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STRUCTURAL BUILDING RESPONSE REVIEW

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SEISMIC SAFETY MARGINS RESEARCH PROGRAM

LAWRENCE LIVERMORE LABORATORY

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REPORT SL-3759

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Dear Mr. Eaton:

We are sending herewith twenty-five (25) copies and one camera ready copy of the following report:

Report SL-3759
Structural Building Response Review
Dated January 15, 1980

This report describes the structural response analysis method, including the structural model, soil structure interaction as it relates to structural models, methods for seismic structural analysis, numerical integration methods, methods for non-seismic response analysis approaches to combine various responses, structural damping values, nonlinear response, uncertainties in structural properties, and structural response analysis using random properties. The report presents the state-of-the-art in these areas for nuclear power plants. It also details the past studies made at Sargent & Lundy to evaluate different alternatives and the conclusions reached for the specific purposes that those studies were intended.

Yours very truly,

J. M. McLaughlin
Manager
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JMM/ens
Enclosures

COPY

SEISMIC SAFETY MARGINS RESEARCH PROGRAM*

(PHASE I)

Project IV - Structural Building Response

Structural Building Response Review

January 15, 1980

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STRUCTURAL BUILDING RESPONSE REVIEW

LAWRENCE LIVERMORE LABORATORY

1. INTRODUCTION

The integrity of a nuclear power plant during a postulated seismic event is required to protect the public against radiation. Therefore, a detailed set of seismic analyses of various structures and equipment is performed while designing a nuclear power plant.

Seismic Category I and other applicable structures are designed for two levels of earthquakes: the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE).

The SSE is selected based on geological and seismological investigations of the region, including earthquake history, potential sources of earthquakes and attenuation of seismic motion from source to site. For identifiable earthquake sources, e.g., faults, the maximum potential earthquake is assumed to occur at the closest approach to the site and the site motion is determined after considering appropriate attenuation. For no identifiable sources, the historically recorded earthquake of maximum intensity is assumed to occur in the site vicinity and the resulting ground motion is determined using appropriate intensity-acceleration correlations.

The OBE selected is based on the seismic risk analysis performed for the site. The risk analysis considers the earthquake history of the region, potential sources of earthquakes, relative frequency of occurrence of different intensities (magnitudes) of past earthquakes, attenuation relationships and the uncertainties of each of these variables. The OBE selected is based on a reasonable probability of occurrence in the operating life of the plant.

After establishing the levels for the two design earthquakes in terms of maximum ground acceleration, their characteristics are defined according to US NRC Regulatory Guide 1.60 (1) as a wide band smooth response spectrum. Alternatively, site

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specific spectrum can also be used on an individual review basis. A synthetic time history corresponding to design response spectra is also obtained such that the time history has a response spectrum compatible with the given smooth response spectra (2). The requirements for this compatibility are given in Standard Review Plan Section 3.7.1 (3). Once the input is defined, the seismic analysis consists of soil-structure interaction analysis, structural response analysis and subsystem response analysis.

This report describes the structural response analysis method, including the structural model, soil-structure interaction as it relates to structural models, methods for seismic structural analysis, numerical integration methods, methods for non-seismic response analysis approaches for various response combinations, structural damping values, nonlinear response, uncertainties in structural properties, and structural response analysis using random properties. The report describes the state-of-the-art in these areas for nuclear power plants. It also details the past studies made at Sargent & Lundy to evaluate different alternatives and the conclusions reached for the specific purposes that those studies were intended. These results were incorporated here because they fall into the general scope of this report. The scope of the present task does not include performing new calculations.

2. STRUCTURAL MODEL

2.1 Introduction

Structures such as the reactor building, auxiliary building, and fuel handling building and support equipment are essential for the safe shutdown of the plant. They are classified as Seismic Category I structures and their integrity is ensured even under extreme environmental and accidental loading conditions. Other structures, such as the turbine building and service building, are designed as normal industrial structures.

Detailed seismic analyses are performed only for Seismic Category I structures. However, if the plant is laid out such that there is considerable interaction under seismic loads between the Seismic Category I structures and other structures, then the non-Seismic Category I structures are also included in the detailed seismic analyses. When the plant layout is such that the failure of a non-Seismic Category I structure under extreme seismic loading will affect the integrity of any Seismic Category I structure or equipment, the non-Seismic Category I structure or equipment is also included in the detailed seismic analysis.

The dynamic response of a structure subjected to a given earthquake excitation is governed by its mass, its stiffness and its interaction with the foundation. To predict the structural behavior during a seismic event, a proper distribution of mass and stiffness of the structure is essential. The nuclear power plant structures are modeled as a multi-degree-of-freedom system. The major structural elements, such as floor slabs, foundation mat, roof slab, shear walls and braced vertical frames, are included in the model. The following sections discuss the different modeling techniques.

2.2 Models for Axisymmetric Structures

Axisymmetric power plant structures such as the chimneys and containment building can be modeled using axisymmetric shell elements or beam elements. The choice of element depends on the physical configuration of the structure.

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For horizontal excitations, axisymmetric structures with a large height-to-radius ratio can be adequately modeled by beam elements, as shown in Subsection 2.5.1, where the beam representation of chimneys and containment structures is found to be acceptable for horizontal seismic response computations. The same shell model can be used for both the vertical and the horizontal response analyses. When a beam model is used for vertical excitations, a separate model is required to compute slab panel responses to vertical excitation. The vertical model in this case is similar to that for the auxiliary, turbine and fuel buildings and is discussed in the next section.

2.3 Models for Auxiliary-Turbine-Fuel Building Complex

The auxiliary-turbine-fuel handling building is a three-dimensional structure. A finite element model would consist of plate elements for shear walls and slab panels and beam elements for columns and floor girders. It is impractical to use a finite element model for power plant structures because they are so large and complex; therefore, they are generally modeled using space frame or slab-shear wall models.

The horizontal excitation induces bending and shear in shear walls and columns and axial load and shear forces in the horizontal slabs. The vertical motions induce axial loads in shear walls and columns and bending in the floor slabs. Thus, the horizontal and vertical analyses can be, and generally are, separate. The horizontal model properly accounts for the bending, shear and torsional stiffness and mass characteristics. The vertical model properly accounts for the axial stiffness of shear walls and columns and the flexural behavior of the slabs. The horizontal and vertical models are discussed in the following sections.

2.3.1 Horizontal Model

The auxiliary-turbine building complex consists mainly of thick, concrete floor slabs supporting heavy equipment, shear walls and vertical braced frames. Two modeling techniques are used to simulate the horizontal behavior of these structures: the Slab-Shear Wall Model and the Space Frame System Model.

Slab-Shear Wall Model

A typical horizontal slab-shear wall system for a typical BWR nuclear power plant structure is shown in Exhibit 2.1. Each floor slab is modeled as a rigid horizontal diaphragm having three in-plant degrees of freedom, two horizontal translations and one rotation about the vertical axis. For rocking, each slab can move two additional degrees of freedom (rotation about two horizontal axes). However, since the bending stiffness of the shear walls is large, all slabs can be assumed to rock by the same angle. Thus, if there are N slabs in the system, the total independent degrees of freedom for slabs is $3N+2$. The mass of a slab and the mass of the equipment and structural components it supports are lumped at its centroid. The mass of a shear wall is distributed among all the floor slabs to which the wall is connected.

Each shear wall is modeled as a set of massless elastic shear springs, one spring between two consecutive floor slabs. The stiffness of the shear spring is given by

$$K = 1/D_t \quad (2.1)$$

where

$$D_t = D_f + D_s$$

and

$$D_f = \text{flexure deflection/unit load} \\ = h^3/12EI$$

$$D_s = \text{shear deflection/unit load} \\ = h/GA$$

in which

- h = height of shear wall
- E = modulus of elasticity
- G = shear modulus
- A = shear area
- I = moment of inertia

Similarly, each braced vertical frame is modeled as a massless elastic shear spring between floors. To simulate the correct torsional stiffness, the locations of these springs in the model should represent the physical location of the shear wall or braced frame in the plant.

Space Frame Model

A space frame model can also be used to simulate the dynamic behavior of power plants. The model consists of mass points joined by flexural members. The mass points represent the slabs and are placed at the slab centroids. The mass components given at these points are those of the actual structure. The flexural members simulate the bending, shear and torsional resistance of the actual shear walls. To correctly simulate the eccentricity between the mass and stiffness centers, the ends of the flexural members are connected to the mass points by horizontal rigid members.

2.3.2 Vertical Model

A typical vertical model is shown in Exhibit 2.2. It consists of vertical elements representing the axial behavior of the shear walls and column systems, and horizontal elements representing the flexural behavior of slab panels. The stiffness for the vertical elements corresponds to the axial stiffness of the walls and columns connecting the various slabs. The mass lumped at each floor elevation consists of the contributory mass of walls and slabs. The slab mass lumped at these wall nodes generally represents half the total slab mass. The other half of the slab mass is distributed among the horizontal elements representing the flexural behavior of the slab panel. Frequencies of these horizontal elements are taken to be equal to the single panel frequency of various panel elements in the floor. With the mass and frequency of each panel thus determined, the properties of horizontal elements in Exhibit 2.2 are defined for purposes of analysis.

A typical floor model is shown in Exhibit 2.2. To represent the multi-period characteristics of such a system, the following procedure is normally used:

- A. Based on the actual geometry of the structure, the representative panels and beams of each floor system are selected. Then the appropriate mass for each panel or beam supporting various equipment is calculated. The fundamental frequency of each panel or beam is calculated using approximate methods for dynamic response. These panels and beams are represented in the dynamic model by a set of single-degree-of-freedom oscillators.

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- B. To account for the entire mass of a typical floor, one half of the floor mass is equally divided among the single-degree-of-freedom oscillators and the other half is lumped at each of the two supports (shear walls).
- C. After computing the frequency ω in Step 1 and the mass m in Step 2, the stiffness k of each oscillator is computed by $k=m\omega^2$.

Because of the large number of composite beams and numerous irregularities in the floor geometry, sometimes a grid system is constructed for the composite floor in the vertical seismic analysis (4). The floor is modeled as an equivalent plain grid with the masses of the slabs and beams lumped along the length of the beams. Depending on the span, several lumped mass points may be selected along each beam. The stiffness of the members connecting these masses is computed for the composite concrete slab and beams effective along that length. Concrete slabs spanning the steel beams are modeled by equivalent beam members having only the slab section properties. This grid system to represent the composite floor can be analyzed for vertical seismic loading. A typical composite floor in the auxiliary building of a nuclear power plant is shown in Exhibit 2.3. The corresponding mathematical model is shown in Exhibit 2.4.

2.4 Modeling Considerations

The details of a mathematical model to predict the structural dynamic response behavior depend on the complexity of the real structure and the design requirements. The information required from the dynamic analysis is a primary consideration in constructing the mathematical model. For example, in addition to predicting the forces which develop in the structure during an earthquake, it is often necessary to check the clearance provided between major internal components. Therefore, enough joints should be modeled on the structure to properly account for the required clearance. For structures such as containment, biological shield and reactor pedestal structures having continuous mass distributions, enough mass points should be chosen so that the significant vibration modes can be adequately defined and the dynamic response can be accurately predicted. Of course, an increase in the

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number of joints in the structure will increase the degrees of freedom in the mathematical model, and thus increase the computational effort and cost. In other words, cost consideration and accurate representation of a real structure should be balanced and optimized when developing a mathematical model.

A rigid body normally has six degrees of freedom. However, some of the dynamic degrees of freedom can be neglected and condensed in most structures, or can be uncoupled so that separate analyses can be performed for different types of motion. For example, rotatory inertia can generally be neglected in the horizontal model because its contribution to the total response of the system is small compared to the contribution from translational inertia.

All subsystems, such as equipment and piping, are decoupled from the floor which supports them, since the structural mass of the supporting floor is large compared to the subsystem masses. However, their masses are included with the structural mass of the supporting floor slabs in the system model.

The floor slabs in building structures, along with the heavy equipment resting on them, have an asymmetric mass-stiffness distribution. Therefore, torsional coupling between the two horizontal motions occurs. The degree of coupling depends on the amount of eccentricity between the center of mass and center of rigidity of the floor slab. If the eccentricities are small, the coupling effects between translation and torsion can be neglected. Normally, the mass matrix is a diagonal matrix representing the mass of the structure lumped at the mass point locations. However, if the structure is continuous and not physically lumped at node points, a mass matrix consistent with the displacements of the system may contain off-diagonal terms.

2.5 In-house Study Examples

As described in the previous sections, power plant buildings can be modeled by different mathematical approaches. The structure's complexity and the

response of interest determine the type of idealization. Sargent & Lundy has made extensive studies on various modeling techniques. Some examples are given below:

2.5.1 Finite Element Idealization vs. Beam Element Model

Shell structures, such as containment vessels and chimneys, subjected to earthquake excitation have usually been modeled as cantilever beams. The beam properties are determined from the shell cross section. Lumped masses, as determined from the dead weight of the shell, are distributed along the beam axis.

The containment as shown in Exhibit 2.5 was modeled as a cantilever beam and as a shell element to determine how the beam model can represent the shell. The containment is connected to a relatively rigid base-slab which is founded on rock; both models are completely restrained at the base and excited with 0.09 g peak ground acceleration. Ten modes of vibration are considered for analysis. The forces and stresses computed from the shell and beam models are compared in Tables 2.1 and 2.2. The results show that the two models are comparable when attention is focused on the meridional modes and forces.

To show the consequences of the type of modeling for a slender structure, a chimney 689'-9" in height was analyzed. The radius varies from 42' at the foundation to 18'-3" at the top. The thickness varies from 18.96" at the bottom to 9" at the top. A bar graph comparison of frequencies is shown in Exhibit 2.6. The modal shapes for the first five modes are shown in Exhibits 2.7 through 2.11. As expected, due to the beam-like shape of the chimney structure, the frequencies and modal shapes for the two different models compare well.

From the analysis of the above two structures, it can be concluded that the greater the slenderness ratio (height/radius), the closer the correspondence between the frequencies and modal shapes obtained from the beam and shell models.

2.5.2 Slab-Shear Wall Model vs. Space Frame Model

It was mentioned in Subsection 2.3.1 that a space frame model can be used to simulate the dynamic behavior of a system of horizontal slabs connected with vertical shear walls for horizontal ground motions. Two systems are dynamically equivalent if the structural properties of the composite elements (joints, rigid members and vertical columns) of the two models are properly calculated. Exhibits 2.12 and 2.13, respectively, show a slab-shear wall model and its corresponding space frame model of the reactor, containment, turbine and auxiliary building complex of a BWR nuclear generation station. Included with the containment model are the reactor pedestal and biological shield wall. Modal frequencies and participation factors of the coupled shear wall and the space frame model are shown in Tables 2.3 and 2.4, respectively. A comparison of values from the two tables shows that the two models are equivalent; and depending on the availability of computer codes, either of the two idealizations can be implemented.

2.5.3 Shear Wall Spring vs. Finite Element Model

It is well known that the primary lateral load-resisting elements in a nuclear power facility normally are shear walls. Because of the low aspect ratio of these buildings, the principal lateral displacements are due to the shear strains rather than to the flexural strains in the shear walls. The typical nuclear structure can therefore be modeled as a shear beam structure. Experience has indicated this to be a reasonable approximation.

A 150'x150'x150' structure with two floor slabs was analyzed using a shear beam element model and a finite element model. The first two modal frequencies obtained from the two models are listed in Table 2.5. The comparison shows a 19% difference for the first modal frequency and only 1% for the second mode. It is apparent that although the finite element model may be more accurate, it is impractical to model the true nuclear power plant structure in this manner because of the large number of finite elements and nodes which would be required.

2.5.4 3-D vs. 2-D Structural Model (5)

The auxiliary-turbine-fuel building complex of a nuclear power plant is a very irregular structure. The torsional response of the overall structure is naturally an important consideration. Using a regular frame model in its entirety to represent a structure with this degree of complexity would be difficult. Hence, the 3-D slab-shear wall system is constructed for this purpose. The degree of torsional coupling of the slab-shear wall system depends entirely on the amount of eccentricity between the center of mass and center of rigidity of the floor slabs.

A study was made to evaluate the effect of torsion on the overall structural response. Exhibits 2.14 and 2.15 show the power plant cross section and its corresponding mathematical model for analysis. The time-history analysis technique was employed. The ground motion used to determine the dynamic response of this structure has a maximum base acceleration of 0.05 g, and the corresponding time-dependent base motions are scaled down from the N-S component of the El Centro, California, earthquake of May 18, 1940. Exhibit 2.16 presents a comparison of the floor acceleration with and without consideration of the torsional effect on the structure. The three-dimensional analysis, including the torsional coupling, produces accelerations in the X-direction, although the base excitation is in the Y-direction. The dotted line shows the results obtained from a two-dimensional analysis in which the torsional movements of the slabs are neglected. Exhibit 2.17 shows a comparison of the distribution of maximum shear forces for a steel bracing system. A great discrepancy exists between the results for the case including torsional coupling and the case which neglects it. The examples in Exhibits 2.16 and 2.17 give some indication of the significance of the torsional effect on the building response. The most significant discrepancies are in the base shear force and the floor acceleration at the higher elevations of the structure. Errors on the order of 200% in the base shear and 33% in the floor acceleration are induced. Since the response of the building components and equipment resting on the building closely depends on the physical behavior of the building itself, the oversight or neglect of the torsion motion could yield serious errors in the response of all items studied. Hence, if

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torsional effects are significant, it may be advisable not only to consider 3-D effects but also to consider the simultaneous action of earthquake components and to develop spectra for more than one location on the slab.

2.5.5 Floor Slab Grid Model vs. Simplified Beam Model for Composite Floor System (4)

Power plant floor systems under vertical seismic excitation can be analyzed by a slab grid model using dynamic analysis. However, the analysis is expensive and results will be sensitive to changes in equipment location and slab opening. A simplified beam model using equivalent static load methods can be used to overcome some of these problems. An example is presented here to compare the grid model and simplified model results.

The slab grid model shown in Exhibit 2.4 consists of a total of 188 nodes and 260 beam elements. Thirty modes were considered in the dynamic response analysis. In the simplified beam model, each girder was analyzed separately by an equivalent static load method, by which the seismic loads on the girder are computed as the product of the weight of structural components and a multiplier. The values of the multiplier are determined based on the type of beam, the natural frequency of the structural components and the appropriate wall response spectra curves.

Table 2.6 shows a comparison of maximum seismic moments obtained from the detailed dynamic model and the simplified beam model. The comparison shows that the equivalent static load method gives satisfactory results, except for a few girders where it gives smaller moments than those obtained from dynamic analysis. These few underestimations may result because the contributions of higher modes are not considered. A conservative amplification factor of 1.5 is found to be the appropriate value to account for the participation of higher modes in the equivalent static load method. Use of this amplification factor for flexible beams having frequencies lower than 33 Hz will ensure the adequacy of the results obtained by the equivalent static load method.

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3. STRUCTURAL MODELS FOR SOIL-STRUCTURE INTERACTION ANALYSIS

3.1 Introduction

The design earthquake motion for nuclear power plants is commonly defined by "free field" response spectra at the elevation of the plant foundation. A knowledge of the impact of the supporting soil on the dynamic characteristics of the plant is essential. The modal characteristics of the structure change due to the flexibility of the soil. Consequently, the response of the system is altered due to the change in structural frequencies and the dissipation of energy through radiation and material damping of the soil. The use of free field spectra, therefore, does not reflect the effects of having a plant structure in the soil medium. The evaluation of soil-structure interaction effects thus has become an important aspect of nuclear power plant design. There are basically two approaches to solve the soil-structure interaction problem: the compliance function method and the finite element method.

A. Compliance Function Method

In the compliance function approach, the soil medium is characterized by lumped parameters or compliance functions which are derived from the assumption of a rigid plate resting on an elastic half space (6), or a rigid plate resting on a layered viscoelastic half space (7). In general, these parameters are complex-valued functions of frequency. The real part is related to the stiffness of the soil and the imaginary part to the damping of the soil. The structure is then coupled to the soil through the equations of equilibrium and the compatibility of the intersurface nodes.

B. Finite Element Method

In this method, the soil-structure system is represented by a finite element model. It is assumed that the ground surface motions result from

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vertically propagating shear waves. The basic steps of the solution process are as follows:

1. The soil in the soil-structure system is modeled by a finite element model whose boundaries are placed such that there is no significant soil-structure interaction effect beyond these boundaries. Alternately transmitting boundaries can be used with a spatially reduced soil model.
2. The motion at the base of the model is obtained by a deconvolution procedure. In this analysis the SHAKE program (8) uses strain-compatible soil properties and the solution of a one-dimensional shear wave equation.
3. The coupled soil-structure model is analyzed using appropriate soil properties and the base motion obtained in the previous step.

It is general practice to use a simplified soil-structure model to obtain the motion at the base of the structure. The structural analysis is then performed using a more detailed decoupled model of the structure with derived input motion. The detailed techniques for the soil-structure interaction analysis are provided in a special project in SSMRP and are not reviewed in this chapter. Instead, only the modeling techniques and structural parameters which affect the structural response in SSI analyses will be discussed here through the following example problems. It should be mentioned that the conclusions drawn from the results of these example problems are problem-oriented and may not be the same for other site conditions.

3.2 Effect of Large Aspect Ratio of Soil Model on Structural Response (9)

In finite element modeling, the element dimensions are important. However, the relative importance of the horizontal or vertical dimension depends on the nature of the problem. If the problem involves only vertically traveling

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shear waves or predominant shear deformations, then only the vertical dimension of the element is of critical importance for proper frequency transmission. If the problem involves transmission of flexure waves, then both horizontal and vertical dimensions are important.

In the seismic soil-structure analysis of large nuclear power plants using a finite element model, it is well recognized that the vertical dimension of soil elements is critically important for proper frequency transmission. Usually, this dimension is between $\frac{V_s}{6 f_{\max}}$ to $\frac{V_s}{4 f_{\max}}$, where V_s is the shear wave velocity in the soil layer and $f_{\max}$ is the highest frequency of interest in the assumed earthquake motion. The horizontal dimension has not been found to be as important, and large horizontal dimensions have been used frequently. This results in a large aspect ratio for the elements. The use of such large aspect ratios is common because of the large horizontal extent of the model. In modeling the soil elements, it is usually necessary to put the vertical boundaries of the soil model quite far from the structure in order to include the interaction effect adequately and without reflection from the boundary. Thus, the horizontal extent of the model becomes very large. This situation, coupled with the requirement of small vertical dimensions for elements, makes it necessary to use large aspect ratios to achieve computational economy.

Exhibit 3.1 shows the lumped mass analytical model of the nuclear power plant structure analyzed. Mass and stiffness properties of the analytical model are listed in Table 3.1 and the modal periods and participation factors for the first seven modes of the fixed base model are listed in Table 3.2.

A 5% critical damping is assumed for the entire structure. The size of the base slab of the structure is 300 ft. x 238 ft. The site consists of 135 ft. of soil overlying the bedrock. The top 15 feet consist of sand; the rest is till. The structure foundation is embedded 45 ft. into the ground. An RG 1.60 spectrum-consistent earthquake time history corresponding to 0.15 g maximum

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ground acceleration is used as the free field design earthquake defined at the foundation level.

Three different finite element models of soil, as shown in Exhibits 3.2 through 3.4, were made using different aspect ratios. The aspect ratios vary from 1.25 to 10 and are shown in Table 3.3. Strain-dependent soil properties were used. Using these three models, the floor response spectra at the base slab and slab 7 were obtained; they are shown in Exhibits 3.5 and 3.6. The comparison is close, with the maximum difference on the order of 20-25%.

It can be concluded from this study that in solving the type of soil-structure problem presented here, the use of large aspect ratios for soil elements is justifiable. This should be expected because of the relatively simple deformation pattern that occurs in the soil profile.

3.3 2-D vs. 3-D Finite Element Soil Models (10)

The use of a 2-D finite element soil model in the soil-structure interaction analysis of nuclear power plants has become acceptable. The 2-D finite element soil model is generally used in the analysis because appropriate 3-D finite element programs are not available. The 2-D finite element soil model leaves open the question of soil participation from the third direction. To investigate this question, an example problem based on 2-D and 3-D soil models has been analyzed and the results compared. The structural model, site soil condition, and design earthquake were the same as those used in Section 3.2 in the study of the effect of aspect ratio of soil model on structural response.

