-0.2%	Inlet Piping Pump 1B
29	34.74	2580	Z	29	34.75	1580	Z	0.0%	Inlet Piping Pump 1B
30	35.23	1580	Z	30	34.94	5580	Z	0.8%	Inlet Piping 1A
31	36.21	5580	Z	31	35.85	1580	Z	1.0%	Inlet Piping 2A
32	36.9	3412	X	32	36.58	34121	X	0.9%	SG 2
33	37.17	412	X	33	36.96	34121	Z	0.6%	SG 1
34	37.27	3412	Z	34	36.97	41201	Z	0.8%	SG 2
35	37.28	412	Z	35	37.3	41201	X	-0.1%	SG 1
36	37.96	2103	Y	36	37.94	2272	Y	0.1%	Pump 1B
37	37.96	4103	Y	37	37.95	4272	Y	0.0%	Pump 2B
38	38.17	5103	Y	38	38.15	5272	Y	0.1%	Pump 2A
39	38.18	1103	Y	39	38.17	1272	Y	0.0%	Pump 1A
40	43.05	4580	Z	40	43.48	4570	Z	-1.0%	Inlet Piping Pump 2B
41	43.51	2580	Z	41	43.96	2570	Z	-1.0%	Inlet Piping Pump 1B

←(EC-8458, R307)

→(EC-8458, R307)

TABLE 3.7-13

HAS BEEN DELETED

←(EC-8458, R307)

TABLE 3.7-14

MAXIMUM STRESS COMPARISON

Sample Problem	Point	Stress (psi) Static	Stress (psi) Dynamic
No. 1	16	6949	5011
No. 2	25	7455	6973
No. 3	2	6265	5968

TABLE 3.7-15
HAS BEEN DELETED

TABLE 3.7-16 (1 of 2)

REACTOR INTERNALS NON-LINEAR HORIZONTAL SEISMIC MODEL NODE LOCATIONS

Node	Distance from Reference (in)	Node Location Description
1	343.0	Reactor Vessel
2	326.0	Reactor Vessel
3	306.375	Reactor Vessel
4	271.875	Reactor Vessel
5	238.375	Reactor Vessel
6	204.375	Reactor Vessel
7	171.375	Reactor Vessel
8	137.875	Reactor Vessel
9	99.125	Reactor Vessel
10	47.5	Reactor Vessel
11	343.0	Core Support Barrel at Lower Flange
12	326.0	Core Support Barrel at Center of Snubbers
13	306.375	Core Support Barrel
14	271.875	Core Support Barrel
15	238.375	Core Support Barrel
16	204.275	Core Support Barrel
17	171.375	Core Support Barrel
18	137.875	Core Support Barrel
19	99.125	Core Support Barrel
20	47.5	Core Support Barrel
21	0.0	Core Support Barrel at Upper Flange
22	271.875	Core Shroud

TABLE 3.7-16 (2 of 2)

REACTOR INTERNALS NON-LINEAR HORIZONTAL SEISMIC MODEL NODE LOCATIONS

Node	Distance from Reference (in)	Node Location Description
23	238.375	Core Shroud
24	204.375	Core Shroud
25	171.375	Core Shroud
26	137.875	Core Shroud
27	306.375	Center of Core Support Plate
28	319.0	Lower Support Structure
29	306.375	Node Not Used
30	286.68	Fuel
31	266.99	Fuel
32	247.29	Fuel
33	227.6	Fuel
34	207.9	Fuel
35	188.21	Fuel
36	168.51	Fuel
37	148.82	Fuel
38	129.125	Center of Fuel Alignment Plate
39	101.0	CEA Shrouds
40	74.5	CEA Shrouds
41	48.0	CEA Shrouds
42	21.75	Center of Upper Guide Support Structure Plate

TABLE 3.7-17 (1 of 2) Revision 10 (10/99)

SEISMIC MONITORING SYSTEM COMPONENTS

Triaxial Time History Accelerograph System

Quantity	Type	Tag No.	Building and Evaluation	FSAR Figure and Nearest Col. Line Loc.
1	T/A	YT-SM 6000	Adjacent to Reactor Bldg. Elevation -35 ft. MSL	1.2-19 M-4A
1	T/A	YT-SM 6001	Adjacent to Reactor Bldg. Elevation +46 ft. MSL	1.2-17 LY-6A
1	T/A	YT-SM 6002	Free Field Yard Area	1.2-1 (South of Make-up Demineralizer)
1	S/U	YS-SM 6000	Adjacent to Reactor Bldg. Elevation -35 ft. MSL	1.2-19 M-4A
1	S/U	YS-SM 6001	Adjacent to Reactor Bldg. Elevation +51 ft. MSL	1.2-17 LY-6A
1	M/TR	YR-SM 6000	Control Room RAB Elevation +46 ft . MSL	Dwg. G134 G-9A (Panel CP-47)
1	C/U	YZ-SM 6000	Control Room RAB Elevation +46 ft. MSL	Dwg. G134 G-9A (Panel CP-47)
1	P/SR	YR-SM 6001	Control Room RAB Elevation +46 ft. MSL	Dwg. G134 G-9A (Panel CP-47

TABLE 3.7-17 (2 of 2) Revision 10 (10/99)

