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# Seismic Analysis of Safety-Related Nuclear Structures and Commentary

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# Seismic Analysis of Safety-Related Nuclear Structures

## 1.0 GENERAL

## 1.1 INTRODUCTION

### 1.1.1 Purpose

This standard provides minimum requirements and acceptable methods for the seismic analyses of safety-related structures of a nuclear facility. This standard provides a methodology for calculating seismic responses in structures and to derive input motions for use in the seismic qualification of electrical and mechanical systems and components.

The purpose of the analytical methods is to provide only small levels of conservatism to account for uncertainties. The intentional conservatism is contained in the following three areas:

1. For soil-structure interaction, three cases are analyzed using different soil modulus values and the results use the envelope of the three cases.
2. For in-structure response spectra, the peaks are broadened.
3. For structural damping, conservative values are specified.

As a result, the output from the analyses using these methods will be at a slightly greater probability of non-exceedance than that of the input. For example, the seismic responses will have about a 90% chance of not being exceeded for an input response spectrum specified at the 84th percentile non-exceedance level.

### 1.1.2 Scope

#### *1.1.2.1 Types of Structures Covered by This Standard*

This standard is intended for use in the seismic analysis of all safety-related structures of nuclear facilities including, but not limited to, above and below ground structures, buried piping, above ground vertical tanks and structures with seismic isolation systems. Analysis of caisson and pile-supported foundations, unlined tunnels, and floating structures are not covered by this standard. However, nothing in this standard should be considered to preclude the use of these structures and structural elements.

#### *1.1.2.2 Foundation Material Stability*

The analysis procedures provided herein assume that the structures analyzed are adequately supported by their foundation materials and that no soil or rock failure occurs that would modify or void the seismic analysis.

### 1.1.3 General Requirements

#### *1.1.3.1 Use of Analysis Results*

The seismic responses determined from the analyses prescribed herein are to be combined with responses due to dead load and other prescribed loads.

#### *1.1.3.2 Alternative Methodologies*

Techniques other than those specified in this standard, including experience gained from past earthquakes, special analyses, and testing, may be used in lieu of the requirements specified herein. However, such alternative methodologies shall be properly substantiated and shall conform to the intent of this standard as expressed in the preface.

## 1.2 DEFINITIONS

The following terms are defined for general use in this standard. Specialized definitions also appear in some individual sections.

**Apparent wave propagation velocity:** The apparent propagation velocity of seismic waves through the ground relative to a fixed local coordinate system on the object analyzed.

**Competent soil:** Any natural or improved soil that has a shear wave velocity, $V_s \geq 1{,}000$ fps (300 m/s).

**Coupled:** A descriptive term for mathematical models of structures and components that are interconnected and which influence the dynamic response of each other.

**"Cut-off" frequency:** The highest frequency which is adequately represented in the model for the soil structure interaction analysis procedure. It may be taken as twice the highest dominant frequency of the coupled soil-structure system for the direction under consideration, but not less than 10 Hz.

**Design (or evaluation) ground acceleration:** The value of the acceleration which corresponds to acceleration at zero period in the design ground-response spectrum.

**Design (or evaluation) response spectrum:** A smooth response spectrum of the free-field input mo-

$\{\dot{X}\}$ = column vector of relative velocities ($n \times 1$);
$\{\ddot{X}\}$ = column vector of relative accelerations ($n \times 1$);
$\{U_b\}$ = influence vector; displacement vector of the structural system when the support undergoes a unit displacement in the direction of the earthquake motion ($n \times 1$);
$n$ = number of dynamic degrees of freedom;
$\ddot{u}_g$ = ground acceleration.

(b) Eq. 3.2-1 may be solved using the modal superposition or direct integration time history methods.

*3.2.2.2.1 Modal superposition*

(a) The modal-superposition method may be used when the equations of motion (Eq. 3.2-1) can be decoupled using the transformation:

$$\{X\} = [\phi]\{Y\} \qquad \text{(Eq. 3.2-2)}$$

where

$[\phi]$ = normalized mode shape matrix; $[\phi]^T[M][\phi] = [{}^\backslash 1_\backslash]$ [This is an ($m \times m$) identity matrix];
$\{Y\}$ = vector of normal, or generalized, coordinates ($m \times 1$);
$m$ = number of modes considered.

(b) The transformation of Eq. 3.2-2 will decouple the equation of motion (Eq. 3.2-1) when terms like $\{\phi_i\}^T[C]\{\phi_j\}$, $i \neq j$, are small and can be neglected. This approximation is used in most practical cases including the structural systems with composite damping described in Sections 3.1.5.2 and 3.1.5.3. When experience shows that such an approximation is inappropriate, or a more accurate analysis is desired, a method which accounts for nonclassically damped systems may be used.