Two-dimensional Soil Model: The model used for this case is shown in Exhibit 3.2. The distance from the foundation cut to the left and right boundaries in the FEM was long enough so that the free field conditions could be satisfied at these boundaries. Vertical movement of the nodes along the side boundaries was restrained. Strain-dependent soil properties were used to determine the stiffness properties of the soil elements.

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Three-dimensional Soil Model: The finite element model in this case consisted of 3-D axisymmetric solid elements. An equivalent circular foundation whose area equaled that of the rectangular foundation was used in the analysis. The diameter of the equivalent circular slab was calculated to be 300 ft. Other parameters of this model were the same as those of the 2-D model.

Response spectra for 5% damping were generated from the two models at different elevations of the structure. Exhibit 3.7 shows the comparison of the foundation response spectra obtained from the 2-D plane strain model and the 3-D axisymmetric model. There is an obvious, appreciable difference between the responses obtained from the two models. For the range of periods in nuclear power plants, the 2-D model gave a higher response than the 3-D model.

Exhibit 3.8 shows the comparison of response spectra obtained from the 2-D and 3-D models at joint 7, which is at a higher elevation. As in the comparison of foundation response spectra, the responses obtained from the 2-D model are greater than those obtained from the 3-D model in the shorter period range. Based on these results, it is concluded that the 2-D analysis yields conservative results in studies of interest for nuclear power plants.

3.4 Plane Strain vs. 3-D Structural Model

Some computer programs, such as FLUSH (11), are incapable of modeling 3-D structures. A structure is sometimes characterized by an equivalent 2-D plane strain model which is coupled with soil finite elements for interaction analysis. The interaction time history at the base of the structure thus obtained is then used for the detailed structural analysis. The difference in the results of these two modeling techniques can be observed through the following example problem.

The structural model is shown in Exhibit 3.9. The equivalent plane strain finite element model was obtained by matching the shear stiffness and mass

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distribution of the detailed 3-D and equivalent 2-D models. The masses and stiffnesses of the 2-D plane strain structural model were divided by the structure width in the third direction to keep the soil-structure mass and stiffness ratios in correct proportion.

Tables 3.4 and 3.5 show the modal periods and modal participation factors obtained from the 3-D detailed model and the 2-D plane strain model. Exhibits 3.10 and 3.11 show the comparison of the normalized mode shapes for the two models. The closeness of the results verifies the equivalence of the 2-D and the 3-D models. These two models can be further verified by comparing the responses using the NRC Regulatory Guide 1.60-consistent time history applied at the fixed base structure models. Table 3.6 shows the comparison of maximum slab displacements. Again, the responses are very close for the two models being considered.

It should be noted that equivalence of 2-D and 3-D models is problem dependent and its accuracy should be studied in each case.

3.5 SSI Results Using 2-D vs. 3-D Structural Models

The 2-D plane strain structural model described in Section 3.4 was used for SSI analysis using the FLUSH program. A modified version of FLUSH, developed by Sargent & Lundy, was used to analyze an SSI problem using the 3-D structural model. A 2-D finite element soil model with viscous boundaries to simulate 3-D effects was used with the above two models.

Exhibit 3.12 shows the interaction spectra generated at the base of the structure for the 2-D and the 3-D models. The 2-D results are generally slightly higher than the 3-D results. This is believed to be the result of the 3-D effects. It can be concluded that the 2-D structural model can be used in SSI analyses, provided the 2-D model closely simulates the characteristics of the 3-D model.

3.6 Influence of Cut-Off Frequencies and Interpolation Intervals

In the complex response solution of a soil-structure interaction problem, various parameters are involved in the analysis. These parameters are mostly

problem-oriented, the two most important being the cut-off frequency and the interpolation interval. The cut-off frequency refers to the maximum frequency to which responses are computed and generally represents the frequency beyond which there is no further amplification in the structural response. To save computation costs in the complex frequency response solution, response is generally computed every second, fourth or eighth definite frequency of the Fourier transform. Based on responses at these frequencies, the responses of the intermediate frequencies are interpolated. The interpolation interval refers to the frequency intervals at which the responses are computed. Proper selection of these parameters reduces computation time, yet maintains solution accuracy. The following example problem shows the effects of these parameters on the structural response.

The structural model used in the analysis is shown in Exhibit 3.9. Two cut-off frequencies, 15 cps and 33 cps, were used. Exhibits 3.13 and 3.14 show the response spectra generated at various elevations in the building for a 15 Hz and a 33 Hz cut-off frequency. Even in the high frequency region, the responses are close for the two cut-off frequencies used in the analysis. It is concluded that for this site, a 15 Hz cut-off frequency is adequate in the complex response solution of the soil-structure interaction problem.

In the interpolation interval study, three models corresponding to different interpolation interval control parameters were used. The basic frequency interval for the three models was from 0 to 15 Hz and the frequency step $\Delta f = 0.09766$ Hz. The three models differ in the manner in which frequencies are selected for the calculations. In Model 1 points at every step of Δf in the interval were selected and used. The 0 to 15 Hz interval in Model 2 was subdivided into 0 to 2, 2 to 8 and 8 to 15 Hz. In the first subinterval, points at every other step were used in the calculations; in the second subinterval, points at every fourth step; and in the third subinterval, points at every eighth step were used. Model 3 was divided into the same subintervals as Model 2, with the same three subinterval selections of frequencies. The three models are summarized in Table 3.7.

Exhibit 3.15 shows the response spectra obtained for the above variation of the frequency interpolation parameter. It can be concluded that responses based on schemes 1 and 2 are comparable. Thus scheme 2 can be used without loss of accuracy for this particular site. Scheme 3 results in a loss of accuracy in the 5 to 10 Hz range and cannot be used.

3.7 Effect of Embedment

The side soil of an embedded structure may affect structural response in the soil-structure interaction analysis. Vertical beams are sometimes provided at the interface in the soil model to simulate the embedment effects. The effect of side soil can be seen through the following example problem.

The structural model, soil properties, and the design earthquake are the same as those used in Section 3.6 and are shown in Exhibit 3.9. Exhibits 3.16 and 3.17 show the response spectra for 1% damping generated at various elevations in the structure with and without embedment effects. There are some differences at the peaks for the two cases considered. Therefore, the effect of side soil should be taken into account in soil-structure interaction studies.

4. METHODS OF DYNAMIC SEISMIC ANALYSIS

4.1 Introduction

In the seismic analysis of structures, the first step is to model the structure with a system of masses, springs, and dashpots suitable for dynamic analysis. The essence of mathematical modeling is to select these parameters so that the responses obtained from the analysis provide a good representation of the motion of the structure under seismic excitation. The modeling techniques are described in detail in Chapter 2.

Once the mathematical model of a structure has been defined, the next step is to form the equations of motion. No matter what method of analysis is selected for a given problem, the equations of motion for a multi-degree-of-freedom linear system are:

$$M\ddot{X}(t) + C\dot{X}(t) + KX(t) = -M\phi_r \ddot{y}_g(t) \quad (4.1)$$

where

ϕ_r = rigid body motion of superstructure corresponding to unit translation and, if possible, rocking of the base

M = mass matrix of the system

K = stiffness matrix

C = damping matrix

$X(t)$ = displacement vector relative to rigid body motion of the base at time t

$\dot{X}(t)$ = velocity vector relative to rigid body motion of the base at time t

$\ddot{X}(t)$ = acceleration vector relative to rigid body motion of the base at time t

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and

$$\ddot{y}_g(t) = \text{time-dependent prescribed support acceleration}$$

The analytical approaches currently available for the seismic analysis of structures governed by Eq. 4.1 can be classified into two broad categories: time domain analysis and frequency domain analysis. Again, the time domain analysis can be performed using the modal and direct methods. Selecting a method which is suitable to a given problem depends on the structural properties, input format, the type of response which is of interest, economic considerations and the criticality of the structure. The most commonly used methods for seismic structure analysis in the nuclear power industry are summarized and discussed herein.

4.2 Time Domain Analysis

Methods generally used in a time domain analysis are described in the following sections.

4.2.1 Modal Method

Modal analysis is the most common analysis technique being used in the industry today. In this method, the modal characteristics of a structure are determined and the response in each mode for the given excitation is calculated. These modal responses are then combined appropriately to determine the overall structural response. For a system whose supports are assumed to experience the same excitation, the equation of motion may be written as

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = -[M]\{\ddot{u}\} \quad (4.2)$$

where $[M]$, $[C]$ and $[K]$ denote the mass, damping and the stiffness matrix of the system, respectively. The vectors $\{\ddot{x}\}$, $\{\dot{x}\}$ and $\{x\}$ are comprised of the system's acceleration, velocity and displacement relative to the base, and $\{\ddot{u}\}$ is the vector of the base acceleration. Assuming that the system has

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classical modes, the equation of motion can be reduced to a set of decoupled equations using the transformation

$$\{x\} = [\phi]\{g\} \quad (4.3)$$

Equation 4.2 then reduces to

$$\{\ddot{g}\} + [2\beta_j\omega_j]\{\dot{g}\} + [\omega_j^2]\{g\} = -\gamma_j\ddot{u} \quad (4.4)$$

where ϕ_j , ω_j , and β_j are the mode shape, the natural frequency, and the fraction of critical damping of the j^{th} mode, respectively. γ_j is the participation factor given by

$$\gamma_j = \frac{\phi_j^T [M] \{\phi_r\}}{\{\phi_j\}^T [M] \{\phi_j\}} \quad (4.5)$$

The response spectrum, time-history and random vibration methods are used in conjunction with the modal analysis.

A. Response Spectrum Method

In the power industry, the maximum seismic response of structures is often obtained by using the response spectrum method. This approach is used extensively because (1) the earthquake input motion is generally defined in terms of a response spectrum; (2) the response spectrum method involves fewer computations, even though the results obtained are generally comparable to those obtained by the time-history analysis; and (3) it yields a response based on a class of earthquake as opposed to a single time history, thus reducing the uncertainties associated with the results obtained.

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In the response spectrum method, the input motion is described in terms of a response spectrum, which is a plot of the maximum response of a single-degree-of-freedom oscillator with different natural frequencies to a given input motion and damping. Exhibit 4.1 shows the response spectrum for a damping of 2% of critical for the earthquake record shown in Exhibit 4.2. The plot relates to the natural frequencies. Empirical rules have also been established, Newmark et al. (12), whereby the response spectrum can be constructed based on maximum probable ground acceleration.

The maximum displacement for each mode X_i can be determined by multiplying the modal participation factor γ_i and modal vector ϕ_i for the given mode by the acceleration spectrum at frequency ω_i . Thus,

$$X_i = \gamma_i \phi_i \frac{S a_i}{\omega_i^2} \quad (4.6)$$

Based on the maximum modal displacement vector given by Eq. 4.6, member forces, joint reactions and other responses may be computed for each mode. As only the maximum is known for each mode, and since these maximum responses are not expected to occur simultaneously, empirical rules must be used to combine modal responses to obtain the final response. The following rules are generally used.

a. Absolute sum of maximum modal responses (ABS sum)

In this method, the final response is evaluated as the sum of the absolute values of the modal responses in each mode. Thus, if R_k denotes the maximum modal response in the k^{th} mode, the total response R_a is expressed as

$$R_a = \sum_{k=1}^N |R_k| \quad (4.7)$$

in which N = number of significant modes. As the maximum responses of the modes do not occur simultaneously, the above rule gives an upper bound, generally very conservative, for the response under consideration.

b. Square root of the sum of squares of modal responses (SRSS)

In this method the probable response is evaluated as

$$R_a = \left[\sum_{k=1}^N R_k^2 \right]^{1/2} \quad (4.8)$$

This is the most commonly used rule for combining maximum modal responses. It generally leads to results in close agreement with those obtained by the time-history analysis, provided the individual modal frequencies are well-separated. In a complex multi-degree-of-freedom system, the modal frequencies are not always well-separated. This is especially true for cases where a major contribution is from higher modes, which are generally close to each other. Because certain coupling terms are ignored in these cases, the results obtained by the above rule may be far from the true values.

c. Double sum method

It has been pointed out that in cases where the coupling terms cannot be ignored, the responses obtained by the previous rule may not be a good prediction of the actual response. In Reference 13, a rule for combining modal responses is proposed which accounts for these coupling terms. According to this rule, R_a is expressed as

$$R_a = \left[\sum_{k=1}^N \sum_{\ell=1}^N R_k R_\ell \epsilon_{k\ell} \right]^{1/2} \quad (4.9)$$

where

$$\epsilon_{kl} = \left[1 + \left\{ \frac{(\omega_k - \omega_l)^2}{(\beta_k^2 \omega_k + \beta_l^2 \omega_l)^2} \right\}^2 \right]^{-1} \quad (4.10)$$

in which

$$\omega_k' = \omega_k \left[1 - (\beta_k')^2 \right]^{1/2} \quad (4.11)$$

$$\beta_k' = \beta_k + \frac{2}{t_d \omega_k} \quad (4.12)$$

with ω_k and β_k = the modal frequency and damping in the k^{th} mode, respectively, and t_d = the duration of the earthquake. It is noted that when model coupling is ignored, i.e., $\epsilon_{kl}=0$ for $k \neq l$ and $\epsilon_{kk}=1$, Eq. 4.9 reduces to Eq. 4.8.

Gungor (14) has shown that the expression for responses obtained by using the random vibration method may be closely approximated by Eq. 4.9; this conclusion is further substantiated by several numerical examples presented in Reference 14. Based on the random vibration approach used by Gungor (14), ϵ_{kl} can also be expressed as

$$\epsilon_{kl} = \frac{\sigma_{kl}^2}{\sigma_{kk} \sigma_{ll}} \quad (4.13)$$

where

$$\sigma_{kl}^2 = \int_{-\infty}^{\infty} G(\omega) H_k(\omega) H_l^*(\omega) d\omega \quad (4.14)$$

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in which

$G(\omega)$ = spectral density function of ground acceleration

$$H_k(\omega) = \frac{1}{\left\{ \omega_k^2 - \omega^2 + 2i\beta_k \omega \omega_k \right\}} \quad (4.15)$$

$H_k^*(\omega)$ = complex conjugate of $H_k(\omega)$

The integral of Eq. 4.14 can be evaluated by the method of residues. Several expressions for $G(\omega)$ are available (14, 15, 54). However, as ξ_{kl} is expressed as a ratio and earthquakes are wide band processes, $G(\omega)$ may be assumed to be a constant for an evaluation of ξ_{kl} .

- d. Other methods of combining modal maxima for closely spaced modes
The following expressions are given in NRC Regulatory Guide 1.92 for this purpose.

Grouping Method: In this case maximum response is calculated from

$$R_a = \left[\sum_{k=1}^N R_k^2 + \sum_{q=1}^P \sum_{i=j}^j \sum_{m=i}^d \left| R_{iq} R_{mq} \right| \right]^{1/2}, \quad 2 \leq m \quad (4.16)$$

where P = number of groups of closely spaced modes considered in analysis; R_{iq} and R_{mq} = the values of modal responses R_i and R_m within group q of closely spaced modes; i = mode number at which a group starts; and j = mode number at which the group ends. To specify each group of closely spaced modes, modes whose frequencies differ by less than 10 percent are called closely spaced; starting from the lowest frequency and working toward successively higher frequencies, the closely spaced modes are identified and grouped.

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Ten Percent Method: In this method

$$R_a = \left[\sum_{k=1}^N R_k^2 + 2 \sum_{i \neq j} |R_i R_j| \right]^{1/2} \quad (4.17)$$

In this equation the second summation covers all mode numbers of closely spaced modes i, j , satisfying the conditions

$$\frac{\omega_j - \omega_i}{\omega_i} \leq 0.1 \quad (4.18)$$

$$1 \leq i < j \leq N \quad (4.19)$$

Absolute Double Sum Method: In this case the maximum response is calculated from the relation

$$R_a = \left[\sum_{k=1}^N \sum_{l=1}^N |R_k R_l| \epsilon_{kl} \right]^{1/2} \quad (4.20)$$

in which ϵ_{kl} is given by Eq. 4.10.

It is worth noting that since conditions 4.18 and 4.19 are imposed on the second summation in Eq. 4.17, this equation and Eq. 4.16 are identical. Also note that Eq. 4.20 is the same as Eq. 4.9 except that absolute values of the summand have been considered. As discussed in Subsection 4.5.1, while Eq. 4.9 gives results close to those obtained from random vibration and it can be justified on this basis, the addition of absolute signs to this equation to obtain Eq. 4.20 has no theoretical justification.

B. Time-History Method

In the time-history method, the input motion is expressed in terms of time function. The analysis is performed by a time integration of the

equation of motion. The uncoupled modal equation, Eq. 4.4, can be solved in a closed form solution by applying a convolution integral scheme. The Duhamel's integral for the generalized coordinate $g_i(t)$ with zero initial conditions can be expressed as (54)

$$g_i(t) = - \int_0^t \frac{f(\tau) e^{-\xi_i \omega_i(t-\tau)} \sin \omega_i(t-\tau) d\tau}{\omega_i} \quad (4.21)$$

where

$$\omega_i' = \omega_i (1 - \xi_i^2)^{1/2}$$

Another way to solve Eq. 4.4 is based on a step-by-step numerical integration scheme. In this approach, the accelerations, velocities and displacements at any time are obtained from values of these quantities at the previous time. To assure the accuracy and stability of the solution in a step-by-step integration of the equation of motion, a time step should be properly selected according to the lowest structural period of the system. Many methods are available for a step-by-step integration to solve structural dynamic problems. The commonly used methods are the Newmark- β method (29), the Wilson- θ method (28), the Houbolt method (27), the central difference method (16), and the Runge-Kutta method (34). These methods are described in detail in Section 5. Once the time histories of each modal response are computed, the time history of the relative displacement of the system can be obtained from

$$X(t) = \sum_{i=1}^N \phi_i g_i(t) \quad (4.22)$$

These displacement time histories are used to compute the time history of forces, etc. The maximum response is computed as the maximum over the earthquake duration.

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The method described above is normally referred to as the time history-modal displacement approach since the final solution is expressed directly in terms of the modal displacements $g_i(t)$. This method is particularly useful for the seismic dynamic analysis of structures in which the significant contribution to the total response is from lower modes. In problems like the dynamic analysis for fluid transient forces caused by sudden valve release and closers, it is necessary to consider many higher modes. The time history-modal acceleration method alleviates this problem to a great extent (18). It has been shown in Ref. 18 that to achieve a faster convergence, the mode acceleration method is superior to the mode displacement method, and for a given degree of convergence, the mode acceleration method also involves fewer computations. In this approach, the dynamic response is computed in terms of the static solution plus a correction to the static solution, thus, significantly accelerating the convergence of the solution. The response can be obtained by a modification of Eqs. 4.21 and 4.22.

$$X(a,t) = \alpha(a) \ddot{y}_g(t) - \sum_{i=1}^N \phi_i(a) \gamma_i \left(\frac{\ddot{g}_i}{\omega_i^2} + 2\zeta_i \frac{\dot{g}_i}{\omega_i} \right) \quad (4.23)$$

where

$\alpha(a)$ = the influence function value at a for a static loading corresponding to the load vector

For earthquake excitation, where there is generally no amplification found beyond, say 33 cps, Eq. 4.23 in truncated form also represents the total solution. Thus, if M^{th} mode corresponds to the cut-off frequency, then Eq. 4.23 can also be expressed as

$$X(a,t) = \sum_{i=1}^M \phi_i(a) \gamma_i \ddot{g}_i(t) + \left\{ \alpha(a) - \sum_{i=1}^M \phi_i(a) \frac{\gamma_i}{\omega_i^2} \right\} \ddot{y}_g(t) \quad (4.24)$$

C. Random Vibration Method

Seismic ground acceleration is usually prescribed by a deterministic time function or by a set of smoothed response spectra curves. As earthquake motions are generated through numerous random reflections, refractions and attenuations of seismic waves, random characterization of earthquake motions has recently been considered desirable (19). In the random vibration approach, the solution vector for a multi-degree-of-freedom system excited by earthquake motion characterized by a base acceleration process $\ddot{Y}_g(t)$ is given as

$$X(t) = - \sum_{k=1}^N \gamma_k \phi_k \int_0^t \ddot{Y}_g(\tau) h_k(t-\tau) d\tau \quad (4.25)$$

where $h_k(t-\tau)$ is the impulse response function at mode k of the system and is given by

$$h_k(t-\tau) = \frac{1}{\omega_k'} e^{-\xi_k \omega_k' (t-\tau)} \sin \omega_k' (t-\tau); \quad t \geq \tau \quad (4.26)$$

and $\omega_k' = \omega_k \sqrt{1-\xi_k^2}$, the damped natural frequency

It follows that the autocorrelation function for the response $R_{xx}(t_1, t_2)$ is

$$R_{xx}(t_1, t_2) = \sum_{k=1}^N \sum_{j=1}^N \gamma_k \gamma_j \phi_k \phi_j \int_0^{t_1} \int_0^{t_2} R_{\ddot{Y}_g \ddot{Y}_g}(t_1, t_2) \times h_k(t_1-\tau_1) h_j(t_2-\tau_2) d\tau_1 d\tau_2 \quad (4.27)$$

For simplicity, stationary models have always been adopted for seismic characterization in linear analysis and have been found to give

conservative results (20). On this basis, the stationary response auto-correlation function is as follows

$$R_{xx}(\tau) = \sum_{k=1}^N \sum_{j=1}^N \gamma_{kj} \phi_{kj} \int_{-\infty}^{\infty} H_k(\omega) H_j^*(\omega) \phi_{\ddot{y}_g} \ddot{y}_g(\omega) e^{i\omega\tau} d\omega \quad (4.28)$$

where

$$H_k(\omega) = 1/\omega_k^2 - \omega^2 + 2i\xi_{\omega_k} \omega \quad (4.29)$$

= the complex frequency response function for mode k

$H_k^*(\omega)$ = the complex conjugate of $H_k(\omega)$

in which $i = \sqrt{-1}$

The variance of the stationary response is then obtained as

$$\begin{aligned} \sigma_{xx}^2 &= R_{xx}(0) \\ &= \sum_{k=1}^N \sum_{j=1}^N \gamma_{kj} \phi_{kj} \int_{-\infty}^{\infty} H_k(\omega) H_j^*(\omega) \phi_{\ddot{y}_g} \ddot{y}_g(\omega) d\omega \end{aligned} \quad (4.30)$$

In the above equations $\phi_{\ddot{y}_g} \ddot{y}_g(\omega)$ is called power spectral density function, PSDF, of the random process $\ddot{y}_g(t)$ and is related to the autocorrelation function $R_{xx}(t)$ by the following formula

$$\phi_{\ddot{y}_g} \ddot{y}_g(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} R_{\ddot{y}_g} \ddot{y}_g(\tau) e^{-i\omega\tau} d\tau \quad (4.31)$$

$$R_{\ddot{y}_g} \ddot{y}_g(t) = \int_{-\infty}^{\infty} \phi_{\ddot{y}_g} \ddot{y}_g(\omega) e^{i\omega t} d\omega \quad (4.32)$$

Eq. 4.30 is commonly used for stochastic seismic analyses; therefore, the random vibration method is sometimes referred to as the power spectral density method. To solve Eq. 4.30, a direct spectral analysis to obtain

the PSDF may be desirable, but because of the inadequate data available the results may not be statistically significant. Some relationships between a prescribed response spectrum and a PSDF are available in the literature (21). For wide-band response spectra used in the seismic design of nuclear power plants, the following form of the PSDF was found to be suitable

$$\ddot{Y}_g \ddot{Y}_g(\omega) = S_0 \sum_{j=1}^3 S_j (\omega_j^4 + 4\omega_j^2 \beta_j^2 \omega^2) / \left[(\omega_j^2 - \omega^2)^2 + 4\omega_j^2 \beta_j^2 \omega^2 \right] \quad (4.33)$$

where S_0 , S_j , β_j and ω_j are the PSDF parameters related to the site condition.

With the PSDF given by Eq. 4.33, the solution for the integral in Eq. 4.30 may be evaluated by the method of residues. Basically, there are three advantages in using the random vibration approach. First, rather than considering only one time history in the analysis, the structures can be seismically analyzed with the aid of a statistical analysis of the earthquake acceleration, expressed as a stochastic process to represent an assemble of possible ground motion histories. Second, instead of performing expensive time-history analyses, the floor response spectra for equipment and piping design can be generated by the power spectra density method (25). Third, this method can handle seismic analysis problems for which the input loads are provided by a PSDF.