SEISMIC MONITORING SYSTEM COMPONENTS

Quantity	Type	Tag No.	Building and Evaluation	FSAR Figure and Nearest Col. Line Loc.
Triaxial Peak Accelerograph System				
1	P/A	YR-SM 6020	Reactor Bldg. Elevation +56 ft. MSL	1.2-17 (Safety Injection Tank 1B)
1	P/A	YR-SM 6021	Reactor Bldg. Elevation +23 ft. MSL	1.2-19
1	P/A	YR-SM 6022	RAB Elevation +21 ft. MSL	Dwg. G135 K-4A
Triaxial Response Spectrum Recorder Syste m				
1	PP/SR	YR-SM 6040	Adjacent to Reactor Bldg. Elevation +50 ft. MSL	1.2-17 LY-6A
1	PP/SR	YR-SM 6041	Adjacent to Reactor Bldg. Elevation _34 ft. MSL	1.2-19 M-4A
1	PP/SR	YR-SM 6042	RAB Elevation +22 ft. MSL	Dwg. G135 K-4A
1	AP/SR	YR-SM 6045	Adjacent to Reactor Bldg. Elevation -34 ft. MSL	1.2-19 M-4A
1	P/SA	YZ-SM 6045	Control Room RAB Elevation +46 ft. MSL	Dwg. G134 G-9A (Panel CP-47)
Triaxial Seismic Switch System				
1	S/S	YS-SM 6060	Adjacent to Reactor Bldg. Elevation -34 ft. MSL	1.2-19 M-4A
1	SS/CU	YZ-SM 6060	Control Room RAB Elevation +46 ft. MSL	Dwg. G134 G-9A (Panel CP-47)

TABLE 3.7-18

COMPARISON OF MAXIMUM RESPONSES

(OBE, REACTOR SHIELD BUILDING ELEMENT
NO. 347 @ EL. +184.0 FT. MSL)

		AXIAL FORCES Kips/Ft		SHEAR FORCES Kips/Ft		MOMENTS Ft-Kips/Ft	
		Fx	Fy	Fxz	Fyz	Mx	My
Max Response Compo-nents (±)	N-S	0.53	0.08	0.046	0.05	0.03	0.12
	E-W	10.95	1.56	0.015	0.46	0.41	1.64
	V	6.67	2.45	0.035	0.79	0.64	3.35
Absolute Sum Method		17.62	4.01	0.081	1.25	1.05	4.99
SRSS Method		12.83	2.91	0.060	0.92	0.76	3.73

TABLE 3.7-19

COMPARISON OF MAXIMUM RESPONSES

(OBE, REACTOR SHIELD BUILDING ELEMENT
NO. 947 @ EL. +61.0 FT. MSL)

		AXIAL FORCES Kips/Ft		SHEAR FORCES Kips/Ft		MOMENTS Ft-Kips/Ft	
		Fx	Fy	Fxz	Fyz	Mx	My
Max Response Components (±)	N-S	0.34	2.44	0.12	0.05	0.003	0.02
	E-W	3.46	24.14	0.16	0.60	0.150	0.10
	V	0.06	9.22	0.07	0.22	0.090	0.04
Absolute Sum Method		3.52	33.36	0.23	0.82	0.240	0.14
SRSS Method		3.48	25.96	0.21	0.64	0.170	0.11

TABLE 3.7-20

COMPARISON OF MAXIMUM RESPONSES

(OBE, REACTOR SHIELD BUILDING ELEMENT
NO. 1287 @ EL. 0.0 FT. MSL)

		AXIAL FORCES Kips/Ft		SHEAR FORCES Kips/Ft		MOMENTS Ft-Kips/Ft	
		Fx	Fy	Fxz	Fyz	Mx	My
Max Response Components (±)	N-S	0.56	0.10	0.0044	0.03	0.011	0.05
	E-W	3.33	42.61	0.0003	0.09	0.070	0.35
	V	0.38	12.92	0.0001	0.03	0.075	0.21
Absolute Sum Method		3.71	55.53	0.0045	0.12	0.145	0.56
SRSS Method		3.40	44.53	0.0040	0.10	0.103	0.41

TABLE 3.7-21 Revision 10 (10/99)

LINES ANALYZED BY CHART METHOD

BORON MANAGEMENT

→

The following lines analyzed for the Boron Management System by Chart Method are located on Figure 11.2-1 with the exception of Line No. 3BM1-305 which is located on Figure 9.3-6 (for Figure 9.3-6, Sheet 1, refer to Drawing G168, Sheet 1).

3BM1-32A/B
3BM1-397
←

CHEMICAL AND VOLUME CONTROL

→
The following lines analyzed for the Chemical and Volume Control System by Chart Method are located on Figure 9.3-6 (for Figure 9.3-6, Sheet 1, refer to Drawing G168, Sheet 1).
←

2CH1-147A/B
2CH1-150A/B

3CH1-161B
3CH1-162A
2CH3/4-166
2CH3/4-170
2CH1/2-174
2CH3/4-231

2CH3-66A/B
2CH1-72A/B
2CH3-86A
2CH3-86B
2CH3-86A/B
2CH3-91A
2CH3-91B
2CH3-91A/B
2CH2-104A/B
2CH3/4-110A
2CH3/4-110B
2CH3/4-110A/B
2CH3/4-116A/B
2CH1/2-122A/B
3CH1/2-123A/B
2CH1-124A/B

WASTE MANAGEMENT

The following lines analyzed for the Waste Management System by Chart Method are located on Figures 11.2-2 and 11.3-1.

7WM1-117A/B
7WM1-125A
7WM1-125B
7WM1-125A/B
7WM1-126A/B
7WM1-137A
7WM1-137B

7WM1-137A/B