(c) The decoupled equation of motion for each mode may be written as:

$$\ddot{Y}_j + 2\lambda_j\omega_j\dot{Y}_j + \omega_j^2Y_j = -\Gamma_j\ddot{u}_g \qquad \text{(Eq. 3.2-3)}$$

where

$Y_j$ = generalized coordinate of $j$th mode;
$\lambda_j$ = damping ratio for the $j$th mode expressed as fraction of critical damping;
$\omega_j$ = circular frequency of $j$th mode of the system (rad/s);
$\Gamma_j$ = modal participation factor of the $j$th mode;

$$= \frac{\{\phi_j\}^T[M]\{U_b\}}{\{\phi_j\}^T[M]\{\phi_j\}} = \{\phi_j\}^T[M]\{U_b\} \qquad \text{(Eq. 3.2-4)}$$

(when mass normalized so denominator equals one).

The single-degree-of-freedom equations shall be integrated using a proven technique, such as those listed in Table 3.2-1.

(d) The techniques used for determining mode shapes and frequencies shall have convergence checks to ensure accuracy.

(e) It shall be sufficient to include all the modes in the analysis having frequencies less than the ZPA frequency, provided that the residual rigid response due to the missing mass is calculated from Eq. 3.2-5 and is combined algebraically with the response from Eqs. 3.2-2 and 3.2-3.

$$[K]\{X_r\} = -[M]\left\{\{U_b\} - \sum_{i=1}^{m} \Gamma_i\{\phi_i\}\right\}\ddot{u}_g \qquad \text{(Eq. 3.2-5)}$$

(f) Alternatively, the number of modes included in the analysis shall be sufficient to ensure that inclusion of all remaining modes does not result in more than 10% increase in total responses of interest.

*3.2.2.2.2 Direct integration*

(a) Direct integration of the equations of motion (Eq. 3.2-1) may be used. ~~Either implicit or explicit~~ methods of numerical integration may be used to solve the equations of motion.

*3.2.2.3 Nonlinear Methods*

(a) When performing a ~~nonlinear analysis, the~~ following shall be considered:

1. Geometric nonlinearities that significantly alter the effective system geometry, such as large displacements or significant gaps;
2. Material nonlinearities, such as plasticity or friction, in the range of response under consideration.

(b) The direct-integration and modal-superposition procedures (when appropriate) are acceptable methods to use for solution.

(c) Nonlinear analyses, shall, in general, consider all three components of earthquake motion, which shall be considered to act simultaneously unless it can be shown that individual component responses are uncoupled.

(d) In general, more than one set of acceleration time histories, meeting the requirements of Section

2.3, should be used, and the results of the analyses shall be averaged.

### 3.2.3 Response Spectrum Method

#### *3.2.3.1 Linear Methods*

(a) When the response spectrum method is used, the basic equations of motion given by Eq. 3.2-1 shall be uncoupled using the linear coordinate transformation of Eq. 3.2-2 and represented by the uncoupled, individual equation for each mode as given by Eq. 3.2-3.

(b) The generalized response of each mode shall be determined from:

$$Y_j(\max) = \Gamma_j \left(\frac{S_{aj}}{\omega_j^2}\right) \qquad \text{(Eq. 3.2-6)}$$

where $S_{aj}$ is the spectral acceleration corresponding to frequency $\omega_j$.

(c) The maximum displacement of node $i$ relative to the base due to mode $j$ is:

$$X_{ij}(\max) = \phi_{ij} Y_j(\max) \qquad \text{(Eq. 3.2-7)}$$

(d) In performing the calculations using Eqs. 3.2-6 and 3.2-7, and in calculation of the response quantities, the signs of the participation factor, $\Gamma_j$, the maximum generalized coordinate, $Y_j(\max)$, the maximum displacement of node $i$ relative to the base due to mode $j, X_{ij}(\max)$, and other response quantities, shall be retained.

(e) Include all the modes in the analysis having frequencies less than the ZPA frequency or cutoff frequency, provided that the residual rigid response due to the missing mass calculated from Eq. 3.2-8 is added.

$$[K]\{X_r(\max)\} = M \times \left\{\{U_b\} - \sum_{i=1}^{n} \Gamma_i\{\phi_i\}\right\} S_{A\max} \qquad \text{(Eq. 3.2-8)}$$

where

$S_{A\max}$ = highest spectral acceleration in the interval between the cut-off frequency and ZPA.

Alternatively, the number of modes to be included in the analysis shall be determined as in Section 3.2.2.2.1(f).

(f) For modal combination purposes the residual rigid response $\{X_r(\max)\}$ shall be considered as an additional mode having a frequency equal to the ZPA or cutoff frequency.

(g) Individual modal and component responses shall be combined in accordance with the requirements of Section 3.2.7.

#### *3.2.3.2 Nonlinear Methods*

The response spectrum method cannot be applied in a rigorous manner to nonlinear multi degree-of-freedom systems because superposition of modes is no longer valid; however, there are approximate methods which may be used with adequate accuracy.