4.2.2 Direct Integration Method

In the preceding discussion of the response of multi-degree-of-freedom systems, it has been assumed that the structures are linear and possess normal modes of vibration. The overall dynamic response can be obtained by combining responses from significant modes of vibration. However, often the structural properties cannot be assumed to be constant during the earthquake excitation, or the structure does not possess a classical mode of vibration, or

the computation time to extract the required number of modes is excessive. In these situations it is necessary to evaluate the response by a step-by-step integration of the coupled equations of motion. This is normally referred to as the direct integration method in dynamic analysis, which is the most appropriate method for the rigorous solution of nonlinear problems. It is also particularly advantageous for large linear structural systems when many modes are excited and the time history duration is very short.

Excitation specified in conjunction with the direct integration analysis is a time history. Depending on the form of the damping matrix of a structural system, different integration schemes can be used in the time-history method. These are classified as time-history analyses with proportional damping and time-history analyses with variable damping.

A. Time-History Method with α, β Damping

When Eq. 4.1 is solved by direct integration, to simplify computations and to minimize computer storage, the damping matrix is assumed to be a linear combination of the mass matrix and stiffness matrix,

$$C = \alpha M + \beta K \quad (4.34)$$

Assuming that the acceleration of each point in the system varies linearly within a small time interval Δt leads to the following linear equations in terms of effective displacement $\bar{X}$.

$$\bar{K} \bar{X}_t = \bar{P}_t \quad (4.35)$$

where

$$\bar{K} = K + C_2 M \quad (4.36)$$

$$\bar{P}_t = -M \ddot{\phi}_P(t) + M [A_t + C_3 B_t] \quad (4.37)$$

$$\bar{X}_t = \frac{1}{C_1} X_t - \beta B_t \quad (4.38)$$

$$C_0 = \frac{3\alpha}{\Delta t} + \frac{6}{\Delta t^2} \quad (4.39)$$

$$C_1 = \frac{1}{1 + \frac{3}{\Delta t} \beta} \quad (4.40)$$

$$C_2 = C_0 C_1 \quad (4.41)$$

$$C_3 = \alpha - C_2 \beta \quad (4.42)$$

$$A_t = \frac{6}{\Delta t^2} X_{t-\Delta t} + \frac{6}{\Delta t} \dot{X}_{t-\Delta t} + 2\ddot{X}_{t-\Delta t} \quad (4.43)$$

$$B_t = \frac{3}{\Delta t} X_{t-\Delta t} + 2\dot{X}_{t-\Delta t} + \frac{\Delta t}{2} \ddot{X}_{t-\Delta t} \quad (4.44)$$

Linear Eq. 4.35 can be efficiently solved using the Frontal equation solver (22). A step-by-step integration, with the displacement, velocity and acceleration at the last step and the load at the present time, is used to form the effective load for the next step.

The modal damping ξ_i can be expressed in terms of α , β and modal frequencies as

$$\xi_i = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2} \quad (4.45)$$

From this equation, the multipliers α and β can be determined by specifying a desired damping at any two frequencies. These are selected to cover the range of frequency of interest. For a discussion of other algorithms of numerical integration of equations of motion, refer to Section 5.

B. Time-History Method with Variable Element Damping

In Section A. above the time-history method with only one set of α , β damping values assigned to all elements of the structural system was described. In many practical problems, it is desirable to have different

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dampings for different elements of the structure. In this case, the damping matrix of an individual element may be assumed to be proportional to its mass and stiffness matrices or explicitly expressed in any arbitrary form. A direct evaluation of damping matrix for any element is possible and can be used directly in the equations of motion.

The damping matrix C in Eq. 4.1 is obtained by assembling the element C^i matrix. This element C^i matrix can either be computed independently or evaluated from the stiffness matrix K^i and mass matrix M^i of the same element by using the following formula:

$$C^i = \alpha^i M^i + \beta^i K^i \quad (4.46)$$

α^i and β^i are the damping coefficients of the i^{th} element. When the equation of motion is solved by direct integration, it is assumed that the acceleration of each point in the system varies linearly within a small time interval Δt , which leads to the following linear equation in terms of displacement X .

$$\frac{6}{\Delta t^2} M + \frac{3}{\Delta t} C + K X_t = -M \ddot{y}_g(t) + M A_t + C B_t \quad (4.47)$$

in which

$$A_t = \frac{6}{\Delta t^2} X_{t-\Delta t} + \frac{6}{\Delta t} \dot{X}_{t-\Delta t} + 2 \ddot{X}_{t-\Delta t} \quad (4.48)$$

$$B_t = \frac{3}{\Delta t} X_{t-\Delta t} + 2 \dot{X}_{t-\Delta t} + \frac{\Delta t}{2} \ddot{X}_{t-\Delta t} \quad (4.49)$$

Linear Eq. 4.47 is solved using the Frontal equation.

4.3 Frequency Domain Analysis

The equation of motion of a multi-degree-of-freedom structure subjected to earthquake loading is rewritten below for convenience

$$[M] \{\ddot{x}\} + [C] \{\dot{x}\} + [K] \{x\} = -[M] \{\phi_r\} f(t) \quad (4.50)$$

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where all quantities are described in Section 4.1. The response can be obtained by the transfer function approach. One way of obtaining the transfer function is to solve Eq. 4.50 with $f(t) = e^{i\omega_j t}$ for a range of frequencies ω_j , which covers the frequency range of interest. The transfer function can be obtained by solving the following algebraic equation for each ω_j :

$$\left[-\omega_j^2 M + i\omega_j C + K \right] T(\omega_j) = -[M] \{ \phi_r \} \quad (4.51)$$

For systems where classical modes exist, the displacements x can be expressed as

$$x = \sum_j \phi_j \xi_j \quad (4.52)$$

where ϕ is the mode shape and ξ is the generalized coordinate; the transfer function can be expressed as

$$T(\omega_j) = - \sum_{j=1}^N \phi_j^T [M] \{ \phi_r \} \cdot \phi_j \cdot a(\omega_j) \quad (4.53)$$

where

$$a(\omega_j) = \frac{1}{\omega_j^2 - \omega_j^2 + 2i\beta_j \omega_j}$$

with ω_j , ϕ_j , and β_j being the modal frequency, modal shape, and modal damping of mode j , respectively.

For large models and models whose high frequency response is important, the above two approaches are time-consuming and expensive, as a wide range of frequencies must be considered. In such cases, a more economical approach is to obtain the time domain solution to Eq. 4.51 using direct integration techniques. The transfer functions are obtained as the ratio between the

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Fourier transform of the response and the Fourier transform of the forcing function for each of the frequencies of interest ω_j from the relation

$$T_k(\omega_j) = \frac{R_k(\omega_j)}{F(\omega_j)} \quad (4.54)$$

The Fourier transforms can be obtained by the fast Fourier transform techniques suggested by Cooley and Tukey (23). Even though Eq. 4.54 is valid for any arbitrary forcing function, the accurate solution is only achieved if a force of constant amplitude in frequency domain, herein called a band-limited white noise, is used. The time history of a typical band-limited white noise is shown in Exhibit 4.3; its Fourier transform is shown in Exhibit 4.4. The band-limited white noise can be expressed as

$$f_{\omega}(t) = \frac{1}{2\pi} \int_{-\omega_0}^{\omega_0} e^{i\omega t} d\omega \quad (4.55)$$

To correctly account for the zero starting condition in the generation to calculate the transfer functions, the inclusion of a quiet zone (trailing zeros) in the forcing function is provided. The quiet zone is long enough to damp out the transient response.

The transfer function expresses the relationship in the frequency domain between a loading and the corresponding structural response. In the frequency domain for elastic structures, this is a linear relationship and can be expressed by

$$R_k(\omega) = T_k(\omega) F(\omega) \quad (4.56)$$

where

$T_k(\omega)$ = transfer function for the k^{th} component

ω = frequency in radians per second

$R_k(\omega)$ = Fourier transform of the k^{th} response

$$= \int_{-\infty}^{\infty} r_k(t) e^{-i\omega t} dt$$

$F(\omega)$ = Fourier transform of the forcing function

$$= \int_{-\infty}^{\infty} f(t) e^{-i\omega t} dt$$

The structural response time history $r_k(t)$ is the inverse Fourier transform of $R_k(\omega)$. Thus,

$$r_k(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} R_k(\omega) e^{i\omega t} d\omega \quad (4.57)$$

4.4 Combination of Effects Due to Three Spatial Components of an Earthquake

Depending on which basic method is used in the seismic analysis, i.e., response spectrum or time-history method, the following two approaches are considered acceptable for the combination of three-dimensional earthquake effects.

4.4.1 Response Spectrum Method

For the response spectrum method of seismic analysis, calculate the square root of the sum of the squares of the maximum representative values of the codirectional structural responses caused by each of the three components of earthquake motion at a particular point of the structure, or of the mathematical model.

4.4.2 Time-History Analysis Method

When the time-history analysis method is used for seismic analysis, two types of analysis are generally performed, depending on the complexity of the problem:

- A. When the maximum responses due to each of the three components of the earthquake motion are calculated separately, the method for

combining the three-dimensional effects is identical to that described in Subsection 4.5.1, except that the maximum responses are calculated using the time-history method instead of the spectrum method.

- B. When the time-history responses from each of the three components of the earthquake motion are calculated by the step-by-step method and combined algebraically at each time step, the maximum response is obtained from the combined time solution.

4.5 Comparative Studies of Methods of Analysis

4.5.1 A Comparison of Random Vibration, Response Spectrum, and Time-History Methods

An example problem is presented to check the relative accuracy of the various rules for combining the modal responses for the time-history method, the response spectrum method and the random vibration method, by comparing the responses obtained using the three methods.

In the time-history method of analysis, the two horizontal components of four earthquake records listed in Table 4.1 are considered. These eight accelerograms were normalized such that the area under their undamped pseudo-velocity curve from $T = 0.1$ to $T = 2.5$ seconds became equal. Essentially, this normalization corresponds to making the records have nearly the same Fourier amplitude spectra, thus providing a better comparison for which the power spectral density is used in the random vibration method. To make the results of the time-history analysis as comparable to random vibration as possible, the power spectral density was determined such that it yields simulated records with the average response spectrum being very close to that obtained from the time-history analysis of the actual recorded motions. Basically random vibration results should be compared to the average of an ensemble of records, not to results obtained from a specific record. The steps outlined above were taken to make such a comparison possible.

From the analysis point of view, there is no difference between a system with torsion and one without. Structures with torsional modes are

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specifically selected because these systems have active modes with close frequencies and because the structural models for nuclear plants have such torsional modes. The system analyzed is shown in Exhibit 4.5. Translational motion is only in the direction of the ground acceleration. By setting the nondimensional parameters $\frac{mK}{Jk}$ and $\frac{me^2}{J}$ to appropriate values, active modes with close frequencies can be produced. Here, e = eccentricity; m and J are the mass and mass polar moment of inertia of each floor; K and k are the torsional and translational stiffnesses, respectively. Small values of $\frac{me^2}{J}$ and the condition $\frac{mK}{Jk} = 1$ correspond to a system with small eccentricity having equal frequencies for independent translational and torsional vibrations. However, when $\frac{mK}{Jk}$ differs from unity even for small $\frac{me^2}{J}$, modal frequencies are well-separated. This is seen from the calculated frequencies for two systems which are presented in the inset table of Exhibit 4.5. Here $\frac{me^2}{J} = 0.003$ for both systems; however system A has $\frac{mK}{Jk} = 1$ and closely spaced frequencies, while System B has $\frac{mK}{Jk} = 2$ which has well-separated frequencies.

A comparison of predictions from random vibration and time-history analyses for Systems A and B is given in Tables 4.2 and 4.3, respectively. When using random vibration results, a specific probability of exceedance should be assumed. To make fair comparisons we compared values at an exceedance probability of 0.5 with the average calculated from the eight earthquake records for the responses and systems considered. Values at an exceedance probability of 0.1 are compared with the second highest response among the eight calculated from the earthquake records. In Tables 4.2 and 4.3, two columns are presented for each probability level: columns 1 and 3 for when the effect of closely spaced modes is not considered when calculating random vibration results, columns 2 and 4 for when the effect of closely spaced modal frequencies is considered; i.e., Eq. 4.9 in conjunction with Eq. 4.13 are used to obtain the results. Table 4.2 shows that columns 1, 2, 3 and 4 do not agree for System A, which has closely spaced modes.

The similarly numbered columns in Table 4.3 indicate that the effect of closely spaced modes is absent in System B. The prediction capability of

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random vibration results with the time-history analysis is seen in the fact that values in Columns 2 and 4 of Tables 4.2 and 4.3 are close to unity. In fact, if more than eight earthquake records had been considered in the analysis, the ratio would have been even closer to unity.

Results of response spectrum methods are compared with accurate random vibration results for Systems A and B in Tables 4.4 and 4.5, respectively. In these tables $(DSM)_{RV}$ denotes accurate random vibration results, $(DSM)_{RS}$ corresponds to results from Eqs. 4.9 and 4.10, and $(SRSS)_{RS}$ denotes results from Eq. 4.8. For a system with closely spaced frequencies (Table 4.4), it is seen that $(DSM)_{RS}$ is close to $(DSM)_{RV}$, indicating that using Eq. 4.9 with 4.10 has acceptable results. It is not acceptable to use Eq. 4.8 for this case because the ratio of $(SRSS)_{RS}$ to $(DSM)_{RV}$ is not close to unity in Table 4.4. Since the effect of closely spaced frequencies is absent for System B in Table 4.5, both sets of ratios are close to unity.

Based on this comparison it is concluded that random vibration results are close to time-history analysis results, provided an ensemble rather than one record of time histories is considered in seismic analysis. Equations 4.9 and 4.10 provide acceptable predictions for random vibration results.

4.5.2 A Comparison of Time-History and Response Spectrum Methods for a Power Plant Structure

In the power industry, the maximum seismic response of structures is generally obtained by either the time-history or the response spectrum method. This example problem compares the responses obtained by these two methods.

In the analytical model used in the analysis (Exhibit 4.6), the auxiliary and radwaste buildings are idealized as three-dimensional shear structures. The floors and roofs have been modeled as diaphragms rigid in their own plane, and the inertial mass of the structure has been lumped for different slab elevations. The mass center of each diaphragm is allowed two horizontal translations and a rotation about the vertical axis. The lateral stiffness of

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reinforced concrete walls and steel trusses have been simulated by linear springs. The first thirty modes are considered in the modal solution. The modal frequencies and participation factors are listed in Table 4.6.

The above structural model has been analyzed for an artificial record generated to be consistent with the NRC Regulatory Guide 1.60 spectrum normalized to 0.2g maximum ground acceleration and the two horizontal components of four earthquakes listed in Table 4.1. In these studies, however, the records have been normalized to a maximum ground acceleration of 0.1g. The response spectrum calculated from the artificial earthquake record is compared to the NRC Regulatory Guide spectrum in Exhibit 4.7. Tables 4.7 through 4.11 show results of the analyses and comparisons when the structure is excited only in the x-direction, Tables 4.12 through 4.16 give results when the structure is excited only in the y-direction and Tables 4.17 through 4.21 show the results when the structure is excited simultaneously by both earthquake components. Each table contains the maximum spring forces in the same springs for specific excitations. Each table also contains the following ratios for responses obtained through response spectrum methods:

- a. ratio of SRSS to time history; the SRSS values are based on Eq. 4.8 and a response spectrum which is calculated from the record;
- b. ratio of predictions from Eqs. 4.9 and 4.10, called double sum, in the table;
- c. ratio of values from Eq. 4.20, called Abs. DSM in the table.

Based on the discussion in the previous section, ratios for b above should, on the whole, be closer to unity than cases a and c. This is seen to be true in the results presented in Tables 4.7 through 4.21. In fact, had we considered the average for a number of records rather than the ratios for one record, the results would have been even closer to unity.

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4.5.3 A Comparison of the Time-History and Random Vibration Methods in the Generation of Floor Response Spectra

The time-history method is usually used to generate floor response spectra for equipment design. Recent research in the stochastic theory has led to the use of the random vibration method to develop floor response spectra (25). This example problem compares the floor spectra generated by the above two methods. The structural model is shown in Exhibit 4.8. The structural periods and modal participation factors are listed in Table 4.22. The 15% widened floor spectra comparison is shown in Exhibits 4.9 and 4.10; the two methods give comparable results. In fact, part of the discrepancy observed results from using time history consistent with NRC Regulatory Guide 1.60; as seen in Exhibit 4.1, the time history spectra do not completely correspond to the smooth spectrum. This difference causes some inconsistency with the random vibration result, which is completely based on the smooth Regulatory Guide spectrum.

5. NUMERICAL INTEGRATION METHODS

5.1 Introduction

The equations of motion can be solved by using either the direct integration or mode superposition method. The particular method chosen depends on the type of problem under consideration. In either method of time-history analysis, the solution is obtained by numerically integrating the equation of motion.

In numerical integration, the equations of motion are integrated using a step-by-step procedure. In essence, numerical integration is based on two ideas. First, instead of trying to satisfy the equation of motion at any time t , the aim is to satisfy these equations only at discrete time instants Δt apart. Basically, this means that equilibrium, which also includes the effects of inertia and damping forces, is sought at discrete time points within the interval of solution. Second, a time variation for displacements, velocities, and accelerations within each time interval Δt is assumed. It is the form of the assumption on the variation of displacements, velocities, and accelerations within each time interval that determines the accuracy, stability, and cost of the solution procedure.

In the following, assume that the displacement, velocity and acceleration vectors at time o , denoted by U_o , $\dot{U}_o$, and $\ddot{U}_o$, respectively, are known, and let the solution be required from time o to time T . In the solution, the time span under consideration, T , is subdivided into n equal time intervals, Δt .

5.2 Types of Integration Schemes

The commonly used methods of integration are the Houbolt method, the Newmark - β method, the Wilson - θ method, the central difference method, the Runge-Kutta method, and Predictor-Corrector method. A detailed description of different types of numerical integration methods are given in Reference 26. In the following sections, some of these integration methods are described. Consideration of accuracy, selection of time step size, and a discussion of the relative advantages of the methods are discussed.

5.2.1 The Central Difference Method

In this method (16), the following relationships are assumed:

$$\begin{aligned}\ddot{U}_t &= \frac{1}{\Delta t^2} \left\{ U_{t-\Delta t} - 2U_t + U_{t+\Delta t} \right\} \\ \dot{U}_t &= \frac{1}{2\Delta t} \left\{ -U_{t-\Delta t} + U_{t+\Delta t} \right\}\end{aligned}\quad (5.1)$$

The error in the expansions (5.1) is of the order $(\Delta t)^2$. The displacement $U_{t+\Delta t}$ is obtained by using Eq. 5.1 and the equation of motion at time t , i.e.,

$$M \ddot{U}_t + C \dot{U}_t + K U_t = R_t \quad (5.2)$$

where M , C , and K are the mass, damping, and stiffness matrices; R is the external load vector. Thus we obtain

$$\begin{aligned}\left(\frac{1}{\Delta t^2} M + \frac{1}{2\Delta t} C\right) U_{t+\Delta t} &= R_t - \left(K - \frac{2}{\Delta t^2} M\right) U_t - \\ \left(\frac{1}{\Delta t^2} M - \frac{1}{2\Delta t} C\right) U_{t-\Delta t}\end{aligned}\quad (5.3)$$

from which $U_{t+\Delta t}$ can be solved. Since the solution of $U_{t+\Delta t}$ is based on the equation of motion at time t , the integration procedure is called the explicit integration method. The calculation of $U_{t+\Delta t}$ involves U_t and $U_{t-\Delta t}$. Therefore, to calculate the solution at time Δt , a special starting procedure must be used. Since U_0 , $\dot{U}_0$ and $\ddot{U}_0$ are known ($\ddot{U}_0$ can be calculated from U_0 and $\dot{U}_0$ using Eq. 5.2), Eq. 5.1 can be used to obtain $U_{-\Delta t}$. Thus we have

$$U_{-\Delta t}^{(i)} = U_0^{(i)} - \Delta t \dot{U}_0^{(i)} + \frac{\Delta t^2}{2} \ddot{U}_0^{(i)} \quad (5.4)$$

where superscript (i) indicates the i^{th} element of the vector considered.

A table which summarizes the integration scheme as it might be implemented in the computer is given in Reference 26.

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The type of problems for which the central difference method is suitable can be seen from Eq. 5.3. If velocity-dependent damping can be ignored in the analysis and if mass matrix is a diagonal matrix, it is not necessary to assemble the system stiffness matrix to carry out the calculations; response calculation can be executed at element level, thus requiring small storage and permitting rapid calculation of large systems. In this manner systems with more than 10,000 degrees of freedom have been handled successfully. This type of calculation would be reasonable for short duration, shock-type loadings for which velocity-dependent damping is not important. Nonlinearities in system performance can also be effectively considered under these conditions with this integration method. In the seismic response of power plants, velocity-dependent damping plays an important role; and for the time-history analysis of a superstructure, damping effects cannot be ignored. Therefore, the central difference method cannot be effectively applied in this case.

A second very important consideration is that the integration method requires that step Δt be taken smaller than a critical value, Δt_{cr} , which is given by

$$\Delta t \leq \Delta t_{cr} \leq \frac{T_n}{\pi} \quad (5.5)$$

where T_n is the smallest period of the mathematical model. This scheme requires that the diagonal mass matrix element $m_{ii} > 0$, for all i . The relation in Eq. 5.5 states this requirement again, because a zero element in a diagonal matrix means that the element assemblage has a zero period. In general, all diagonal elements of the mass matrix can be assumed to be larger than zero, in which case Eq. 5.5 gives a limit on the magnitude of the time step Δt that can be used in the integration.

The central difference method is said to be a conditionally stable scheme of integration because the time step of integration Δt should be kept smaller than the critical time step Δt_{cr} , defined in Eq. 5.5, for a satisfactory

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solution. If the time step used is larger than Δt_{cr} , the integration is unstable, meaning that any errors resulting from the numerical integration or round-off in the computer grows without bounds in most cases.

In the following sections we consider commonly employed integration schemes which are unconditionally stable. The effectiveness of an unconditionally stable integration scheme derives from the fact that for an accurate integration, the time step Δt can be selected without a requirement such as the one given in Eq. 5.5 and, in many cases, Δt can be of magnitudes larger than Δt_{cr} defined in Eq. 5.5.

All integration methods discussed in the following sections are implicit, i.e., a triangularization of the stiffness matrix K is required for the solution.

5.2.2 The Newmark γ - β Method

The Newmark integration scheme (29) can be considered an extension of the linear acceleration method. The following assumptions are used:

$$\dot{U}_{t+\Delta t} = U_t + \left[(1-\delta)\ddot{U}_t + \delta \ddot{U}_{t+\Delta t} \right] \Delta t \quad (5.6)$$

and

$$U_{t+\Delta t} = U_t + \dot{U}_t \Delta t + \left[\left(\frac{1}{2} - \beta \right) \ddot{U}_t + \beta \ddot{U}_{t+\Delta t} \right] \Delta t^2 \quad (5.7)$$

where δ and β are parameters that can be determined to obtain integration accuracy and stability. When $\delta = 1/2$ and $\beta = 1/6$, Eqs. 5.6 and 5.7 correspond to the linear acceleration method (which is also obtained using $\theta = 1$ in the Wilson γ - θ method, described in the next section). Newmark originally proposed the constant-average-acceleration method as an unconditionally stable scheme, in which $\delta = 1/2$ and $\beta = 1/4$.

To solve the displacements, velocities, and acceleration at time $t+\Delta t$, the equation of motion, Eq. 5.2, at time $t+\Delta t$ is considered. The complete algorithm using Newmark's integration scheme is given in a table in Reference 26.

5.2.3 The Wilson- θ Method

The Wilson- θ method (28) is essentially an extension of the linear acceleration method, in which a linear variation of acceleration in the time range from t to $t+\Delta t$ is assumed. In this method, the acceleration is assumed to be linear from time t to time $t+\theta\Delta t$. The method reduces to the linear acceleration scheme, but it will be shown later that for unconditional stability θ should be greater than 1.37. Usually, a value of θ equal to 1.4 is used in the practical problems.

Let τ denote the increase in time, where $0 \leq \tau \leq \theta\Delta t$; then for the time interval t to $t+\theta\Delta t$, it is assumed that

$$\ddot{U}_{t+\tau} = \ddot{U}_t + \frac{\tau}{\theta\Delta t} (\ddot{U}_{t+\theta\Delta t} - \ddot{U}_t) \quad (5.8)$$

Integrating Eq. 5.8, one obtains

$$\dot{U}_{t+\tau} = \dot{U}_t + \ddot{U}_t \tau + \frac{\tau^2}{2\theta\Delta t} (\ddot{U}_{t+\theta\Delta t} - \ddot{U}_t) \quad (5.9)$$

and

$$U_{t+\tau} = U_t + \dot{U}_t \tau + \frac{1}{2} \ddot{U}_t \tau^2 + \frac{1}{6\theta\Delta t} \tau^3 (\ddot{U}_{t+\theta\Delta t} - \ddot{U}_t) \quad (5.10)$$

Using Eqs. 5.9 and 5.10, at time $t+\theta\Delta t$,

$$\dot{U}_{t+\theta\Delta t} = \dot{U}_t + \frac{\theta\Delta t}{2} (\ddot{U}_{t+\theta\Delta t} + \ddot{U}_t) \quad (5.11)$$

$$U_{t+\theta\Delta t} = U_t + \theta\Delta t \dot{U}_t + \frac{\theta^2 \Delta t^2}{6} (\ddot{U}_{t+\theta\Delta t} + 2\ddot{U}_t) \quad (5.12)$$

from which $\ddot{U}_{t+\theta\Delta t}$ and $\dot{U}_{t+\theta\Delta t}$ can be solved in terms of $U_{t+\theta\Delta t}$:

$$\ddot{U}_{t+\theta\Delta t} = \frac{6}{\theta^2 \Delta t^2} (U_{t+\theta\Delta t} - U_t) - \frac{6}{\theta\Delta t} \dot{U}_t - 2\ddot{U}_t \quad (5.13)$$

$$\dot{U}_{t+\theta\Delta t} = \frac{3}{\theta\Delta t} (U_{t+\theta\Delta t} - U_t) - 2\dot{U}_t - \frac{\theta\Delta t}{2} \ddot{U}_t \quad (5.14)$$

To solve for displacements, velocities and accelerations at time $t+\Delta t$, the equations of motion given in Eq. 5.2 are considered at time $t+\theta\Delta t$. However, because the accelerations are assumed to vary linearly, a linearly projected load vector is used, i.e., the following equation is used:

$$M\ddot{U}_{t+\theta\Delta t} + C\dot{U}_{t+\theta\Delta t} + KU_{t+\theta\Delta t} = \bar{R}_{t+\theta\Delta t} \quad (5.15)$$

where

$$\bar{R}_{t+\theta\Delta t} = R_t + \theta (R_{t+\Delta t} - R_t) \quad (5.16)$$

The displacement $U_{t+\theta\Delta t}$ can be solved by using Eqs. 5.13, 5.14 and 5.15. The acceleration $\ddot{U}_{t+\theta\Delta t}$ can be found from Eq. 5.13 and can then be used in Eqs. 5.8, 5.9, and 5.10 at $\tau = \Delta t$ to calculate $\ddot{U}_{t+\Delta t}$, $\dot{U}_{t+\Delta t}$ and $U_{t+\Delta t}$. The complete algorithm used in this integration scheme is given in a table in Reference 26.

As pointed out earlier, the Wilson α method is also an implicit integration method because the stiffness matrix K is a coefficient matrix to the unknown displacement vector. No special starting procedures are needed since the displacements, velocities, and accelerations at time $t+\Delta t$ are expressed in terms of the same quantities at time t only. The stability and accuracy of the integration scheme is described in Subsections 5.3.1 and 5.3.2.

5.2.4 The Houbolt Method

The following finite difference expansions are used in this method (27):

$$\ddot{U}_{t+\Delta t} = \frac{1}{\Delta t^2} \left\{ 2U_{t+\Delta t} - 5U_t + 4U_{t-\Delta t} - U_{t-2\Delta t} \right\} \quad (5.17)$$

and

$$\dot{U}_{t+\Delta t} = \frac{1}{6\Delta t} \left\{ 11U_{t+\Delta t} - 18U_t + 9U_{t-\Delta t} - 2U_{t-2\Delta t} \right\} \quad (5.18)$$

These two equations are known as backward-difference formulas with errors of the order $(\Delta t)^2$. The displacement $U_{t+\Delta t}$ can be determined by using Eqs. 5.17 and 5.18 and the following equilibrium equations

$$M\ddot{U}_{t+\Delta t} + C\dot{U}_{t+\Delta t} + KU_{t+\Delta t} = R_{t+\Delta t} \quad (5.19)$$

Thus we obtain

$$\begin{aligned} & \left(\frac{2}{\Delta t^2} M + \frac{11}{6\Delta t} C + K \right) U_{t+\Delta t} = R_{t+\Delta t} + \\ & \left(\frac{5}{\Delta t^2} M + \frac{3}{\Delta t} C \right) U_t - \left(\frac{4}{\Delta t^2} M + \frac{3}{2\Delta t} C \right) U_{t-\Delta t} + \\ & \left(\frac{1}{\Delta t^2} M + \frac{1}{3\Delta t} C \right) U_{t-2\Delta t} \end{aligned} \quad (5.20)$$

As shown in Eq. 5.20, the solution of $U_{t+\Delta t}$ depends on the values of U_t , $U_{t-\Delta t}$, and $U_{t-2\Delta t}$. To start the solution these values can be found by using U_0 , $\dot{U}_0$ and $\ddot{U}_0$. However, a more accurate starting procedure for solving $U_{\Delta t}$ and $U_{2\Delta t}$ is to use a conditionally stable method, such as the central difference scheme. A table in Reference 26 summarizes the integration procedure as it might be implemented in the computer.

One basic difference between the Houbolt method and the central difference method is the appearance of the stiffness matrix K as a factor to the required displacement $U_{t+\Delta t}$. The term $K U_{t+\Delta t}$ appears because the equilibrium is considered in Eq. 5.20 at time $t+\Delta t$ and not at time t as in the central difference method. The Houbolt method is, for this reason, an implicit integration scheme. There is no critical time-step limit that can be used in the integration, and Δt can be selected much larger than that given in Eq. 5.5 for the central difference method.

The step-by-step solution scheme based on the Houbolt method reduces directly to a static analysis if mass and damping effects are neglected, whereas the central difference scheme cannot be used directly for static analysis.

5.3 Schemes for Analysis of Integration Methods

Two principal procedures can be used to solve the following dynamic equilibrium equation:

$$M\ddot{U}(t) + C\dot{U}(t) + KU(t) = R(t) \quad (5.21)$$

where the matrices and vectors have been defined in Subsection 5.2.1. The two procedures are mode superposition and direct integration. Four commonly used integration schemes are described in previous sections. The central difference scheme requires that a time step Δt smaller than a critical time step Δt_{cr} be used; no similar time step limitation is required for the other three integration schemes.

The cost of a direct integration analysis is directly proportional to the number of time steps required for solution. It follows that selecting an appropriate time step in direct integration is very important. On the one hand, the time step must be small enough to obtain an accurate solution; on the other hand, it must not be too small because this would mean that the solution is more costly than necessary. In selecting a proper Δt in the integration scheme, the two fundamental concepts, i.e., stability and accuracy of the integration scheme, should be considered. An analysis of the stability and accuracy characteristics of the integration methods results in guidelines for selecting an appropriate time step.

The fundamental observation for the analyses of a direct integration method is the relationship between mode superposition and direct integration. In essence, both procedures solve by numerical integration. However, in the mode superposition analysis, a change from finite element coordinates to eigenvectors of the generalized eigenproblem,

$$K\phi = \omega^2 M\phi \quad (5.22)$$

is performed prior to the time integration.

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Then

$$U(t) = \phi X(t) \quad (5.23)$$

where ϕ is the modal matrix, and Eq. 5.21 is transformed to

$$\ddot{X}(t) + \Delta \dot{X}(t) + \Omega^2 X(t) = \phi^T R(t) \quad (5.24)$$

where Ω^2 is a diagonal matrix containing the eigenvalues $\omega_1^2, \omega_2^2 \dots \omega_n^2$, and assuming that the damping is proportional, Δ is a diagonal matrix containing $2\omega_i \xi_i$, where ξ_i is the damping ratio in the i^{th} mode.

Eq. 5.24 consists of n uncoupled equations which can be solved, for example, using Duhamel integration. Alternatively, one of the four numerical integration schemes described in Section 5.2 can also be used. Because the period of vibration for each mode is known, an appropriate time step that assumes a required level of accuracy can be used for each modal equation. On the other hand, if all n equations in Eq. 5.24 are integrated using the same time step Δt , then the mode superposition analysis is completely equivalent to a direct integration analysis, in which the same integration scheme and the same time step Δt are used. In other words, the solution of the systems of equilibrium equations would be identical using either procedure. Therefore, to study the accuracy of direct integration, Eq. 5.24 can be considered with a common time step Δt . In this way variables to be considered in the stability and accuracy analysis of the direct integration schemes are only Δt , ω_i , and ξ_i , $i = 1, \dots, n$. Furthermore, because all n equations in Eq. 5.24 are similar, we only need to study the integration of one typical equation, which may be written as

$$\ddot{X} + 2\xi\omega \dot{X} + \omega^2 X = \gamma \quad (5.25)$$

Considering the solution characteristics of a direct integration method, the problem is, therefore, to estimate the integration errors in the solution of

Eq. 5.25 as a function of $\Delta t/T$, ξ and γ , where $T = \frac{2\pi}{\omega}$ is the undamped period of vibration. Various procedures could be followed for this purpose; for example see standard textbooks on numerical analysis. However, a simple procedure based on Ref. 26 is presented in the following section.

5.3.1 Stability Analysis

In the direct integration method it is assumed that the solutions for discrete times $0, \Delta t, 2\Delta t, \dots, t-\Delta t, t$ are obtained and that the solution for time $t+\Delta t$ is required next. Then for a specific integration scheme, the following recursive relationship is established

$$\hat{X}_{t+\Delta t} = A \hat{X}_t + L \gamma_{t+v} \quad (5.26)$$

where $\hat{X}_{t+\Delta t}$ and $\hat{X}_t$ are vectors storing the solution quantities and γ_{t+v} is the load vector at time $t+v$. The values of v , L and A depend on the specific integration scheme under consideration. Eq. 5.26 can be used recursively to calculate the solution at any time $t+n\Delta t$. Thus, we have

$$\begin{aligned} \hat{X}_{t+n\Delta t} = & A^n \hat{X}_t + A^{n-1} L \gamma_{t+v} + A^{n-2} L \gamma_{t+\Delta t+v} + \\ & \dots + A L \gamma_{t+(n-2)\Delta t+v} + L \gamma_{t+(n-1)\Delta t+v} \end{aligned} \quad (5.27)$$

The stability and accuracy of the direct integration method can be studied through the relationship given in Eq. 5.27. The derivation of A and L for each integration scheme is given in Reference 26.

The stability of an integration scheme is determined by examining the behavior of the numerical solution for arbitrary initial conditions. Therefore, we consider Eq. 5.27 with γ equal to zero. An integration scheme is unconditionally stable if the solution for any initial condition does not grow without bound for any time step Δt , in particular when $\Delta t/T$ is large. The method is conditionally stable only if $\Delta t/T$ is smaller than a certain number, usually known as the stability limit.

To investigate the stability of an integration scheme, the spectral decomposition of A is needed and is given by Eq. 5.28; see Ref. 26.

$$A^n = P^{-1} J^n P \quad (5.28)$$

where P is the matrix of eigenvectors and J is the Jordan form of A with eigenvalues λ_i of A on its diagonal. Let $\rho(A)$ be the spectral radius of A , defined as

$$\rho(A) = \max |\lambda_i| \quad i = 1, 2, 3 \quad (5.29)$$

then J^n is bounded for $n \rightarrow \infty$ if and only if $\rho(A) \leq 1$. This is the stability criterion (26). Furthermore, $J^n \rightarrow 0$ if $\rho(A) < 1$ and the smaller $\rho(A)$, the more rapid is the convergence. A comparison of stability between different integration schemes is shown in Exhibit 5.1 for a particular case of $\xi = 0$. The central difference method is only conditionally stable and the Newmark, Wilson, and Houbolt methods are unconditionally stable.

5.3.2 Accuracy Analysis

Generally the accuracy of a numerical integration depends on the loading, the physical parameters of the system and the time step size. If a conditionally stable algorithm such as the central difference method is used, the time step size, and hence, the number of time steps for a given time range considered, is determined by the critical time step Δt_{cr} , and very little choice is available. However, for an unconditionally stable operator, the time step must be chosen to yield an accurate and effective solution. The numerical solution of the equilibrium equations using the different integration methods shows that the errors in the integrations can be measured in terms of period elongation and amplitude decay. In general, the numerical integrations using any one of the methods are accurate (26) when $\Delta t/T$ is smaller than about 0.01. However, when the $\Delta t/T$ ratio is larger, the various integration methods exhibit quite different characteristics. Notably, for a given ratio $\Delta t/T$, the

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Wilson θ method with $\theta = 1.4$ introduces less amplitude decay and period elongation than the Houbolt method, and the Newmark constant-average-acceleration method ($\delta = 1/2$, $\beta = 1/4$) introduces only period elongation and no amplitude decay.

An important problem is to choose an appropriate time step Δt . Using the central difference method, the time step must be chosen such that Δt is smaller than Δt_{cr} . However, when using one of the unconditionally stable schemes, the time step Δt can be much larger and should only be small enough to calculate an accurate response to all modes which significantly contribute to the total structural response. The other modal response components are not evaluated accurately, but the errors are unimportant because the response measured in those components is negligible.

It was previously mentioned in this section that the time step of integration Δt usually depends on the type of numerical integration scheme used in the computer program and the type of problem under consideration. However, our experience shows that in the dynamic analysis of power plant structures, $\Delta t = T/10$, where T is the shortest period of interest, is adequate to reasonably calculate an accurate response when using unconditionally stable schemes of numerical integration. This is demonstrated in the following section through the dynamic analysis of an underground nuclear power plant structure.

5.4 Analysis of an Underground Nuclear Power Plant Structure

Exhibit 5.2 shows the sectional elevation of the axisymmetric reinforced concrete nuclear power plant structure with the surrounding soil, and the location of the shock load generated from a nuclear blast. Material properties for concrete and soil in each of the four zones shown in Exhibit 5.2 are given in Table 5.1. The shock load consists of a peak pressure of 50 psi acting on the surface of a sphere 60 feet in diameter. The triangular pressure time history of the shock load is shown in Exhibit 5.4. The magnitude of the peak pressure is arbitrarily chosen. The purpose is to compute the response at various locations in the containment structure.

5.4.1 Method of Analysis

A finite element method of analysis is used to calculate the response of the soil-structure system due to the propagation of waves generated in the medium by the shock load. Axisymmetric finite elements are used to represent the geometry of the system: conical, thin shell elements represent the concrete structure; the axisymmetric, quadrilateral and triangular solid elements (30) are used to model the soil continuum.

The equations of motion are integrated using the Newmark- β method (29). The DYNAX computer program (30), which is based on the theory discussed above, is used in the analysis.

5.4.2 Element Size and Time Step

The element size of the finite element model and the lengths of the time step used in the analysis have considerable effect on the final results. The criteria for selecting the element size and the time step of integration used in the analysis are described in the following section. To model an infinite medium by a finite medium and to eliminate the reflected waves from the boundary, a special type of viscous boundary (31) is assumed at the base and the side of the finite element model. The damping constants α and β are computed by matching the critical damping ratio of 2 percent at frequencies of 10 Hz and 40 Hz.

A. Element Size

In solving the problem of wave propagation using the finite element method, the element size determines the highest frequency content of the model. An approximate rule to determine element size Δl is to make it equal to or less than $c/4f$, where Δl is the maximum size of the elements, c is the wave speed, and f is the highest desired frequency content of the model. The desired frequency content of this model is about 40 Hz.

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B. Time Step

The DYNAX program uses the Newmark - β method in the integration scheme. It is an approximate method of integration and the accuracy of the solution depends on the time step size used. A large time step induces the erroneous effects of period elongation and amplitude deamplification (26) in the system's response. An appropriate time step is usually taken to be a fraction of the minimum period of interest, i.e., T/N or $1/Nf$, where T is the minimum period of interest, f is the frequency corresponding to the period T , and N is a constant. It was pointed out in Subsection 5.3.2 that $\Delta t = T/10$ is sufficient to reasonably integrate an accurate response. Therefore, a time step of 0.0025 sec. is assumed to be adequate to integrate the response up to 40 Hz frequency.

5.4.3 Results

After determining the basic parameters needed to compute the response, the time-history analysis is performed for 0.20 sec. The time histories and response spectra for 5% damping computed for various locations in the containment structure are shown in Exhibits 5.5-5.8. To check the accuracy of the results, the same analysis is performed by varying the length of the time step of integration. Results for maximum acceleration at one of the nodes are listed in Table 5.2, which shows that Δt smaller than 0.0025 produces essentially the same results as $\Delta t = 0.0025$, whereas a larger value produces different results from those at .0025 sec. Therefore, for the given element size with a maximum representative frequency of about 40 Hz, a time step of .0025 sec. (i.e., $T/10$) is adequate. However, if the desired frequency of interest is more than 40 Hz, the parameters needed to calculate responses must be computed again by a method similar to the one described in Subsection 5.4.2.

6. METHODS USED TO EVALUATE NON-SEISMIC RESPONSES AND APPROACHES TO COMBINING VARIOUS RESPONSES

This chapter briefly describes the load combination methods used in the design of a nuclear power plant. Details of the topic are part of a separate task of the Seismic Safety Margin Research Program (SSMRP).

6.1 Definition of Loading Conditions

- D - The dead load of the structure plus any other permanent load, including vertical and lateral pressure of liquids, piping, cable pans, and weight of permanent equipment and their contents.
- L - The conventional floor and roof live loads, moveable equipment loads, and other loads which vary in intensity, such as lateral soil pressure. Live load intensities may vary from zero to their maximum values to determine the most critical effects on the structure for the load combination under consideration.
- P_o - The normal operating pressure differentials.
- P_a - The design accident pressure load or the equivalent static load within or across a compartment and/or building, including appropriate dynamic load factor (dlf).
- P' - Test pressure.
- R_o - Normal operating or shutdown reactions of piping at supports or anchor points based on the most critical transient or steady state condition. These reactions are provided by the piping engineers.
- R_a - Pipe reactions generated by the design-basis accident, including R_o .

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- Y_r - Equivalent static load generated by the reaction due to the ruptured high-energy pipe during the postulated break, including appropriate dlf. The postulated pipe break could be either a cross-sectional rupture or a longitudinal rupture. The longitudinal rupture is assumed to be equal to twice the pipe diameter. The rupture area multiplied by the internal pipe pressure will give the dynamic force generated by the rupture.
- Y_j - Jet impingement equivalent static load resulting from a high-energy pipe break, including appropriate dlf.
- Y_m - Missile impact equivalent static load generated by or during the postulated break, including dlf. These loads could be generated by pipe whipping or by flying parts of pipes or restraints which might strike the structure.
- E - Loads generated by operating basis earthquake (OBE), including dynamic lateral soil pressure and hydrodynamic pressure of liquids.
- E' - Loads generated by the safe shutdown earthquake (SSE), including dynamic lateral soil pressure and hydrodynamic pressure of liquids.
- T_o - Thermal effects and loads associated with normal operating, shutdown, construction and test conditions.
- T_a - Thermal effects and loads generated by the design-basis accident, including T_o .
- H' - Forces associated with maximum probable flood, such as hydrostatic pressure and submergence consideration.
- W - Design wind load. The design wind pressure is obtained from the design wind velocity which will be considered for the project site. Velocities of 50-year recurrence and 100-year recurrence may be used.

W' - Tornado load and loads from tornado-generated missiles. The pressure and loads generated by the tornado can be calculated after obtaining the appropriate data associated with tornados in the vicinity of the site. Tornado-generated missiles include objects that can be uprooted or dragged into the tornado current and can strike the structure or the building.

M - Loads associated with missiles other than tornado- and postulated accident-generated missiles. Rotating equipment with the potential to be subjected to overspeed conditions may separate and act as a missile striking other structures.

6.2 Load Combination Categories

The load categories defined here include any condition encountered during construction, normal operation of the plant, single failure of the reactor coolant system, and extreme environmental conditions postulated during the life of the facility. Structures will be analyzed and designed to meet performance and strength requirements for the applicable load combinations given in Table 6.1. The load categories are defined as follows:

- A. Construction - includes all loads during construction.
- B. Test - includes all loads during the structure integrity test.
- C. Normal - includes all loads on the structure during normal operating and shutdown.
- D. Abnormal - includes pressure loads and temperature effects due to pipe break accident. It includes pipe rupture loads and impingement loads. Also, missile effects other than tornado.

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- E. Severe environmental - includes loading during operating basis earthquake and design wind.
- F. Extreme environmental - includes loadings which are highly improbable, such as a safe shutdown earthquake and forces due to tornado.
- G. Abnormal/Severe environmental - simultaneous occurrence of loadings D and E.
- H. Abnormal/Extreme environmental - simultaneous occurrence of loadings D and F.

6.3 Design Criteria

A. Structural Steel

All systems are designed in accordance with the AISC Manual of Steel Construction. The loading combination and allowable stresses are defined in Table 6.1a.

B. Reinforced Concrete

All concrete structures will be designed using the load combinations given in Table 6.1b, in accordance with the ACI Building Code Requirements for Reinforced Concrete Ultimate Strength Design.

C. Special Systems

Include pipe hangers, pipe whip restraints and penetration anchors. Table 6.1c will be applied for load combinations.

6.4 Analysis

A. Steel Structures

The design forces are computed by conventional methods. The following computer programs can be used for the analyses: STRESS, STRUDL, SLSAP, STAND and EASE.

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B. Concrete Structures

All major concrete structures are analyzed by the finite element techniques using computer programs such as SLSAP, DYNAX, PLFEM and CSEF.

All loading combinations will be considered in the analysis. Areas of significant stress concentrations, such as discontinuities and around openings, will be identified and reanalyzed using a finer grid and the boundary conditions obtained from analyzing the entire structure. For both normal operating and accident conditions, the temperature will be calculated applying the principles of heat transfer. The stresses in the concrete and reinforcing will be found by using the computer program TEMCO, which takes into account the extent of cracking of the section. The moment resulting from the thermal gradient is the only stress resultant which is permitted to change due to cracking. For sections which are critical in terms of allowable stress, the capacity of a section under combined loads will be checked by using TEMCO.

A dynamic plastic analysis will be used in flexure for the effects of missiles and pipe ruptures in loading combinations F, G and H, as given in Section 6.2.

7. STRUCTURAL DAMPING VALUES

7.1 Introduction

The energy dissipation within a structure due to the material and structural damping resulting from an earthquake depends on factors such as types of joints or connections within the structure, the structural material, and the magnitude of deformations experienced. In a dynamic elastic analysis, this energy dissipation is usually accounted for by specifying an amount of viscous damping resulting from energy dissipation in the analytical model which is equivalent to that expected to result from material and structural damping in the real structure. In this chapter the damping values used in the design of nuclear power plants and damping values obtained from a limited number of tests of structures and components are discussed.