### 3.2.4 Complex Frequency Response Method

#### *3.2.4.1 General Requirements*

When the complex frequency response method is used for seismic time history analysis, the following requirements shall be met:

(a) The time interval for the input time history shall be chosen so that the maximum frequency of interest is retained.
(b) The frequency interval for calculation of transfer functions shall be selected to accurately define the transfer functions at structural frequencies.
(c) A quiet zone (trailing zeros) shall be added to the excitation time history. The quiet zone shall be long enough to damp out the transient response to ensure zero initial conditions.
(d) The transfer functions shall be established at a minimum 150 points in the 0 to ZPA frequency range unless the use of a lesser number of points or a lower upper frequency limit is justified.

#### *3.2.4.2 Response Time History*

When the complex frequency response method is used, the response time history, $R(t)$, may be expressed as:

$$R(t) = \frac{1}{2\pi}\int_{-\infty}^{+\infty} R(\omega)e^{i\omega t}\, d\omega \qquad \text{(Eq. 3.2-9)}$$

where $R(\omega)$ is the response in the frequency domain and is given by:

$$R(\omega) = T(\omega)\ddot{u}_g(\omega) \qquad \text{(Eq. 3.2-10)}$$

where

$T(\omega)$ = transfer function for the structure at circular frequency $\omega$;
$\omega$ = circular frequency;

**TABLE 3.3-1. Lumped Representation of Structure-Foundation Interaction at Surface for Circular Base**

| Motion | Equivalent Spring Constant | Equivalent Damping Coefficient |
|---|---|---|
| Horizontal | $k_x = \dfrac{32(1-\nu)GR}{7-8\nu}$ | $c_x = 0.576k_xR\sqrt{\rho/G}$ |
| Rocking | $k_\psi = \dfrac{8GR^3}{3(1-\nu)}$ | $c_\psi = \dfrac{0.30}{1+B_\psi}k_\psi R\sqrt{\rho/G}$ |
| Vertical | $k_z = \dfrac{4GR}{1-\nu}$ | $c_z = 0.85k_zR\sqrt{\rho/G}$ |
| Torsion | $k_t = 16GR^3/3$ | $c_t = \dfrac{\sqrt{k_tI_t}}{1+2I_t/\rho R^5}$ |

Notes: $\nu$ = Poisson's ratio of foundation medium; $G$ = shear modulus of foundation medium; $R$ = radius of circular basemat; $\rho$ = mass density of foundation medium; $B_\psi = 3(1-\nu)I_0/8\rho R^5$; $I_0$ = total mass moment of inertia of structure and basemat about the rocking axis at the base; and $I_t$ = polar mass moment of inertia of structure and basemat.

fixed base analysis of the flexible structure representation.

*3.3.1.2 Spatial Variations of Free-Field Motion*

(a) Vertically propagating shear and compressional waves may be assumed for an SSI analysis provided that torsional effects due to nonvertically propagating waves are considered. The consideration of an accidental eccentricity of 5% of the structure's plan dimension, as discussed in Section 3.1.1, will fully account for the torsional effects.

(b) Variation of amplitude and frequency content with depth may be considered for embedded structures.

*3.3.1.3 Three-Dimensional Effects*

The three-dimensional phenomenon of radiation damping and layering effects of foundation soil shall be considered in SSI analysis.

*3.3.1.4 Nonlinear Behavior of Soil*

The nonlinear behavior of soil shall be considered and may be approximated by equivalent linear material properties. Two types of nonlinear behavior may be identified: primary and secondary nonlinearities. "Primary nonlinearity" denotes nonlinear material behavior induced in the soil due to the excitation alone, i.e., ignoring structure response. "Secondary nonlinearity" denotes nonlinear material behavior induced in the soil due to structural response as a result of SSI. Primary nonlinearities shall be considered in the SSI analysis. Except for the provisions o tion 3.3.1.9, secondary nonlinearities, including nonlinear behavior in the vicinity of the soil-structure interface, need not be considered.

*3.3.1.5 Structure-to-Structure Interaction*

Structure-to-structure interaction may be generally neglected for overall structural response but shall be considered for local effects due to one structure on another, such as required in Section 3.5.3 for walls.

*3.3.1.6 Effect of Mat and Lateral Wall Flexibility*

The effect of mat flexibility for mat foundations and the effect of wall flexibility for embedded walls need not be considered in the SSI analysis performed to establish seismic responses.

*3.3.1.7 Uncertainties in SSI Analysis*

The uncertainties in the SSI analysis shall be considered. In lieu of a probabilistic evaluation of uncertainties, an acceptable method to account for uncertainties in SSI analysis is to vary the low strain soil shear modulus. Low strain soil shear modulus shall be varied between the best estimate value times $(1 + C_v)$ and the best estimate value divided by $(1 + C_v)$, where $C_v$ is a factor that accounts for uncertainties in the SSI analysis and soil properties. If sufficient, adequate soil investigation data are available, the mean and standard deviation of the low strain shear modulus shall be established for every soil

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