7.2 Damping Values for Design

In the design of power plants, modal damping values as recommended in NRC Regulatory Guide 1.61 (40) are used. Table 7.2 lists the recommended damping values for various types of structures and piping. When the modal method of solution is used and all components of the structure have the same modal damping, the values of the modal dampings given in Table 7.2 can be directly used. In general, however, the different components of a power plant's complex structures are made of materials with different damping values, e.g., the prestressed concrete containment, reinforced concrete shear wall floor slab and base mat, steel floor beams, girders and supporting columns. For these cases the equivalent modal damping is determined using the mixed modal damping approach (41, 42). Two relations which are extensively used today to determine an equivalent modal damping matrix are based on proportioning the damping factors according to the mass or stiffness of each element. The formulation leads to:

$$\bar{B}_j = \frac{\sum_{i=1}^n \langle \phi_i \rangle [M]_{ij} \left\{ \phi_i \right\}}{\langle \phi_j \rangle [M] \left\{ \phi_j \right\}} \quad (7.1)$$

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$$\bar{\beta}_j = \frac{\sum_{i=1}^n \langle \phi_j \rangle [K]_{ii} \beta_i \{ \phi_j \}}{\langle \phi_j \rangle [K] \{ \phi_j \}} \quad (7.2)$$

where

n = total number of elements

$\bar{\beta}_j$ = equivalent modal damping ratio of the j^{th} mode

$\{ \phi_j \}$ = the undamped mode shape vector for j^{th} mode

$[M]$ = mass matrix of the system

$[K]$ = stiffness matrix of the system

β_i = damping ratio of the i^{th} element (Table 7.2)

$[M]_i$ = mass matrix associated with element i

$[K]_i$ = stiffness matrix associated with element i

When a direct integration approach is used, the modal damping values given in Table 7.2 cannot be directly used in the analysis because modal frequencies and shapes are not evaluated in this solution technique. Instead a damping matrix consistent with the specified modal damping values must be generated. When the solution integration scheme requires a mass and stiffness proportional damping, the damping matrix C is determined as follows:

$$C = \alpha M + \beta K \quad (7.3)$$

where α and β are scalar constants. The values of α and β are determined from prescribed damping values as follows:

$$\alpha = \frac{2\omega_1\omega_2}{(\omega_2^2 - \omega_1^2)} (\xi_1\omega_2 - \xi_2\omega_1) \quad (7.4a)$$

$$\beta = \frac{2(\xi_2\omega_2 - \xi_1\omega_1)}{(\omega_2^2 - \omega_1^2)} \quad (7.4b)$$

where ω_1 and ω_2 are two frequencies and ξ_1 and ξ_2 are the system damping values at these two frequencies, respectively. The values of $\omega_1, \omega_2, \xi_1$ and ξ_2 are chosen so that in the frequency range of interest Eq. 7.3 yields damping values close to the actual damping in the structure.

7.3 Damping Values Based on Tests

The damping in structures is a function of the type of material, type of construction and the state of stress produced by the excitation. For example, concrete structures have higher damping than steel structures; bolted steel structures have higher damping than welded steel structures. Similarly ductile structures stressed to or beyond their yield limits will absorb more energy, and hence will have a higher equivalent damping than similar structures at very low stress levels.

Tests of the actual structure and their scaled models provide a viable means to determine the damping values under different levels of excitation and states of stress. However, the data on nuclear plant structures are very limited. Most of the tests reported to date have been on scaled models and on non-nuclear structures. The following summarizes a series of tests and includes both nuclear and other types of structures and components, which at this time can serve as a guide for dynamic analysis.

7.3.1 San Onofre Nuclear Generating Station Vibration Tests (32, 33, 34)

A series of forced vibration tests were performed on the San Onofre Nuclear Generating Station (Unit 1) at the request of the Southern California Edison Company. The tests, conducted by a group from the University of California at Los Angeles, focused on the reactor building and primary coolant loop. Coolant loop "B", which consisted of a pressurizer, steam generator, and primary pump, was subjected to detailed tests. The main objective of the steady-state vibration tests was to develop an analytical model that would predict the response of particular components in the power plant when subjected to strong ground motions.

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The San Onofre Nuclear Generating Station (SONGS) is based on a pressurized water-type reactor capable of generating 1347 MW of heat and 450 MW of continuous electrical energy. The reactor pressure vessel, primary coolant loop, and certain auxiliaries are located in a 140-ft diameter steel containment sphere. Reinforced concrete foundations and structural walls, also located inside the sphere, provide support and shielding for equipment. The base of the containment sphere, located 40 ft below grade, is continuously supported by a concrete cradle between the steel sphere and the soil.

All components, systems, and structures important to the nuclear safety of the power plant were designed according to the following guidelines:

- a. The primary steady-state stresses combined with seismic stresses, resulting from a ground motion with a maximum acceleration of 0.25 g, must be less than the allowable working stress.
- b. The primary steady-state stresses combined with seismic stresses, based on a ground motion with a maximum acceleration of 0.5 g, must not impair the function of the component, system, or structure, nor prevent a safe and orderly shutdown.

The forces applied during the tests were provided by two vibration generators anchored on the operating floor equidistant from the north-south centerline of the reactor and on a line parallel to the east-west centerline. Two sizes of vibration generators were used in the tests. The larger generator was effective up to 9.5 cps; the frequency range of the smaller generator reached 55 cps. The large and small vibration generators were capable of developing forces as large as 5,000 lbs and 10,000 lbs, respectively.

The responses of major cooling loop components, the reactor vessel, the containment sphere, and the operating floor were measured with accelerometers and strip chart recorders. When the response of a particular major component of the cooling loop was desired, four accelerometers were placed

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on that component and one accelerometer was placed on each of the other two major pieces of equipment. One accelerometer was placed at the top, two at the middle, and one at the bottom of the component being studied. The accelerometers at the top and bottom, and one of the middle accelerometers, were oriented in the forcing direction. The other middle accelerometer was oriented perpendicular to the forcing direction. Three accelerometers were placed on the reactor vessel head flange; two were oriented in the north-south direction, and the third was oriented in the east-west direction. Additional accelerometers were located on the operating floor and containment sphere, on the north-south centerline.

Experimental data were matched best by a linear, viscously damped, lumped parameter model with existing orthogonal modes. The eight-degrees-of-freedom model described the east-west response of the steam generator, pressurizer, pump, reactor vessel, operating floor, and containment sphere, as well as the north-south response of the pressurizer and a 4 cps response that appeared to be the coupling of the steam generator and pump response.

An iterative procedure was used to determine system masses and mode shapes from experimental results. However, mode shapes determined by this procedure were not orthogonal. A gradient search on the orthogonality error was used to obtain orthogonal modes that approximately produced the experimental response. Closer agreement with the experimental data was obtained by adjusting the masses with a common factor and using a least-mean-square-fit method. Responses at a particular resonant frequency were also adjusted by making appropriate changes in damping ratios.

Initial estimates of damping were determined from response curves with the single-degree-of-freedom formula relating frequency width at one-half of peak frequency to the peak frequency. Refinement of a particular damping ratio, after the least squares fitting method was completed, was accomplished by multiplying the damping ratio by the ratio of the sum of calculated displacements for the appropriate mode to the sum of measured displacements for that same mode.

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Damping factors determined from the steady-state vibration tests (expressed as percent of critical) were as follows: 2.7 for the steam generator; 0.7 for the east-west response of the pressurizer; 1.5 for the north-south response of the pressurizer; 0.6 for the pumps; 2.7 for the coupled response of the pump and steam generator; 2.5 for the operating floor; 1.5 for the reactor vessel; and 4.6 for the containment sphere.

7.3.2 Response and Energy Dissipation of Reinforced Concrete Frames Subjected to Strong Base Motions (35)

Gulkan and Sozen tested small-scale frames in order to develop an analytical model for predicting the response of reinforced concrete systems subjected to earthquakes. From these tests emerged a method for determining a substitute damping ratio that is used with a linear model to match the maximum experimental response of the system.

The test specimens were reinforced concrete, one-story, one-bay frames. Two types of specimens, representing approximately one-eighth and one-fourth scale models of a frame, were tested. The smaller frame had gross section dimensions of 5 by 5 in. and 5 by 10 in. for the columns and girder, respectively. The girder spanned 29.5 inches from centerline to centerline of the columns. The columns measured 13 inches from the foot of the column to the bottom of the girder. The dimensions associated with the larger specimen are double those of the smaller specimen. Reinforcement ratios for the columns were 0.018 and 0.016 for the smaller and larger specimens, respectively (based on gross section dimensions).

Some of the specimens were subjected to steady-state (sinusoidal) base motions; others were excited by simulated earthquakes. The earthquake record applied to the smaller specimens was the N-S component of the 1940 El Centro earthquake, with duration compressed by a factor of eight. The larger specimens were subjected to the N21E component of the 1952 Taft earthquake, with duration compressed by a factor of five.

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The simulated earthquakes were designed so that yielding occurred in the specimens during the first earthquake simulation test of each specimen. Steel plates weighing 4000 lbs and 649 lbs were attached to the girders of the larger and smaller specimens to enable development of the desired inertia forces. All dynamic tests were carried out on the University of Illinois Earthquake Simulator.

The maximum response of the test specimens was reconciled with a linear system that used substitute damping and assumed the specimens to be single-degree-of-freedom systems.

A substitute damping ratio was determined as follows:

- a. The relative velocity function was determined from the relative acceleration record using a trapezoidal integration formula.
- b. The energy input to the system was determined from the base-acceleration record and the relative velocity function.

$$W = -m \int_0^T \ddot{y} \dot{x} dt$$

- c. The constant of proportionality (c) for the idealized linear damper was calculated:

$$c = \frac{-W}{\int_0^T \dot{x}^2 dt}$$

- d. The characteristic circular frequency (ω_o) was calculated:

$$\omega_o = \frac{\sqrt{\frac{(\ddot{x} + \ddot{y})_{\max}}{x_{\max}}}}$$

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e. The substitute damping ratio was calculated:

$$\xi_s = \frac{c}{2m\omega_0}$$

The characteristic circular frequency was based on the effective stiffness obtained from joining the origin and the point on the primary curve corresponding to the maximum response. Primary curves for both specimen types were described by static tests. The values of the substitute damping ratio calculated for initial steady-state tests and earthquake simulation tests for the smaller specimens ranged from 6.2% to 8.7%, with a mean of 7.4%. Values for the larger test specimens varied more, ranging from 1.8% to 15.4%, with a mean of 8.4%.

7.3.3 Damping Values of Nuclear Power Plant Components (36)

Several published and unpublished damping values of nuclear power plant components and other structures were investigated. The major emphasis was placed on the values for the primary coolant loop components, such as the steam generator, piping, etc. The results of this investigation, summarized in Table 7.1, consist of damping values recorded for various components as a function of the type and magnitude of excitation, where applicable. The data have been obtained from several sources, including (1) the recorded response of actual nuclear power plant components to seismic excitations and forced vibrations, and (2) results from laboratory and other field tests. For some cases involving several small displacements, only the damping values for the larger displacements are shown in Table 7.1. The variation with the displacement is then presented in graphical form by the applicable figures. The damping values in Table 7.1 associated with the primary coolant loop equipment were also plotted in Exhibit 7.1. The resultant linear fit curve on this figure, which is based on data from three sources, illustrates the tendency of the damping to increase with amplitude.

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Structural systems are subjected to two main kinds of damping mechanisms: (1) material (hysteretic) damping, which corresponds to the internal energy dissipation within the structural material at the microscopic boundaries; and (2) system damping, which is related to the energy dissipation at macroscopic boundaries, such as interface shear at structural joints and supports. The small damping values currently being used in seismic analyses of primary coolant loop components appear to be estimates based mostly on material damping and on low levels of excitation. These are typically 0.5% to 1% of critical for the Operating Basis Earthquake (OBE), and 1% to 2% for the Design Basis Earthquake (DBE). Laboratory tests on pipe specimens have shown that these small material damping values are valid for low stress levels; however, for higher stress levels, considerably higher damping values were obtained. Material damping would be applicable, for example, to small isolated piping whose vibrations do not appreciably affect the whole system, and which have little friction at the support connections. For larger piping and component systems where the structural response is interdependent, system damping appears to be predominant. This is shown by the data obtained from the response of these components to actual earthquakes and to forced vibrations.

The actual damping values of a component or structural system for a certain level of excitation are difficult to obtain, particularly when the calculation method is based on the resonant buildup from forced vibration tests. In this case, any inaccuracies in the vibration input and in the duration of the input at the resonant frequencies tend to overestimate the damping. During the laboratory tests on pipe specimens, this effect was minimized by performing a dwell test at resonance. The sweep rate was then set slow enough so that it yielded a maximum response amplification equal to that given by the dwell tests. This effect was also considered in the forced vibration tests of Refs. 32 and 33. For these tests, the vibrators were reportedly held at a constant frequency until all transient effects decayed, so that only the steady-state response of the system was measured. Even more care and accuracy are required to obtain and measure material damping. The support connections for the pipe specimens had to be carefully designed to eliminate

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friction during the laboratory tests. Slip and interface shear at the supports resulted in considerably higher damping values. Therefore, some of the damping values reported are probably overestimated. On the other hand, additional equivalent damping may be obtained during a large seismic disturbance due to a slight shift in the natural frequencies of the system; this, in turn, is due to stiffness degradation such as loosening of joints and cracking of concrete. This effect would be important for the floor-supported equipment, which is designed to response spectra derived with narrow-band floor motion.

7.3.4 Damping for Dynamic Analysis of Reactor Coolant Loop Systems (37)

Experimental work has been performed to determine the dynamic characteristics of the primary coolant loop in nuclear power generating stations. Forced vibration tests have been conducted on the Experimental Gas-Cooled Reactor (EGCR) in Oak Ridge, the San Onofre Nuclear Generating Station (SONGS), and the Indian Point II Power Plant. Underground blast tests were performed on the Fermi Nuclear Power Plant and the EGCR. SONGS was also excited by the Lytle Creek and the San Fernando earthquakes on September 12, 1970 and February 9, 1971, respectively.

Energy losses during excitation of the reactor coolant loop are attributed to internal and external damping, viscous damping, damping by impact, etc. From the analytical point of view, these damping forces can be replaced by an equivalent viscous damping that dissipates the same amount of energy.

Equivalent viscous damping ratios for SONGS and EGCR were obtained from sinusoidal vibration tests. During the tests, the forcing frequency was varied to determine the natural frequencies of each system. Damping values were determined from the Fourier spectra by measuring the band widths of the peaks at half-power points and using $\beta = \frac{\Delta\omega}{2\omega}$.

Damping values for the Indian Point Power Plant were determined by measuring free decaying vibrations which resulted from exciting the system with

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half-beats at the natural frequencies and evaluating damping from the relation.

$$\beta = \frac{1}{2\pi N} \ln \left(\frac{x_1}{x_N} \right) \quad (7.5)$$

where the two peaks of interest are N cycles apart. The natural frequencies of the system were obtained by applying a low-amplitude sinusoidal load and by varying the forcing frequency. Damping ratios from blast tests on the EGCR were determined in the same manner.

The response of SONGS to the San Fernando earthquake was submitted to a spectrum analyzer. The resulting power spectral density function contained a number of resonance peaks corresponding to the system's natural frequencies. Damping values were determined from these peaks using the band width method.

The vibration tests on the EGCR yielded equivalent viscous damping ratios of 0.5% to 1.5% of critical for the coolant loop and 3% to 7% of critical for the steam generator. The larger damping ratios were obtained at larger values of deflection. Damping ratios of 1% to 3% were measured at the top of the steam generator for the blast tests. Similar vibration tests on SONGS produced damping ratios of 0.8% to 1.2%, 1% to 2%, and 1% to 2% for the pump, pressurizer, and steam generator, respectively. Vibration tests on the Indian Point II Power Plant produced larger equivalent damping ratios for the steam generator than were measured at SONGS. However, no major differences were apparent for the damping ratios associated with the pump. Damping ratios ranged from 0.7% to 5% for the steam generator, 0.6% to 1.5% for the pump, and 3.8% to 5% for the piping.

The response of SONGS to earthquake excitations yielded larger damping values than those determined from the vibration tests. Damping ratios of 3.4% to 5% for the steam generator and 2% to 9% for the pressurizer were obtained from the response to the Lytle Creek earthquake. Damping ratios

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from the response to the San Fernando earthquake were 2% to 4% (+1%) for the steam generator and pump and 2% (+1%) for the pressurizer. The larger damping ratios were explained by the greater nonlinear behavior of the system, enhanced by large amplitude vibrations.

7.3.5 Experimental Study of the Dynamic Response of a Ten-Story Reinforced Concrete Frame with a Tall First Story (38)

Small-scale, ten-story, three-bay, reinforced concrete frames with a tall first story were tested by Healey and Sozen to study nonlinear dynamic behavior. The frames had first- and tenth-story heights of 279 mm, measured from centerline to centerline of the beams. The second- through ninth-story heights were 229 mm. The beams spanned 305 mm from centerline to centerline of the columns. Gross section dimensions of the beams and columns were 38 mm by 38 mm and 58 mm by 38 mm, respectively. Columns at the base and second level interior columns had reinforcement ratios of 0.0132 (based on gross section dimensions). Beams in the first through seventh levels had reinforcement ratios of 0.0110 (based on beam width and effective depth). All other beams and columns had reinforcement ratios of 0.0074 and 0.0088, respectively. A 4540 kg concrete and steel mass supported by a pair of frames at every beam level was used. The masses were necessary to develop the desired inertia during the tests.

The test structure was designed using a linear dynamic analysis that incorporated the cracked section stiffness of each member modified by an appropriate damage ratio. In the initial stages of design, the ratio of cracked to gross-section moment of inertia was one to three for beams and columns. The damage ratio was taken as four for beams and one for columns. A design response spectrum considering a modified time scale (condensed by 2.5) was used for the dynamic analysis. The maximum base acceleration for the design earthquake was 0.4 g.

The test structure was subjected to free-vibration tests, steady-state tests (constant amplitude, sinusoidal base motion) and simulated earthquake tests.

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The free-vibration test consisted of displacing the structure at the top level a small distance, then releasing the structure to vibrate. The simulated earthquake tests were for various amplifications of the N-S component of the 1940 El Centro earthquake with duration compressed by a factor of 2.5. All dynamic tests were conducted on the University of Illinois Earthquake Simulator.

An equivalent damping ratio was calculated for the test structure from the tenth level acceleration record obtained from the free-vibration test that preceded the first simulated earthquake test. Assuming a linear model, a damping ratio of 1.9% of critical was calculated using the log-decrement method.

7.3.6 Earthquake Simulation Tests of a Ten-Story Reinforced Concrete Frame with a Discontinuous First Level Beam (39)

Tests were conducted by Moehle and Sozen on small-scale, ten-story, three-bay, reinforced concrete frames with flexible lower stories and tall first and tenth stories. The object of the tests was to observe the effect of strong simulated earthquake motions on the test frames.

The basic test structure consisted of two frames facing each other, with masses spanning the distance between them. The mass at level one was nominally 302 kg; all other masses were 454 kg. Masses were used for support and to increase inertial forces during the tests. Flexible lower stories were partially the result of one beam being omitted from one side of both frames. Story heights were 279 mm for the first and tenth stories, and 229 mm for all other stories. Bay widths were 305 mm. Gross dimensions of beams and columns were 38 mm by 38 mm, and 38 mm by 51 mm, respectively.

Test frames were designed according to the substitute structure method, using a linear modal spectral analysis. It was specified that lateral displacements less than displacements related to yielding in the frames were not to be exceeded if the test structure experienced a base motion corresponding to the design spectrum. The maximum acceleration for the design earthquake

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simulation was 0.4 g. Stiffnesses of substitute structure elements were related to cracked-section stiffnesses of actual elements by appropriate damage ratios. Ratios of four for beams and one for columns were chosen so that energy was primarily dissipated by the beams.

Longitudinal reinforcement was selected according to design forces and moments. However, frequent changes in reinforcement were avoided to make construction reasonable. Design forces and moments were root-sum-square forces and moments. Column-end moments were multiplied by a factor of 1.2, except at the base of the frames, to assure yielding in the beams only. The first level of the interior columns and the first and second levels of the exterior columns had a reinforcement ratio of 1.76% (based on gross area). All other column levels had a reinforcement ratio of 0.88%. The first through seventh level beams and the eighth through tenth level beams had reinforcement ratios of 1.10% and 0.74%, respectively (based on beam width and effective depth).

The test structure was subjected to simulated earthquake motions, free-vibration tests, and steady-state tests. The simulated earthquake motion was modeled after the N-S component of the 1940 El Centro earthquake. The time duration of the earthquake was compressed by a factor of 2.5 and the accelerations were amplified by a different factor for each test. A free-vibration test consisted of displacing the structure at the top level by some small amount, then suddenly releasing the displacing force, allowing the structure to vibrate freely. Steady-state tests subjected the test structure to a sinusoidal base motion with constant amplitude and slowly increasing frequency. These dynamic tests were conducted on the University of Illinois Earthquake Simulator.

An equivalent viscous damping ratio was calculated from the tenth level acceleration record of the first free-vibration test. Assuming a linearly elastic system, an equivalent damping ratio of 2% of critical was calculated using the logarithmic-decrement method.

It is to be noted that damping values quoted in Subsections 7.3.5 and 7.3.6 correspond to values at low amplitude vibration. As the frames were progressively subjected to high intensity loads, the damping of the damaged frame increased. In particular, for the frame described in Subsection 7.3.5, the equivalent damping increased from 1.9% to 5.6%, 7.8% and 10.2% after three runs of high-intensity, earthquake-like loads were applied to the frame.

8. NONLINEAR RESPONSE ANALYSIS

8.1 Introduction

Nonlinear structural response is of interest in the earthquake engineering field because it is recognized that most civil structures cannot resist even moderate intensity earthquakes without experiencing some degree of nonlinearity. The design of nuclear power plants is presently based on elastic analysis. If the behavior of nuclear power plants were to be investigated for earthquake levels higher than the design levels, nonlinearities caused by material behavior and geometric deformation magnitude (P- Δ effects) would be of principal importance.

The nonlinear response of single-degree-of-freedom (SDF) systems has received extensive attention in the literature. With reference to earthquake nonlinear response, approximate bounds for maximum response of SDF systems are available in Ref. 43, and specifically for bilinear hysteretic systems in Refs. 44 and 45. The general effects of nonlinearity on multi-degree-of-freedom (MDF) systems have often been studied, but because of the number of parameters affecting the behavior of such systems it has not been possible to effectively generalize the results and arrive at conclusions of wide applicability, such as for SDF systems.

Evidently, nonlinear analysis of MDF systems to a prescribed time history of ground motion can be carried out by the step-by-step integration of equations of motion, dividing the response history into short time increments and assuming that the properties of the structure remain constant during each increment but change in accordance with the deformation state existing at the end of the increment. Thus, nonlinear analysis actually is treated as a sequence of linear analyses of a successively changing structure. In practice, application of such an analysis - herein called detailed nonlinear analysis (DNA) - requires specific attention to the following items:

1. Stiffness properties of members (which are needed to formulate the stiffness of the system to be applied during a time increment, based on

the deformation state at the end of the preceding increment) should be available from analytical studies guided by experimental results.

2. It is known that structural response is sensitive to the specific record used in the analysis; therefore, an analysis for a collection of a consistent set of records is required.
3. Execution of DNA on computers for moderately complex structures of nuclear power plants, even using one ground motion, is expensive, and because of the effort needed to carry out DNA, parametric studies of design variables cannot be made conveniently.

In regard to Item 1, although additional analytical and experimental studies are needed, particularly for reinforced concrete shear walls and at frequencies reflecting those encountered in power plants and not in tall buildings, algorithms are currently available to implement practical solutions. The process of collecting a consistent set of records, as mentioned in Item 2, can currently be handled by using simulated earthquake motions. In this manner a collection of records corresponding to the same intensity can be considered without adding variabilities due to different source mechanisms and transmission paths. Procedures for generating such motions for a variety of conditions are available.

The only way to avoid the difficulty mentioned in Item 3 is to have reliable, approximate methods available for making nonlinear, dynamic response calculations. Several proposals, including the use of an inelastic response spectrum (46), reserve energy technique (47), and the substitute structure method (48), have been made. However, the reliability of these methods as a replacement for a DNA application has not been extensively studied for a variety of structures and responses of interest, such as floor spectra. In fact, there is evidence that these methods should be modified to make them suitable (49). Therefore, it is necessary to use DNA in conjunction with simulated earthquake records to study the reliability of the above methods for the specific structural types and responses of interest. Such a study should also

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consider the applicability of other approximate methods, such as the stochastic linearization techniques which are presented in Refs. 50 and 51.

This chapter focuses on the state of the available stiffness algorithms and methods of generating artificial earthquake records, and discusses isolated results which are now available on the reliability of the specific, approximate, nonlinear analysis techniques cited above. Finally, the use of nonlinear methods suitable for SSMRP is discussed.

8.2 Solution Algorithms

Three major groupings of building structures in a nuclear power plant are considered for seismic analysis: the containment structure, the auxiliary buildings, and the turbine building. The containment structure is a continuous shell-type structure. The auxiliary buildings are generally heavy, reinforced concrete, shear wall structures with the shear walls interconnected by concrete floor diaphragms. The turbine building complex is typically a series of transverse ductile frames that are connected longitudinally by bracing or shear walls.

Ref. 52 describes an investigation of the available computer codes for applying a DNA-type analysis to these three types of building. The results of this investigation show that no computer software is currently available to efficiently apply DNA to reinforced concrete containment shell-type structures. A procedure for treating reinforced concrete as orthotropic material and allowing for cracking of concrete as loading is changed is available from Ref. 73. Special computer coding techniques are needed to implement a DNA-type analysis of containment structures using this procedure. The studies in Ref. 52 show the DRAIN-TABS computer code is an efficient means to apply DNA-analysis to auxiliary and turbine buildings. Table 8.1 summarizes the computer codes which have been reviewed in Ref. 52.

In view of the above observation, the element library and material models in DRAIN-TABS are considered to have the most useable algorithms for the

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stiffness formulation required in a DNA analysis. The following structural elements are used in the DRAIN-TABS program: truss, beam with degrading stiffness, shear (infill) panel, and beam column. Elements are assumed to be connected to form horizontal load-resisting elements oriented in vertical planes. These substructures are connected by rigid horizontal diaphragms representing the floors. In this manner, the three-dimensional behavior is considered. A brief description of each element follows.

8.2.1 Truss Element

Truss elements may be arbitrarily oriented in the two-dimensional plane but can transmit axial loads only. Two alternative modes of inelastic behavior may be specified: yielding in both tension and compression (Exhibit 8.1a) and yielding in tension but elastic buckling in compression (Exhibit 8.1b). Strain-hardening effects are included by considering that an element consists of two parallel components, one elastic and one inelastic (Exhibit 8.2).

Large displacement effects may be approximated by including a geometric stiffness in the dynamic phase of the analysis, based on the element axial force under static load.

8.2.2 Beam Element with Degrading Stiffness

The beam element with degrading stiffness is formulated to model reinforced concrete beams, which characteristically exhibit degrading flexural stiffness properties when subjected to cyclic loads. The degrading-stiffness beam may be arbitrarily oriented in the two-dimensional plane. The element possesses flexural and axial stiffness, and elements of variable cross section can be considered by specifying appropriate flexural stiffness coefficients. Flexural shear deformations and the effects of eccentric end connections can be taken into account.

Yielding may take place only in concentrated plastic hinges at the element ends. Strain-hardening and degrading flexural stiffnesses are approximated by assuming that the element consists of a linear elastic beam element with

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nonlinear rotational springs at each end, as shown in Exhibit 8.3. All plastic deformation effects, including the effects of degrading stiffness, are introduced by means of the moment-rotation relationships for the hinge springs.

The moment-rotation relationship for each hinge is an extended version of Takeda's model (53); its behavior is illustrated in Exhibit 8.3. The basic relationship is in the form of a bilinear curve, with an initial stiffness and a subsequent strain-hardening stiffness characteristic of monotonic loading conditions. The degrading stiffness of the hinges is introduced when reversed loading is applied. Exhibit 8.3 also indicates rules that are followed for small-amplitude oscillations. The numbers on the legs of the relationship are yield codes printed by the computer program.

The extensions that have been made to the Takeda model are shown in Exhibit 8.4. These include (1) a reduction of the unloading stiffness by an amount that depends on the largest previous hinge rotation and (2) incorporation of a variable reloading stiffness that is larger than that of the Takeda model and that depends on the past rotation history. To a large degree, these extensions reflect behavior observed in practice. However, some of the features assumed for the extended model, particularly those associated with small-amplitude oscillations, should be selected on the basis of engineering judgment and because of the need to avoid illogical or inconsistent patterns of hinge behavior. Such assumed features are necessary because no test data appear to be available for small-amplitude loading cycles that follow large-amplitude cycles.

The unloading stiffness K_u depends on the maximum hinge rotation and is controlled by the input parameter α , which controls the unloading stiffness by locating the recovery point R_{rec} , as shown in Exhibit 8.4a. It must be non-negative and might typically lie between zero and 0.4. Regardless of the value of α , the unloading slope is never permitted to be less than the reloading slope; otherwise, a hysteresis loop with a negative area would be produced.

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The reloading stiffness K_2 also depends on the maximum hinge rotation and is governed by the input parameter β , as shown in Exhibit 8.4b. The parameter β must be non-negative and might typically lie between zero and 0.6.

The yield moments may be specified to be different at the two element ends and for positive and negative bending. There is no interaction between axial force and bending moment in producing yield.

Static loads applied within any element length may be taken into account by specifying fixed-end force values. The results of separate static load analyses can be incorporated by specifying initial force values. Large displacement effects may be approximated in the dynamic analysis by including simple geometric stiffnesses based on the element axial forces under static load.

In the analysis of reinforced concrete frame structures, difficulties inevitably arise in assigning cross-sectional and material properties. The nonlinear material behavior of the concrete, the non-homogeneous composition of the cross section, and the presence of a varying cross section that results from cracking and steel-area changes combine to make it difficult to assign accurate stiffness values. Considerable experience and experimentation are needed before the element properties can be specified with confidence.

An effective flexural stiffness EI , which might typically be the EI value for the cracked section, must be specified for the linear elastic line element. An axial stiffness EA and an effective shear stiffness GA' also must be specified.

8.2.3 Infill Panel Element

Infill panel elements are included to permit approximate consideration of infill panels of masonry and similar materials. They may also be used to construct inelastic shear beams.

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The element is assumed to have only shear stiffness in the two-dimensional plane. Hence, it provides resistance, through shear deformation, to relative horizontal and/or vertical displacement of the nodes it connects. The relationship between shear stress and shear strain may be inelastic, as shown in Exhibit 8.5. If sudden failure occurs, the forces being resisted by the element immediately before failure will suddenly be transferred to the remaining structure, essentially as a shock loading.

Infill panel elements are usually rectangular. However, provisions are included to permit variations from a rectangular shape without violating equilibrium. Any variations from a rectangular shape should normally be small; otherwise, the assumption that the element has only shear stiffness may be grossly incorrect.

The shear strain in XY coordinates at the element center is the only deformation considered. The element is treated as an isoparametric finite element, and the increment in shear strain, γ_{xy} , is related to the increments of horizontal and vertical displacement at the nodal points by a displacement transformation. The inelastic deformation is the amount of shear strain beyond yield in the elasto-plastic component of the element.

It is important to note that infill panel elements have shear stiffness only. If these elements are used without a surrounding frame (for example, to represent a shear beam), it is essential that the displacement degrees of freedom permitted do not allow modes of deformation for which there is no shear strain at the element center; otherwise, the structure stiffness matrix will be singular.

8.2.4 Beam-Column Element

Beam-column elements may be arbitrarily oriented in the two-dimensional plane. The elements possess flexural and axial stiffness, and elements of variable cross section can be considered by specifying appropriate flexural stiffness coefficients. Flexural shear deformations and the effects of eccentric end connections can be taken into account.

Yielding may occur only in concentrated plastic hinges at the element ends. Strain-hardening is approximated by assuming that the element consists of elastic and elasto-plastic components in parallel, as for the truss element. The hinges in the elasto-plastic component yield under constant moment, but the moment in the elastic component may continue to increase. The yield moments may be specified to be different at the two element ends and for positive and negative bending. The interaction between axial force and moment in producing yield may be taken into account approximately.

Yield interaction surfaces of three types may be specified, as follows:

- Beam type (shape code = 1, Exhibit 8.6a), specified where axial forces are small or are ignored. Yielding is affected by bending moment only.
- Steel column type (shape code = 2, Exhibit 8.6b), intended for use with steel columns.
- Concrete column type (shape code = 3, Exhibit 8.6c), intended for use with concrete columns.

For any combination of axial force and bending moment within a yield surface, the cross section is assumed to be elastic. If the force-moment combination lies on or outside the surface, a plastic hinge is introduced. Combinations outside the yield surface are permitted only temporarily, being compensated for by applying corrective loads in the succeeding time step.

This procedure is not strictly correct because the axial and flexural deformations interact after yield, and it is therefore wrong to assume that only the flexural stiffness changes and that the axial stiffness remains unchanged. However, this procedure is believed to be reasonable for practical analyses of buildings.

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8.3 Simulation of Earthquake Records

To consider a consistent set of acceleration records in the analysis, it is necessary to model the earthquake acceleration history as a random function of time. The type of random function one considers depends entirely on the objective of the analysis. For linear systems, a stationary random process whose power spectral density $S(\omega)$ can be decided from a smooth design spectrum is appropriate for structures having periods shorter than about 2 seconds. The record of such a process can be simulated on a digital computer from the expression

$$\ddot{Y}_{gr}(t) = \sum_1^r [4S(i\Delta\omega)\Delta\omega]^{1/2} \sin(i\Delta\omega t + \phi_{ir}) \quad (8.1)$$

in which $\ddot{Y}_{gr}(t) = r^{th}$ random acceleration record; $\Delta\omega$ is a frequency interval; ϕ_{ir} = a set of mutually independent random variables identically distributed as a random variable with a uniform probability density function over the range from 0 to 2π ; and i and r = integers. For a concise discussion of types of $S(\omega)$, see Ref. 54.

For a nonlinear system analysis, it is necessary to consider the nonstationarity of earthquake records. This can be implemented, in its simplest form, by multiplying the records generated from Eq. 8.1 by a shape function $\zeta(t)$. The general form of this function is shown in Exhibit 8.7. For a discussion on selecting the parameters of this shape to make it correspond to earthquakes of different magnitudes, see Ref. 55.

Random functions have also been developed to consider the influence of fault rupture, focal distance, and direction of transmission path in simulating artificial records. For a discussion of this model, see Ref. 56.

8.4 Comparison of Results Obtained from Approximate Methods and DNA

In Ref. 52, four approximate methods of nonlinear earthquake response prediction have been considered, and a study of the reliability of these methods has

been made. Based on that study, it is suggested that the reserved energy technique may be the most suitable of the four methods studied. The conclusions of this report are limited because extensive results, structural systems, and more importantly, all the response types of interest in nuclear power plant structures have not been considered in the evaluation. For this reason, this section will cite comparative results of the reserved energy method from Ref. 52, the inelastic response spectrum method from Ref. 57, and the substitute structure method from Ref. 48. The aim of this presentation is not to qualify one of these methods as best, but to justify the claim that approximate methods can be developed to be used in practice instead of the time-consuming DNA procedure, provided extensive Monte Carlo studies are carried out.

8.4.1 Reserved Energy Technique (RET) (52)

For a description of this method, see Chapter 5 of Ref. 52. When the method is applied to a four-story, single-bay, reinforced concrete frame using the NRC Regulatory Guide 1.60 spectrum with 1 g and .5 g ground acceleration, the calculated values of the ductility factor for the roof deflection relative to the ground are 4 and 2, respectively, when the damping is assumed to be 7%. When DNA is applied using 5% damping and a ground acceleration history which is consistent with the spectrum, the ductility values are 4.12 and 1.75. When the RET method is applied to a hypothetical auxiliary building and the results are compared to a DNA analysis, it is found that the method needs some modification, including a criterion for redistributing forces based on the yielding of members. Thus, the method requires additional study.

8.4.2 Substitute Structure Method (48)

For a description of this method, see Ref. 48. In this procedure, the flexural stiffness of members is reduced by a damage ratio which is comparable to but not exactly the same as the ductility based on the ratio of maximum to yield rotation. Even if the method were satisfactory, it would be necessary to conduct studies to relate ductility and damage ratios.

In Ref. 48, the method was used to determine design forces for the two series of single-bay reinforced concrete frames. Each series consisted of three-, five- and ten-story frames. In one series, the girders were assumed to be rigid and a column damage ratio of 6 was used. In the other series, flexible girder behavior was assumed and the damage ratio was assumed to be 6 for girders and 1 for columns. Frames were designed using the derived forces from a smooth spectrum and were subjected to analysis by the DNA method for six actual ground motions, scaled to make their spectra agree with the assumed design response spectrum. Obviously, this introduced complications to the problem of comparing results; such complications could be avoided by using artificial records consistent with the assumed spectrum.

The results of the comparison are shown in Exhibit 8.8a for frames with rigid girders and in Exhibit 8.8b for frames with flexible girders. Note that with the exception of the values for the top stories, the values of mean plus standard deviation for six records indicate a uniform distribution of damage ratio with height, as was assumed in the design. This indicates that the method can be calibrated by Monte Carlo studies for approximate analysis purposes. For further details about the two figures, see Ref. 48.

8.4.3 Inelastic Response Spectrum Method (46, 47)

The proposal for using inelastic response spectra with the usual modal analysis for the nonlinear response of MDF systems is given in Ref. 46. The procedure has been investigated in Ref. 57. In this reference, a series of steel frames having three bays and four, ten and eighteen stories was considered. For an assumed elastic response spectrum, the corresponding inelastic spectrum was constructed using the recommendations of Ref. 46. Systems were then designed using the constructed inelastic response spectrum and modal analysis. The designed systems were analyzed using DNA methods and three artificial ground motions whose response spectra are consistent with the assumed design response spectrum. Partial results of this study are summarized in Tables 8.2 and 8.3 for interstory and curvature ductilities, respectively. It can be seen that interstory ductilities can be predicted with fair accuracy, but additional studies are necessary to predict the curvature ductilities.

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8.5 Implication for Seismic Safety Margin Research Program (SSMRP)

Since the probability of radioactive release cannot be realistically predicted without considering nonlinear structural response, the use of dependable methods to obtain nonlinear structural response in the Seismic Safety Margin Research Program is unavoidable. Approximate methods described in Subsection 8.4 are in the developmental stage. Since the reliability of their predictions must presently be questioned, it is more reasonable to use DNA-type analyses in the SSMRP evaluations. With proper idealizations and parametric variations, using DNA for the specific systems selected should be economically possible. However, further development of approximate methods of nonlinear-dynamic-structural analysis is urgently needed in order to consider nonlinear response in design applications.

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9. UNCERTAINTIES IN STRUCTURAL PROPERTIES

9.1 Introduction

Earthquakes are very complex phenomena and the response of structures and components depends not only on the earthquake motion, but also on their own properties. The properties most critical in determining the dynamic response are mass, section moment of inertia, shear and axial area, Young's modulus, the distance between the center of mass and center of stiffness, and the damping in the structure. In the following sections the variability of these structural properties is quantified.

9.2 Variation in Mass

In power plants, the mass comes from the equipment, weight of structural floors, supporting columns and shear walls, block walls, piping and its supports, cables and cable support systems, HVAC ducts and their support systems. In the seismic model of power plants the mass and location of each of these components is properly considered. The floor weights in general account for most of the mass; they also account for most of the uncertainty in mass. In power plants only normal weight concrete is used. The weight would normally vary between 144 and 150 pounds per cubic foot. The tolerances on construction require that the actual thickness of the member be within 1 inch of specified thickness for a member 14 inches or more, and within 1/4 inch of specified thickness for slabs 12 inches or less. A coefficient of variation of 0.05 seems adequate to account for all of the above-mentioned factors. Data on material variability to improve this estimate should be readily available, but its small influence on total response does not justify even this expense.

9.3 Variation in Location of Center of Mass and Center of Stiffness

In power plants the center of mass and the center of stiffness do not generally coincide due to the unsymmetrical nature of the buildings. This eccentricity leads to torsional forces under translatory excitations. In seismic models the mass and stiffness centers are computed based on the actual mass and location of each component. Most of the variability is due to the different

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mass distributions of the floor slabs and shear walls. It is estimated that the standard deviation of eccentricity is likely to be 0.05 of the slab dimension perpendicular to the direction of motion at the floor considered.

9.4 Variation of Young's Modulus of Concrete

In concrete structures the variation of Young's modulus represents a major uncertainty and depends on (a) variability of cylinder strength; (b) the relationship between cylinder strength and in situ strength of concrete; and (c) the relationship between the Young's modulus and cylinder strength.

The actual cylinder strength would differ from the specified strength in two aspects (1) the mean strength of the cylinders would be significantly higher than specified strength; and (2) the variation between different batches of concrete. The mean strength is higher because the concrete mix design requires that the mean value be

$$f'_{cr} = f'_c + 2.326\sigma - 500 \quad (9.1)$$

where

f'_{cr} = average strength to be used as the basis for selecting concrete proportions, in psi

f'_c = specified concrete strength, in psi

σ = standard deviation of the test cylinder strengths.

Equation 9.1 is based on the ACI-318 code commentary, and is for when the standard deviation of cylinder strengths is greater than 500 psi, a condition likely to be realized in nuclear plant construction. When the standard deviation of cylinder strengths is less than 500 psi, refer to the ACI-318 code commentary.

Besides being higher than specified, the mean strength of the different batches of concrete would vary due to the heterogeneous nature of the concrete and its ingredients. Table 9.1 quantifies these variations for two nuclear power plants.

The in situ strength data can be generated using nondestructive testing techniques of ultrasonic pulse velocity and rebound hammer (58). Davis and Martin (59) used these techniques to compute strengths of 47 elements comprising two different strength levels being cast on a non-nuclear construction site. Table 9.2 presents the mean and the standard deviation of the ratio between the 28-day cylinder strength and the member strength (in situ) at 28 days, 3 months and 12 months.

Based on the ACI 318-71 building code (60), the Young's modulus of concrete can be expressed as

$$E = 33 w^{1.5} \sqrt{f'_c} \quad (9.2)$$

where E is the Young's modulus in psi, w is the unit weight in pcf, and f'_c is the compressive strength in psi. The above expression is not exact and is based on least square fit of sample data as developed by Pauw (61). In his data Pauw (61) included both normal weight and lightweight concrete. The factor 33 was actually computed to be 33.6. In nuclear plant construction only normal weight concrete is used. The effect of the variability of the unit weight on E is relatively minor compared to the effect of the variability of f'_c . Hamilton and Hadian (62) used the data corresponding to the region of normal weight concrete and computed an error term e such that

$$E = 33 w^{1.5} \sqrt{f'_c} + e \quad (9.3)$$

where e is the error term with an expected value of 99,000 psi and a variance of $108 \times 10^9 \text{ (psi)}^2$. Based on Eq. 9.3 and data presented in Tables 9.1 and 9.2, the coefficient of variation of E is estimated at 0.12, with the design value of E computed based on Eq. 9.2 equal to a mean minus 0.8 sigma level.

9.5 Variation of Young's Modulus of Steel

The variability of steel properties is less than that for concrete. Based on extensive test data (63, 64) the coefficient of variation for the Young's modulus

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of steel has been determined as 0.6, with the mean value of Young's modulus as 29×10^6 psi.

9.6 Variation of Moment of Inertia

The variation of the moment of inertia of steel members is due to fabrication tolerances. The coefficient of variation is judged to be 0.1, with the mean value corresponding to those computed based on nominal section dimensions. For concrete members the variation of the moment of inertia is due to construction tolerances and also to the cracking of concrete. It is estimated that a coefficient of 0.15 would be adequate for an uncracked section's moment of inertia.

9.7 Variation of Shear and Axial Area

The variation of the shear area of both steel and concrete member is due to construction and fabrication. A coefficient of variation of 0.05 seems appropriate for both concrete and steel. The mean values correspond to those computed based on nominal section dimensions.

9.8 Variations in Damping Values

The energy dissipation phenomenon of structural elements is rather complex, but for mathematical convenience it is generally represented in the form of equivalent viscous damping. It is seen that blanket use of the viscous damping model leads to deviations in results from the true behavior and, therefore, is a source of uncertainty. Quantification of this variation has not been made because of a lack of physical data and the complexity of the resulting analytical techniques. Exhibit 9.1 shows the variation of damping values obtained experimentally (65) for shear wall structure; notice that the distribution is skewed to the left. The mean value of damping is 2.3% with a coefficient of variation of 0.4. These experimental values were obtained at relatively low levels of strain. However, for lack of better data, it may be assumed that at higher strain levels the mean shifts to a higher value and the coefficient of variation is unaltered. Thus, it is suggested that Regulatory Guide 1.61 values be taken as the mean values of damping and a coefficient of variation of 0.4 be considered for this purpose.

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10. STRUCTURAL RESPONSE ANALYSIS USING RANDOM PROPERTIES

10.1 Introduction

The dynamic response of structures and equipment located within these structures is a function of both the input motion and the properties of structures and components. For a number of years the influence of random input loading on the structure has been studied both in the aircraft and the nuclear power plant fields. For the most part, the dynamic systems under consideration were assumed to have known or fixed properties. The analysis was performed using conservatively assigned values of the random structural parameters. The effect has been that the responses are overly conservative in many frequency ranges and not so conservative in others. This leads to an undue penalty in certain frequency ranges without overall improvement in reliability. To overcome this shortcoming, the random structural variables in analysis should be considered and methods of structural analysis should account for these variables. One such method is the Monte Carlo simulation. The uncertainty may arise either from randomness in structural properties, or from arbitrary decisions made in the process of idealization, or from both. When the variabilities are small compared to the mean values, the perturbations can be linearized as analyzed by several authors (66, 67, 69, 72). The modal characteristics of the structure may be then treated as random variables which are linear combinations of the random variables associated with specific structural properties. The Central Limit Theorem suggests that the modal characteristics will tend to have Gaussian distributions, regardless of how the basic random variables are distributed. Probability estimates regarding the complete structure can then be derived from a knowledge of only the first and second order statistics of the elemental structural properties.

10.2 Linear Perturbation of Eigenvalues and Eigenvectors (66)

Eigenvalue problems in structural dynamics arise from the homogeneous

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differential equations of motion which may be written in the matrix form

$$m\ddot{x}(t) + kx(t) = 0 \quad (10.1)$$

in which x and $\ddot{x}$ = vectors of the n coordinate displacements and accelerations, and m and k = $n \times n$ square matrices of constant coefficients which embody the mass and stiffness properties of the structure. Solution of the eigenvalue problem

$$(k - \lambda_j m) \phi_j = 0 \quad (10.2)$$

results in a set of n eigenvalues λ_j , and corresponding eigenvectors ϕ_j , which are interpreted, respectively, as the squared natural frequencies and mode shapes of the structure. When λ_j and ϕ_j are considered to be functions of the vector of random variables r , linear perturbations may be formed by expanding these functions in Taylor's series about the mean values of r , and truncating the series after the second term. In so doing it follows that

$$\lambda_j(r) = \lambda_j(\bar{r}) + \sum_u \frac{\partial \lambda_j}{\partial r_u} (r_u - \bar{r}_u) \quad (10.3a)$$

and

$$\phi_j(r) = \phi_j(\bar{r}) + \sum_u \frac{\partial \phi_j}{\partial r_u} (r_u - \bar{r}_u) \quad (10.3b)$$

From this formulation, the mean values of λ_j and ϕ_j are given by

$$\bar{\lambda}_j(r) = E [\lambda_j(r)] = \lambda_j(\bar{r}) \quad (10.4a)$$

and

$$\bar{\phi}_j(r) = E [\phi_j(r)] = \phi_j(\bar{r}) \quad (10.4b)$$

The variances of λ_j and ϕ_j are then

$$\text{Var} (\lambda_j) = E \left[(\lambda_j - \bar{\lambda}_j)^2 \right] = \sum_u \sum_v \frac{\partial \lambda_j}{\partial r_u} \frac{\partial \lambda_j}{\partial r_v} \text{Cov} (r_u, r_v) \quad (10.5a)$$

and

$$\text{Var} (\phi_j) = E \left[(\phi_j - \bar{\phi}_j)^2 \right] = \sum_u \sum_v \frac{\partial \phi_j}{\partial r_u} \frac{\partial \phi_j}{\partial r_v} \text{Cov} (r_u, r_v) \quad (10.5b)$$

in which $\text{Var} (\lambda_j)$ is a scalar; $\text{Var} (\phi_j)$ is a vector; and $\text{Cov} (r_u, r_v)$ is the covariance between the two random variables r_u and r_v .

As documented in the literature (67, 68, 71), computation of $\partial \lambda_j / \partial r_u$ may be executed in a straightforward manner from

$$\frac{\partial \lambda_j}{\partial r_u} = \frac{\phi_j^T \left(\frac{\partial k}{\partial r_u} - \frac{\lambda_j \partial m}{\partial r_u} \right) \phi_j}{m_j^*} \quad (10.6)$$

The quantity m_j^* is the generalized mass for mode j , i.e., $m_j^* = \phi_j^T m \phi_j$. A similar expression for $\partial \phi_j / \partial r_u$ was also given by McCalley (71) as

$$\frac{\partial \phi_j}{\partial r_u} = \phi \frac{\partial \eta_j}{\partial r_u} \quad (10.7a)$$

in which $\partial \eta_j / \partial r_u$ is a vector whose elements are

$$\frac{\partial \eta_{ij}}{\partial r_u} = \phi_i^T \left[\frac{1 - \delta_{ij}}{m_j^* (\lambda_j - \lambda_i)} \left(\frac{\partial k}{\partial r_u} - \lambda_j \frac{\partial m}{\partial r_u} \right) \right] \phi_j \quad (10.7b)$$

and ϕ is simply the modal matrix having columns ϕ_j . The Kronecker delta, δ_{ij} , is used to define $(1 - \delta_{ij})/(\lambda_j - \lambda_i) = 0$ when $i = j$. The condition $\partial \eta_{jj} / \partial r_u = 0$ is implied if the original and perturbed eigenvectors are normalized so as to have the same magnitude. Fox and Kapoor (68) published the same formulation, except instead of normalizing $\partial \phi_j / \partial r_u$ to give a constant vector length, they

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normalized it to keep the generalized mass a constant unity. The type of normalization imposed supplies the one additional condition required to solve the otherwise undetermined problem.

Some have objected that the complete modal matrix ϕ is required in this formulation to obtain $\partial\phi_j/\partial r_u$. An alternative formulation presented by Fox and Kapoor requires only ϕ_j to compute $\partial\phi_j/\partial r_u$, but this formulation reportedly leads to numerical difficulty. Kiefeling (70) noted this and suggested normalizing $\partial\phi_j/\partial r_u$ so that some ϕ_{lj} will remain constant, implying $\partial\phi_{lj}/\partial r_u = 0$.

10.3 Design Response Based on Monte Carlo Trials

When the structural properties are best described as a random variable, Monte Carlo simulation can be used to compute the design responses. The Monte Carlo simulation method consists of the following steps:

- a. Determine the probability distribution of the variables which influence the structural response.
- b. Based on the probability distribution of these variables, generate a statistically representative set (Monte Carlo trials) of possible structural properties and models.
- c. Analyze the system for each of these Monte Carlo trial sets and generate a statistically representative set of possible maximum system responses.
- d. Determine a design response from this set of responses by computing an 84-95 (or 95-95) confidence response. An 84-95 confidence level means that when the postulated event occurs during the operation of the plant, we can state with 95 percent confidence that there is an 84 percent probability that the response will be less than the design response.

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Two methods are used to determine the design response with the appropriate confidence level from the trial set: the attribute sampling method and the variable sampling method. These two methods are briefly described here.

Attribute Sampling Method. In this method, the design response is determined by enveloping a number of trials. The number of trials, n , depends on the confidence level required and is given by

$$\alpha = 1 - \theta^n \quad (10.8)$$

where θ is the probability of nonexceedance and α is the confidence. Typically, the above equation yields the following relation between the desired confidence and the number of trials to be enveloped:

<u>Confidence Probability Level (Percent)</u>	<u>Number of Trials</u>
84-84	11
84-95	17
95-95	59
99-99	457

Variable Sampling Method. When the distribution of the response can be approximated by a normal distribution, the variable sampling method can be used. Assuming a normal distribution, the design response is given by

$$R = u + t_{\alpha} \sigma \quad (10.9a)$$

where u is the mean and σ is the biased standard deviation of the response R in question. R may be the element forces and moments, restraint reactions, node

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displacements, or accelerations. t_α is a multiplier and depends on the confidence required and the number of trials, and it is given by

$$t_\alpha = \frac{u_\theta + u_\alpha \left[(1 - u_\alpha^2/2f)/n + u_\theta^2/2f \right]^{1/2}}{1 - u_\alpha^2/2f} \quad (10.9b)$$

where

n = number of trials

$f = (n-1)$

θ = nonexceedance probability

α = confidence

u = fractile from normal distribution

Typically, no less than 10 and not more than 30 trials are used. The values of t_α as a function of the number of trials and the desired confidence are as follows:

Number of Trials	t_α for Various Confidence Levels		
	95-95	84-95	84-84
10	2.88	1.90	1.47
15	2.54	1.67	1.36
20	2.38	1.55	1.31

10.4 Example to Determine Accuracy of Linear Perturbation Assumption

To study the accuracy of the linear perturbation assumption for eigenvalues and eigenvectors, the structure shown in Exhibit 10.1 was solved. The model has four degrees of freedom. Table 10.1 presents its mean characteristics. The structural properties considered, i.e., Young's modulus, mass and damping, are described in Table 10.2. The example was solved for the effect of

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a 10% increase in mass using the exact solution and a solution based on the linear approximation as presented in Section 10.1. The results are presented in Table 10.3. It can be observed that the linear perturbation assumption is quite accurate and can be used.

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TABLES

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TABLE 2.1
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FORCES AT CONTAINMENT BASE (R.M.S.) IN FT-KIP/FT AND KIP/FT

Model	M_{θ} FT-K/FT	N_{θ} K/FT	$N_{\theta\theta}$ K/FT	Q_{θ} K/FT	M_{θ} FT-K/FT	N_{θ} K/FT	$M_{\theta\theta}$ FT-K/FT	Q_{θ} K/FT
DYNAX (shell)	36.07	104.9	42.3	4.18	6.57	18.24	1.67	.89
DYNAS (beam)		109.5	45.7					

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TABLE 2.2
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STRESSES AT CONTAINMENT BASE (R.M.S.) IN P.S.I.

Model	M_{θ}	N_{θ}	$N_{\theta\theta}$	Q_{θ}	M_{θ}	N_{θ}	$M_{\theta\theta}$	Q_{θ}
	FT-K/FT	K/FT	K/FT	K/FT	FT-K/FT	K/FT	FT-K/FT	K/FT
DYNAX (shell)	122.68	208.135	83.93	8.29	22.35	36.19	5.68	1.76
DYNAS (beam)		217.26	90.67					

MODAL FREQUENCIES AND PARTICIPATION FACTORS FROM
SHEAR WALL MODEL IN EXHIBIT 2.12 (INCLUDING RPV)

Mode	Frequency C/S	Participation Factor
1	1.5632	- 3.30669
2	3.8590	- .02242
3	4.1123	- .02230
4	4.4948	- 2.34442
5	4.5130	3.34306
6	4.5149	.06275
7	4.8508	-23.79308
8	4.8890	.12571
9	5.0768	29.54609
10	5.5756	- 7.34409
11	4.5778	.56953
12	5.7532	46.57051
13	6.1273	7.67083
14	6.1997	- 2.02299
15	7.2705	- 7.23919

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TABLE 2.4
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MODAL FREQUENCIES AND PARTICIPATION FACTORS FROM
SPACE FRAME MODEL IN EXHIBIT 2.13 (INCLUDING RPV)

Mode	Frequency C/S	Participation Factor
1	1.5625	- 3.30246
2	3.8557	- .02193
3	4.1100	- .02180
4	4.4936	- 2.27366
5	4.5107	- 3.29397
6	4.5127	.08972
7	4.8503	-22.78899
8	4.8855	- .11222
9	5.0976	-31.16286
10	5.5737	6.26250
11	5.5753	- .63352
12	5.7715	46.03931
13	6.1294	- 7.74739
14	6.1967	2.08258
15	7.2665	- 7.30944

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TABLE 2.5
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COMPARISON OF MODAL FREQUENCIES FOR A
150' X 150' X 150' BOX IN LATERAL MOTION

Analytical Model	Frequencies (cps)	
	Mode 1	Mode 2
Shear Beam	5.24	13.2
Finite Element	4.40	13.1

COMPARISON OF SEISMIC MOMENTS IN MAIN GIRDERS
AS PREDICTED BY STATIC AND DYNAMIC METHODS

Type of Girder	Beam	Ratio of Moments (static/dynamic) (%)	
		OBE	SSE
A*	B 16	78.6	79.7
	B 25	100.6	102.9
	B 43	126.3	127.2
	B 44	111.7	113.2
B**	B 22	96.7	95.2
	B 26	138.5	136.9

*Type A - the two ends are directly supported on columns

**Type B - long beams framed into Type A beams.

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TABLE 3.1
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PROPERTIES OF BUILDING MODEL OF EXHIBIT 3.1

Joint	Member	Mass (KIP-SEC ² /FT)	Mass Moment of Inertia (KIP-FT-SEC ² x 10 ⁻³)	Shear Stiffness of Members (KIP/FT ² x 10 ⁻³)
1		4560.0	35800.0	
2	1	275.0	760.0	21200.0
3	2	2045.0	34000.0	3010.0
4	4	1230.0	7800.0	12400.0
5	5	411.0	987.0	2845.0
6	6	555.0	1240.0	1860.0
7	7	298.0	583.0	2080.0
8	8	17.6	32.9	0.47
9		35.5	254.0	
4	9			6350.0
3				
1	3			11640.0

FIXED BASE FREQUENCIES AND PARTICIPATION FACTORS
OF THE BUILDING MODEL IN EXHIBIT 3.1

Mode	Period (sec.)	Frequency (CPS)	Participation Factor
1	.392	2.55	7.04
2	.215	4.64	50.89
3	.113	8.86	44.38
4	.064	15.89	7.61
5	.052	19.14	5.80
6	.044	22.61	7.58
7	.034	29.86	9.90

ASPECT RATIOS FOR THE THREE SOIL MODELS

Model	Aspect Ratio Below Base Slab	Aspect Ratio Away from Base Slab
1	5	10
2	2	2 and 4
3	1.25	2 and 2.6

PREDOMINANT E-W MODAL PERIODS AND MODAL PARTICIPATION
FACTORS FOR EQUIVALENT AND DETAILED MODELS

Detailed Model			Equivalent Model		
Mode	Period (sec.)	Participation Factor	Mode	Period (sec.)	Participation Factor
2	.20	22.1	1	.20	26.0
4	.19	43.2	2	.19	45.9
6	.17	73.4	3	.16	73.9
15	.073	28.9	4	.073	44.9

PREDOMINANT N-S MODAL PERIODS AND MODAL PARTICIPATION
FACTORS FOR EQUIVALENT AND DETAILED MODELS

Detailed Model			Equivalent Model		
Mode	Period (sec.)	Participation Factor	Mode	Period (sec.)	Participation Factor
1	.24	9.7	1	.24	8.9
5	.18	33.1	2	.17	35.8
8	.14	52.1	3	.14	63.0
18	.071	38.8	5	.066	41.1

COMPARISON OF MAXIMUM DISPLACEMENTS (FT)
FOR A 3-D AND A 2-D FIXED BASE STRUCTURE MODEL

Slab No.	3-D		2-D	
	E-W	N-S	E-W	N-S
8	.04569	.02708	.0542	.0253
7	.00850	.00471	.0079	.00679
6	.00843	.00455	.0078	.00627
5	.00804	.00376	.00726	.00532
4	.00602	.00230	.00558	.00339
3	.00423	.00128	.00431	.00203
2	.00253	.00074	.00287	.00119
1	.00105	.00031	.00121	.00044
14	.00089	.00020	.000795	.00025
16	.00052	.00012	.000303	.000012
15	0	0	0	0
0	0	0	0	0

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TABLE 3.7
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INTERPOLATION INTERVAL CONTROL PARAMETERS OF THREE MODELS
(15 Hz Cut-off Frequency, $\Delta f = 0.09766$ Hz)

Frequency	Foundation Input		
Intervals (Hz)	Model 1	Model 2	Model 3
1st	0 ~ 15 (1)	0 ~ 2 (2)	0 ~ 2.5 (2)
2nd		2 ~ 8 (4)	2.5 ~ 4 (4)
3rd		8 ~ 15 (8)	4 ~ 15 (8)

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TABLE 4.1
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EARTHQUAKE RECORDS USED IN COMPARATIVE STUDIES

Record	Component
El Centro (12/30/34)	NS, EW
El Centro (5/18/34)	NS, EW
Olympia (4/13/49)	S80° W, S10° E
Taft (7/21/52)	N21° E, S69° E

RATIO OF RANDOM VIBRATION RESULTS TO
TIME HISTORY FOR SYSTEM A (DAMPING = 0.05)

Response	$R_{0.5}/\text{Ave.}$		$R_{0.1}/\text{2nd Highest}$	
	a (1)	b (2)	a (3)	b (4)
Story Drifts				
Z_1	0.79	1.05	0.85	1.13
Z_2	0.77	1.03	0.79	1.06
Z_3	0.74	0.78	0.76	1.01
Z_4	0.68	0.90	0.73	0.96
Z_5	0.69	0.89	0.72	0.94
Floor Accel.				
$\ddot{x}_1$	0.74	0.99	0.68	0.92
$\ddot{x}_2$	0.72	0.98	0.81	1.11
$\ddot{x}_3$	0.70	0.93	0.74	0.98
$\ddot{x}_4$	0.77	0.98	0.85	1.08
$\ddot{x}_5$	0.68	0.90	0.72	0.94

- (a) Random vibration results for which effect of closely spaced modes is not considered
- (b) Random vibration results for which effect of closely spaced modes is considered

RATIO OF RANDOM VIBRATION RESULTS TO
TIME HISTORY FOR SYSTEM B (DAMPING = 0.05)

Response	$R_{0.5}/\text{Ave.}$		$R_{0.1}/\text{2nd Highest}$	
	a (1)	b (2)	a (3)	b (4)
Story Drifts				
Z_1	1.05	1.05	1.05	1.05
Z_2	1.04	1.05	1.11	1.12
Z_3	1.02	1.02	1.05	1.05
Z_4	0.95	0.95	1.01	1.00
Z_5	0.99	0.96	1.02	1.01
Floor Accel.				
$\ddot{x}_1$	0.98	0.99	0.90	0.91
$\ddot{x}_2$	0.97	1.00	1.00	1.03
$\ddot{x}_3$	0.95	0.95	0.98	0.98
$\ddot{x}_4$	1.06	1.02	1.14	1.10
$\ddot{x}_5$	0.96	0.95	1.02	1.00

(a) Random vibration results for which effect of closeness of modes is not considered

(b) Random vibration results for which effect of closeness of modes is considered

RANDOM VIBRATION AND RESPONSE SPECTRA
RESULTS FOR SYSTEM A ($P_e = 0.50$)*

Response	$\frac{(DSM)_{RS}}{(DSM)_{RV}}$	$\frac{(SRSS)_{RS}}{(DSM)_{RV}}$
Story Drifts		
Z_1	1.02	0.74
Z_2	1.00	0.73
Z_3	0.99	0.73
Z_4	0.98	0.73
Z_5	0.97	0.74
Floor Accel.		
$\ddot{x}_1$	1.09	0.73
$\ddot{x}_2$	0.97	0.71
$\ddot{x}_3$	0.95	0.72
$\ddot{x}_4$	0.95	0.74
$\ddot{x}_5$	0.96	0.74

* P_e is the probability of exceedance in random vibration results

$(DSM)_{RV}$ = Results from Eqs. 4.9 and 4.13

$(DSM)_{RS}$ = Results from Eqs. 4.9 and 4.10

$(SRSS)_{RS}$ = Results from Eqs. 4.8

RANDOM VIBRATION AND RESPONSE SPECTRA
RESULTS FOR SYSTEM B ($P_e = 0.50$)

Response	$\frac{(DSM)_{RS}}{(DSM)_{RV}}$	$\frac{(SRSS)_{RS}}{(DSM)_{RV}}$
Story Drifts		
Z_1	0.99	0.98
Z_2	0.98	0.97
Z_3	0.97	0.97
Z_4	0.96	0.97
Z_5	0.94	0.98
Floor Accel.		
x_1	1.07	0.98
x_2	0.96	0.94
x_3	0.94	0.96
x_4	0.93	0.97
x_5	0.94	0.97

P_e = Probability of exceedance in random vibration results

$(DSM)_{RV}$ = Results from Eqs. 4.9 and 4.13

$(DSM)_{RS}$ = Results from Eqs. 4.9 and 4.10

$(SRSS)_{RS}$ = Results from Eq. 4.8

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TABLE 4.6
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STRUCTURAL PERIODS AND MODAL PARTICIPATION FACTORS
OF THE STRUCTURE MODEL IN EXHIBIT 4.6

Mode	Structural Periods		Participation Factors	
	Period (Sec)	Frequency (C/S)	X Factor	Y Factor
1	.9046	1.1054	- .26111	11.29027
2	.5694	1.7562	.42246	5.48795
3	.5292	1.8898	- 3.54793	- 6.19042
4	.4682	2.1360	11.02987	- 1.20024
5	.4456	2.2444	5.76859	.59620
6	.3728	2.6822	.76864	31.29209
7	.3728	2.6822	- 1.45813	16.49558
8	.3649	2.7402	- 6.13635	- 1.45862
9	.3339	2.9948	- 6.13479	2.00166
10	.3018	3.3136	1.82074	- 8.27259
11	.2538	3.9405	23.18280	- .44132
12	.2538	3.9405	- 1.89492	- 5.39931
13	.2099	4.7649	-27.18319	- .18532
14	.2099	4.7649	- 2.03406	2.47669
15	.1987	5.0324	4.33842	17.39960
16	.1821	5.4901	.70937	-82.82818
17	.1761	5.6793	7.76818	- .27826
18	.1565	6.3916	24.35825	- 2.57997
19	.1505	6.6442	-59.69906	- 2.50941
20	.1476	6.7762	- 4.95915	3.54339
21	.1216	8.2222	-62.61692	.68072
22	.0905	11.0514	- 1.16035	-14.06592
23	.0805	12.4240	- 9.25589	- 7.71012
24	.0773	12.9303	- 3.31290	1.50038
25	.0717	13.9510	5.34781	-14.38019
26	.0673	14.8572	.14678	-49.60102
27	.0556	17.9966	-48.20078	1.09478
28	.0509	19.6558	.08336	-28.76981
29	.0477	20.9864	- 5.76392	- 2.00736
30	.0440	22.7356	.65432	5.62425

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X-DIRECTION: SPECTRUM-CONSISTENT TIME HISTORY

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	5108	0.78	0.89	0.89
07 08 134	24532	0.83	0.97	0.97
08 09 001	18275	0.85	1.00	1.00
09 10 001	17501	0.86	1.00	1.01
10 14 001	5104	0.99	1.10	1.10
00 07 101	6970	0.79	0.94	0.94
Forces for Y-Springs				
00 07 085	1413	1.28	1.29	1.49
07 08 135	5425	1.04	1.11	1.25
08 09 003	4020	1.22	1.23	1.43
09 10 002	3770	1.20	1.21	1.40
10 14 002	638	1.48	1.45	1.58
00 07 104	3736	1.32	1.29	1.50

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X-DIRECTION: EL CENTRO 1940 NS COMP

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	3171	0.87	0.98	0.98
07 08 134	13910	0.90	1.06	1.06
08 09 001	11185	0.90	1.04	1.05
09 10 001	10751	0.90	1.05	1.05
10 14 001	3613	1.02	1.11	1.11
00 07 101	4476	0.79	0.93	0.93
Forces for Y-Springs				
00 07 085	1166	1.07	1.06	1.22
07 08 135	4114	0.88	0.92	1.05
08 09 003	3211	1.05	1.04	1.20
09 10 002	2970	1.03	1.03	1.19
10 14 002	610	1.15	1.12	1.20
00 07 104	3077	1.10	1.07	1.23

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X-DIRECTION: EL CENTRO 1934 NS COMP

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	4423	0.70	0.79	0.79
07 08 134	20929	0.75	0.88	0.88
08 09 001	16998	0.71	0.83	0.83
09 10 001	16224	0.72	0.85	0.85
10 14 001	5184	0.77	0.85	0.85
00 07 101	6276	0.68	0.81	0.81
Forces for Y-Springs				
00 07 085	1086	1.27	1.28	1.48
07 08 135	3747	1.16	1.24	1.39
08 09 003	2918	1.29	1.30	1.50
09 10 002	2724	1.26	1.28	1.48
10 14 002	496	1.46	1.43	1.56
00 07 104	2839	1.32	1.30	1.51

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X-DIRECTION: TAFT 1952 N21E COMP

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	3611	0.74	0.85	0.85
07 08 134	16657	0.87	1.04	1.04
08 09 001	12608	0.87	1.03	1.03
09 10 001	11990	0.89	1.05	1.05
10 14 001	3271	1.03	1.16	1.16
00 07 101	51509	0.77	0.91	0.91
Forces for Y-Springs				
00 07 085	1091	1.09	1.12	1.30
07 08 135	3940	1.00	1.09	1.21
08 09 003	3056	1.07	1.10	1.27
09 10 002	2831	1.06	1.10	1.26
10 14 002	464	1.28	1.26	1.39
00 07 104	2914	1.11	1.11	1.29

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X-DIRECTION: OLIMPIA 1949 S04E COMP

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	4309	0.90	1.00	1.00
07 08 134	19521	0.83	0.98	0.98
08 09 001	14998	0.87	1.02	1.02
09 10 001	14334	0.87	1.02	1.02
10 14 001	4447	1.13	1.22	1.22
00 07 101	5819	0.80	0.93	0.93
Forces for Y-Springs				
00 07 085	2282	0.80	0.78	0.89
07 08 135	6706	0.74	0.76	0.88
08 09 003	6104	0.79	0.78	0.89
09 10 002	5593	0.79	0.78	0.89
10 14 002	1024	1.01	0.98	1.05
00 07 104	6126	0.80	0.78	0.89

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
Y-DIRECTION: SPECTRUM-CONSISTENT TIME HISTORY

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	77	1.84	1.14	2.52
07 08 134	824	1.96	1.25	2.64
08 09 001	560	1.98	1.28	2.65
09 10 001	671	1.59	1.18	2.00
10 14 001	74	2.45	1.54	3.32
00 07 101	1891	1.01	0.97	1.09
Forces for Y-Springs				
00 07 085	7374	0.85	0.86	0.86
07 08 135	8915	1.01	1.01	1.02
08 09 003	5786	1.09	1.09	1.10
09 10 002	5348	1.12	1.11	1.13
10 14 002	1992	1.06	1.07	1.08
00 07 104	8480	0.76	0.80	0.80

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
Y-DIRECTION: EL CENTRO 1940 EW

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	63	1.68	1.10	2.25
07 08 134	908	1.45	0.97	1.90
08 09 001	673	1.34	0.92	1.76
09 10 001	867	1.02	0.81	1.24
10 14 001	80	1.65	1.00	2.25
00 07 102	1508	1.00	0.95	1.07
Forces for Y-Springs				
00 07 085	5416	0.90	0.92	0.92
07 08 135	6922	1.02	1.02	1.03
08 09 003	4927	1.00	1.00	1.01
09 10 002	4689	0.99	0.99	1.00
10 14 002	1712	0.95	0.97	0.97
00 07 104	6242	0.81	0.85	0.85

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
Y-DIRECTION: EL CENTRO 1934 EW

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	80	1.29	0.89	1.71
07 08 134	839	1.49	0.97	1.98
08 09 001	638	1.34	0.89	1.77
09 10 001	888	0.93	0.73	1.14
10 14 001	81	1.64	1.01	2.23
00 07 101	1215	1.22	1.16	1.31
Forces for Y-Springs				
00 07 083	5190	0.94	0.95	0.96
07 08 135	6258	1.11	1.12	1.13
08 09 003	4222	1.16	1.16	1.67
09 10 002	3986	1.17	1.16	1.18
10 14 002	1568	1.06	1.08	1.08
00 07 104	6310	0.80	0.84	0.85

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
Y-DIRECTION: TAFT 1952 S69E

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	61	1.67	1.08	2.26
07 08 134	764	1.60	1.00	2.16
08 09 001	537	1.54	0.98	2.08
09 10 001	658	1.21	0.91	1.51
10 14 001	70	1.79	1.05	2.46
00 07 101	1362	1.09	1.04	1.17
Forces for Y-Springs				
00 07 085	5048	0.96	0.98	0.98
07 08 135	6762	1.04	1.04	1.05
08 09 003	4523	1.08	1.08	1.09
09 10 002	4258	1.08	1.08	1.09
10 14 002	1495	1.07	1.09	1.09
00 07 104	5652	0.88	0.92	0.93

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
Y-DIRECTION: OLIMPIA 1949 S86W

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	67	1.57	0.97	2.13
07 08 134	745	1.57	0.97	2.12
08 09 001	520	1.54	0.98	2.08
09 10 001	626	1.25	0.93	1.58
10 14 001	76	1.63	0.92	2.26
00 07 101	1204	1.03	0.98	1.13
Forces for Y-Springs				
00 07 085	4060	1.00	1.02	1.02
07 08 135	5782	1.01	1.02	1.03
08 09 003	4015	1.02	1.01	1.03
09 10 002	3769	1.02	1.02	1.04
10 14 002	1322	1.03	1.05	1.05
00 07 104	4491	0.94	0.98	1.00

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X- & Y-DIRECTIONS: SPECTRUM-CONSISTENT TIME HISTORY

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	5108	0.78	0.89	0.89
07 08 134	24545	0.83	0.97	0.98
08 09 001	18283	0.85	1.00	1.00
09 10 001	17513	0.86	1.01	1.01
10 14 001	5104	0.99	1.10	1.10
00 07 101	7222			
Forces for Y-Springs				
00 07 085	7508	0.87	0.88	0.89
07 08 135	10435	1.02	1.04	1.09
08 09 003	7045	1.14	1.14	1.22
09 10 002	6543	1.14	1.15	1.22
10 14 002	2092	1.10	1.11	1.13
00 07 104	9266	0.88	0.90	0.95

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X- & Y-DIRECTIONS: EL CENTRO 1940 NS & EW

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	3171	0.87	0.98	0.98
07 08 134	13940	0.90	1.06	1.06
08 09 001	11205	0.09	1.04	1.05
09 10 001	10785	0.90	1.05	1.05
10 14 001	3614	1.02	1.11	1.11
00 07 101	4723	0.82	0.93	0.94
Forces for Y-Springs				
00 07 085	5540	0.91	0.93	0.94
07 08 135	8052	0.99	1.00	1.04
08 09 003	5881	1.02	1.01	1.07
09 10 002	5550	1.00	1.00	1.06
10 14 002	1817	0.98	0.99	1.00
00 07 104	6959	0.87	0.90	0.94

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X- & Y-DIRECTIONS: EL CENTRO 1934 NS & EW

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	4424	0.70	0.79	0.79
07 08 134	20945	0.75	0.88	0.89
08 09 001	17009	0.71	0.83	0.84
09 10 001	16248	0.72	0.84	0.85
10 14 001	5184	0.77	0.85	0.85
00 07 102	6392	0.71	0.82	0.83
Forces for Y-Springs				
00 07 085	5302	0.95	0.97	0.98
07 08 135	7294	1.13	1.15	1.20
08 09 003	5132	1.20	1.20	1.29
09 10 002	4828	1.20	1.20	1.28
10 14 002	1644	1.10	1.12	1.12
00 07 104	6919	0.91	0.94	0.99

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X- & Y-DIRECTIONS: TAFT 1952 N21E & S69E

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	3611	0.74	0.86	0.86
07 08 134	16674	0.87	1.04	1.04
08 09 001	12619	0.87	1.03	1.04
09 10 001	12008	0.89	1.05	1.05
10 14 001	3272	1.04	1.16	1.16
00 07 101	5287	0.79	0.92	0.93
Forces for Y-Springs				
00 07 085	5164	0.97	0.98	1.00
07 08 135	7826	1.03	1.05	1.09
08 09 003	5459	1.08	1.08	1.15
09 10 002	5113	1.07	1.08	1.15
10 14 002	1563	1.09	1.11	1.12
00 07 104	6368	0.94	0.97	1.02

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

FORCES IN SPRINGS BY TIME HISTORY AND
RESPONSE SPECTRUM METHOD - EXCITATION IN
X- & Y-DIRECTIONS: OLIMPIA 1949 S04E & S86W

Spring	Time History	Ratio to Time History of		
		SRSS	Double Sum	Abs. DSM
Forces for X-Springs				
00 07 083	4309	0.92	1.00	1.00
07 08 134	19535	0.83	0.98	0.98
08 09 001	15007	0.87	1.02	1.02
09 10 001	14348	0.87	1.02	1.02
10 14 001	4447	1.13	1.23	1.23
00 07 101	5942	0.81	0.94	0.94
Forces for Y-Springs				
00 07 085	4657	0.96	0.97	0.99
07 08 135	8854	0.87	0.88	0.94
08 09 003	7306	0.87	0.86	0.94
09 10 002	6744	0.87	0.86	0.94
10 14 002	1672	1.02	1.03	1.05
00 07 104	7596	0.85	0.86	0.93

SRSS - Eq. 4.8

Double Sum - Eqs. 4.9 and 4.10

Abs. DSM - Eq. 4.20

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TABLE 4.22
SL-3759
1-15-80

MODAL CHARACTERISTICS OF THE STRUCTURE MODEL IN EXHIBIT 4.16

Structural Periods			Participation Factors	
Mode	Period (Sec)	Frequency (C/S)	Mode	Factor
1	.4680	2.1369	1	-.08072
2	.4085	2.4482	2	122.64186
3	.3313	3.0188	3	-.46403
4	.3070	3.2575	4	-26.19726
5	.2610	3.8318	5	.93635
6	.2093	4.7779	6	-1.03147
7	.2049	4.8812	7	62.40117
8	.1943	5.1465	8	55.75472
9	.1868	5.3524	9	-.58692
10	.1509	6.6249	10	25.26595
11	.1506	6.6409	11	.26120
12	.1471	6.7967	12	2.22177
13	.1426	7.0109	13	-24.80309
14	.1327	7.5380	14	44.71430
15	.0968	10.3316	15	-.74653
16	.0729	13.7105	16	-54.78065
17	.0673	14.8536	17	.27027
18	.0602	16.6150	18	-.04664
19	.0559	17.8888	19	.00619
20	.0511	19.5765	20	30.58854
21	.0450	22.2216	21	11.82846
22	.0439	22.7679	22	.05243
23	.0377	26.5161	23	-.06080
24	.0364	27.4732	24	-17.48697
25	.0359	27.8834	25	-.11014
26	.0330	30.3055	26	.04366
27	.0316	31.6443	27	1.18027
28	.0310	32.2237	28	-.04187
29	.0273	36.6316	29	17.62593

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TABLE 5.1
SL-3759
1-15-80

MATERIAL PROPERTIES FOR DIFFERENT ZONES
OF STRUCTURE IN EXHIBIT 5.2

Zones	Material	Modulus of Elasticity, E psf	Poisson's Ratio,	Weight Density, pcf	Compression Wave Velocity C* ft/sec
1	Crushed Stones	1.1×10^8	0.35	128.8	6643.3
2	Soil above Water Table	1.7×10^7	.4	112.7	3226.
3	Soil below Water Table	4.4×10^7	.4	125.6	4916.
4	Concrete	5.86×10^8	.17	150.	11628.

$$C^* = \sqrt{\frac{1 - \nu}{(1+\nu)(1-2\nu)}} \frac{E}{\rho}$$

EFFECT OF INTEGRATION STEP Δt ON
MAXIMUM ACCELERATION AT CENTER OF BASE SLAB
LOADING - EXHIBIT 5.4, MODEL EXHIBIT 5.3

Δt Sec.	Max. Vertical Acc. ft./sec ²
0.0003	6.31
0.0006	6.31
0.0012	6.30
0.0025	6.41
0.01	7.03

LOAD COMBINATION FOR SAFETY CATEGORY I STRUCTURAL STEEL
(ELASTIC DESIGN)

Load Conditions	Load Factors																	Design Stresses	
	D	L	R _O	R _A	E	E'	W	W _t	T _O	T _A	P _A	H'	M	Y _r	Y _j	Y _m			
Construction	1	1.0	1.0				1.0		1.0									1.33 AISC Allowable	
Test	2	1.0	1.0	1.0					1.0									1.33 AISC Allowable	
Normal	3	1.0	1.0	1.0					1.0									AISC Allowable	
Severe Environmental	4	1.0	1.0	1.0			1.0		1.0									AISC Allowable	
Severe Environmental	5	1.0	1.0	1.0		1.0			1.0									AISC Allowable	
Abnormal	6	1.0	1.0		1.0					1.0	1.0							1.6 AISC Allowable ≤ 0.95 Fy	
Abnormal	7	1.0	1.0	1.0					1.0				1.0					1.6 AISC Allowable ≤ 0.95 Fy	
Extreme Environmental	8	1.0	1.0	1.0		1.0			1.0									1.6 AISC Allowable ≤ 0.95 Fy	
Extreme Environmental	9	1.0	1.0	1.0				1.0	1.0									1.6 AISC Allowable ≤ 0.95 Fy	
Extreme Environmental	10	1.0	1.0	1.0	1.0	1.0			1.0			1.0						1.6 AISC Allowable ≤ 0.95 Fy	
Abnormal/Severe Environmental	11	1.0	1.0		1.0					1.0	1.0			1.0	1.0	1.0		1.6 AISC Allowable ≤ 0.95 Fy	
Abnormal/Extreme Environmental	12	1.0	1.0			1.0				1.0	1.0			1.0	1.0	1.0		1.6 AISC Allowable ≤ 0.95 Fy	

Notes:

- The construction loading combination will be used to check the capacity of structural members for construction loading. The live load to be used will be specified by the contractor to reflect shoring loading, etc. For the construction category, the wind load corresponding to the ten-year recurrence interval will be used.
- If, for any load combination, the effect of any load other than dead load reduces the stresses or reduces the net uplift when checking for the uplift condition, it shall be deleted from the load combination.
- Loads not applicable to a particular system may be deleted.
- For both E and E', the resultant effects (resultant stresses) for both horizontal and vertical earthquake forces shall be determined by combining the individual effects by the square root of the sum of the squares method.
- AISC allowables shall be calculated in accordance with Part I of AISC-69 excluding section 1.56.

LOAD COMBINATION FOR SAFETY CATEGORY I REINFORCED CONCRETE
(ULTIMATE STRENGTH DESIGN)

Load Conditions	Load Factors																Design Stresses
	D	L	R _O	R _a	E	E'	W	W _t	T _O	T _a	P _a	H'	M	Y _r	Y _j	Y _m	
Construction	1	1.1	1.3				1.3		1.3								ACI 318-71
Test	2	1.1	1.3	1.3					1.3								ACI 318-71
Normal	3	1.4	1.7	1.3					1.3								ACI 318-71
Normal	4	1.4	1.7	1.3			1.7		1.3								ACI 318-71
Severe Environmental	5	1.2		1.3			1.7		1.3								ACI 318-71
Severe Environmental	6	1.4	1.7	1.3		1.9			1.3								ACI 318-71
Severe Environmental	7	1.2		1.3		1.9			1.3								ACI 318-71
Abnormal	8	1.0	1.0		1.0					1.0	1.5						ACI 318-71
Abnormal	9	1.0	1.0	1.0					1.0				1.0				ACI 318-71
Extreme Environmental	10	1.0	1.0	1.0		1.0			1.0								ACI 318-71
Extreme Environmental	11	1.0	1.0	1.0				1.0	1.0								ACI 318-71
Extreme Environmental	12	1.0	1.0	1.0					1.0			1.0					ACI 318-71
Abnormal/Severe Environmental	13	1.0	1.0		1.0	1.25				1.0	1.25			1.0	1.0	1.0	ACI 318-71
Abnormal/Extreme Environmental	14	1.0	1.0		1.0		1.0			1.0	1.0			1.0	1.0	1.0	ACI 318-71

Notes:

- The construction loading combination will be used to check the capacity of structural members for construction loading. The live load to be used will be specified by the contractor to reflect shoring loading, etc. For the construction category, the wind load corresponding to the ten-year recurrence interval will be used.
- If, for any load combination, the effect of any load other than dead load reduces the stresses or reduces the net uplift when checking for the uplift condition, it shall be deleted from the load combination.
- Loads not applicable to a particular system may be deleted.
- For both E and E', the resultant effects (resultant stresses) for both horizontal and vertical earthquake forces shall be determined by combining the individual effects by the square root of the sum of the squares method.

LOAD COMBINATIONS AND LOAD FACTORS
CONTAINMENT-SERVICE LOAD AND FACTORS LOAD CONDITIONS

Load Combination Category	Load Factors																		
	D	L	F	P'	P _a	P _o	T _o	T _a	E	E'	W ^{(d)(e)}	W'	R _o	R _a	Y _r	Y _m	Y _j	H'	M
Construction	1	0.75	0.75	1.0			0.75				0.75 ^(a)								
Construction	2	1.0	1.0	1.0			1.0												
Test	3	1.0	1.0	1.0	1.0		1.0												
Normal	4	1.0	1.0	1.0		1.0	1.0						1.0						
Normal	5	1.0	1.0	1.0		1.0	1.0				1.0		1.0						
Severe Environmental	6	1.0	1.0	1.0		1.0	1.0		1.0				1.0						
Abnormal	7	1.0	1.0	1.0	1.5			1.0 ^(b)						1.0					
Abnormal	8	1.0	1.0	1.0			1.0						1.0						1.0
Extreme Environmental	9	1.0	1.0	1.0		1.0	1.0					1.0	1.0						
Extreme Environmental	10	1.0	1.0	1.0		1.0	1.0		1.0				1.0						
Extreme Environmental	11	1.0	1.0	1.0		1.0	1.0						1.0						1.0
Abnormal/Severe Environmental	12	1.0	1.0	1.0	1.25			1.0 ^(b)	1.25					1.0	1.0	1.0	1.0		
Abnormal/Severe Environmental	13	1.0	1.0	1.0	1.25			1.0 ^(b)			1.25			1.0	1.0	1.0	1.0		
Abnormal/Extreme Environmental	14	1.0	1.0	1.0	1.0			1.0 ^(b)	1.0					1.0	1.0	1.0	1.0		

Notes:

- (a) For the construction category, the wind load for a ten-year recurrence interval will be used.
- (b) T_a is based on a temperature corresponding to the factored pressure.
- (c) Loads not applicable to a particular system may be deleted.
- (d) If for any load combination, the effect of any load other than D reduced the load, it will be deleted from the combination.

SUMMARY OF DAMPING VALUES
(FROM REFERENCE 36)

Source of Data	Component or Structure	Type of Excitation	Amplitude or Stress Level	Damping (% of Critical)	
NUCLEAR POWER PLANT COMPONENTS AND STRUCTURES					
San Onofre Nuclear Generating Station Forced Vibration Tests and Responses to Lytle Creek and San Fernando Earthquakes	Steam Generator	Vibration Generators	1.4 mils	1.5, 2.0	
		S. Fernando Earthquake	75 mils 0.23g max. (Transverse)	3.3	
			70 mils 0.18g max. (Radial)	3.0	
			1.8 mils 0.15g max. (Vertical)	0.4	
			Lytle Creek Earthquake	0.03g max. (Transverse)	≈ 5
	Pressurizer	Vibration Generators	1.3 mils	0.9, 1.5	
		Lytle Creek Earthquake	0.04g and 0.08g max.	2 and 9	
	Pump	Vibrations Generators	3.7 mils N-S	1.3	
			6.6 mils E-W	0.8	
			N-S and E-W	0.4	
Indian Point No. 2	Steam Generator		9.5 mils, 10.5 mils (Tangential)	3, 2.2	
			37 mils (Radial)	5	
	Primary Loop Vibration Test Program	Crossover Leg	Shaker (Sine Beat)	0.3 mils	5
		Pump		1.9 mils, 1.8 mils	0.95, 1.3

SUMMARY OF DAMPING VALUES, CONT.

Source of Data	Component or Structure	Type of Excitation	Amplitude or Stress Level	Damping (% of Critical)
NUCLEAR POWER PLANT COMPONENTS AND STRUCTURES (Cont.)				
Laboratory Testing of Steel Pipe and Reinforced Concrete Specimens	Fixed-End Pipe		7.5 to 108% of Yield	0.3 to 14.0
	Simply-Supported Pipe	Laboratory Shaker	27 to 130% of Yield	5 to 20
	Fixed and Simply-Supported Pipe	(Sinusoidal)	0.3 to 3.0g 1000 to 2300 psi Pressure	1.2 to 3.6
	Reinforced Concrete		18 to 100% of UCS	1.9 to 8.9
Tsuruga Nuclear Power Plant Piping Vibration Tests	6 to 16 in. Pipelines	Static Displacement	-	3.2 to 8.6 Ave. = 5.4
	0.75 to 2.5 in. Pipelines		Low	0.18 to 3.4 Ave. = 1.1 (Noninsulated) and 1.7 (Insulated)
Oak Ridge Experimental Gas-Cooled Reactor (EGCR)	Steam Generator	Forced	10 mils E-W 5 mils N-S	1.3 0.9
	Containment Structure (Steel shell with concrete wall lining)	Vibrations	2.5 mils @ 4.65 Hz E-W 2.5 mils @ 4.2 Hz N-S	1.5-2.0 3.0-3.5
	Steam Generator		Torsion	2.0
	Steam Generator	Static	Small Deflections	0.5-1.5
	Steam Line	Displacement and Explosives	Higher Deflections	3 - 7
			1.0 in. N-S	2 - 7
			0.2 in. E-W	3
Laboratory Tests on KEP-L-105 CRDM	CRDM and Support System	Static Displacement	0.125 to 0.375 in.	2.5 to 17.5 Ave. = 8
Carolinas-Virginia Tube Reactor Tests	Concrete Containment with Internals and Components	Vibration Generator	Various	5 to 10

SUMMARY OF DAMPING VALUES, CONT.

Source of Data	Component or Structure	Type of Excitation	Amplitude or Stress Level	Damping (% of Critical)
NON-NUCLEAR TYPE CONCRETE AND STEEL STRUCTURES				
Tests on Las Vegas Buildings	Reinforced Concrete Highrise Buildings	Nuclear Explosion Induced Ground Motion	0.009 g to 0.052 g	1.0 to 11.0 Ave. = 5.6
4-Story Test Structures, AEC Nevada Test Site	Reinforced Concrete Frames with Partitions	Static Displacement and Vibration	0.001 to 0.200 cm.	0.8 to 5.0 Ave. = 2.4
	Reinforced Concrete Frames w/o Partitions	Generator	0.004 to 0.06 cm.	0.6 to 2.4 Ave. = 1.1
Various School and Apartment Type Buildings	Reinforced Concrete Buildings with Concrete Shear Walls, Floors, Partitions	Vibration Generator	0.0001 g to 0.0008 g	Lowest Average for any Building = 1.3 Highest Value = 12.4
Tests of Small Concrete Beams	Prestressed Concrete Beams	Steady-State and Free Vibrations	3 to 11 mils Steady-State Amplitude 30 to 730 mils Free Deflection	0.54 to 2.50 Ave. = 1.15 2.0 to 6.6 Ave. = 4.3
Intake Tower Encino Dam Calif.; Ferribridge Chimney	Round, Reinforced Concrete Water Intake Tower Circular Reinforced Concrete Chimney	Vibration Generator	Max. at Top = 0.043g	2.0, 2.6 and 2.8
Univ. of Calif. Medical Center East Bldg.; Tokyo Kaijo Bldg.; Central Engineering, C.I.T. Campus Bldg.	Steel Frame Buildings with Concrete Floor Slabs	Rocket Firing	0.56 in. max.	0.8 and 1.0
Chimes Tower Structure, New York World's Fair; Talcahuano Steel Plant Stacks	Steel Tower Structure	Vibration Generator	750 mils	2 to 3 (First Mode) 5 to 6 (Second Mode)
	Steel Stacks with Gunite Lining	Static Displacement	Small	0.3 to 0.4

DAMPING VALUES (PERCENT OF CRITICAL DAMPING)

Structure or Component	Operating Basis Earthquake	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
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

**CHECKLIST OF VARIOUS NONLINEAR PROGRAM CAPABILITIES
(AFTER REF. 52)**

Program	Availability	Dimension	Input Ground Motion	Steel Brace Element	Steel Beam- Column Element	Reinforced Concrete Beam- Column Element	Reinforced Concrete Shear Panel Element
DRAIN-2D	Public	2-D	Yes	Yes	Yes	Yes	Yes
DRAIN-TABS	Public	3-D	Yes	Yes	Yes	Yes	Yes
NONSAP	Public	3-D	No	No	No	No	Yes
ADINA	Private	3-D	No	No	Yes	No	Yes
ANSR	Public	3-D	Yes	Yes	No	No	No
SAKE	Public	2-D	Yes	No	No	Yes	Yes
MARC-CDC	Private	3-D	Yes	Yes	Yes	No	No
ANSYS	Private	3-D	Yes	Yes	Yes	No	No

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TABLE 8.1
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RATIOS OF STORY DUCTILITIES TO DESIGN DUCTILITY LEVEL -
3-BAY STEEL FRAMES (AFTER REF. 57)

Design ductility	Motion	Four-Story Frame				10-Story Frame				18-Story Frame			
		Story number				Story number				Story number			
		1	2	3	4	1	4	7	10	1	5	11	15
4	1	1.32	1.27	0.75	0.73	0.98	0.98	0.80	0.93	—	—	—	—
4	2	1.05	1.02	0.95	0.98	1.12	1.12	0.93	0.73	0.85	0.93	0.70	0.53
4	3	0.95	0.92	0.48	0.80	1.45	1.52	0.65	0.87	—	—	—	—
2	2	1.25	1.15	0.95	1.10	1.10	0.92	1.10	—	0.85	0.90	0.95	1.30

TABLE 3. - Ratios of Curvature Ductilities to Design Ductility Level - Design for Earthquake Only

		Four-Story Frame			10-Story Frame		
Design ductility	Motion	Ratio 1	Ratio 2	Ratio 3	Ratio 1	Ratio 2	Ratio 3
(a) Girders							
4	1	1.70	1.01	0.97	1.32	0.98	0.92
4	2	1.50	0.98	0.94	1.51	1.00	0.87
4	3	1.31	0.84	0.81	1.67	1.10	0.99
(b) Columns							
4	1	2.20	1.43	1.11	4.22	1.71	1.14
4	2	1.92	1.28	1.04	3.20	1.62	1.21
4	3	1.49	1.04	0.86	3.80	1.61	1.24

Note: Ratio 1 - Absolute maximum for all members in the frame. Ratio 2 - Average of floor maxima, for all floors. Ratio 3 - Average for all member ends in the frame.

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TABLE 9.1
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VARIATION OF CONCRETE CYLINDER STRENGTHS

	Specified Strength, psi	Actual	Strength	Sample Size
		Mean, psi	Standard deviation, psi	
Plant #1	3500	5428	676	454
	5500	7188	692	246
Plant #2	3500	5209	607	546
	5500	6595	673	331

RATIO OF 28-DAY CYLINDER STRENGTH AND THE
IN SITU CONCRETE STRENGTH

Ratio between cylinder and in situ strength		
Age	Mean Value	Standard Deviation
28 days	1.31	0.188
3 months	1.15	0.125
12 months	1.00	0.098

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TABLE 10.1
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MEAN CHARACTERISTICS OF STRUCTURE
OF EXHIBIT 10.1

Mode	Frequency Hz	Participation Factor
1	7.65	2.86
2	21.78	0.85
3	32.6	0.38

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TABLE 10.2
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1-15-80

PROPERTIES OF STRUCTURE
OF EXHIBIT 10.1

Property	Mean Value
Young's Modulus	3×10^5
Mass	100 ± 10 Percent
Damping	0.05

VARIATION IN DYNAMIC CHARACTERISTIC
DUE TO 10% INCREASE IN MASS

Mode	Frequency		Participation Factors	
	Exact	Perturbation	Exact	Perturbation
1	7.29	7.25	3.00	3.04
2	20.77	20.67	0.893	0.894
3	31.09	30.93	0.399	0.399

EXHIBITS

LEGEND

- FLOOR DIAPHRAGM
- | SHEER WALL
- LUMPED MASS
- FLEXURAL MEMBERS

TYPICAL HORIZONTAL MODEL FOR A BWR PLANT

VERTICAL MODEL

EXHIBIT 2.3
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$$\left[\begin{array}{c} \text{---} \end{array} \right] N$$

COMPOSITE FLOOR

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EXHIBIT 2.4
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1-15-80

MATHEMATICAL MODEL FOR THE COMPOSITE FLOOR

CONTAINMENT MODEL

COMPARISON OF NATURAL FREQUENCIES OF A CHIMNEY STRUCTURE
AS CALCULATED BY TWO TYPES OF MODEL

COMPARISON OF MODAL DISPLACEMENT FOR BEAM ELEMENT
AND SHELL ELEMENT - MODE 1

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EXHIBIT 2.7
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COMPARISON OF MODAL DISPLACEMENT FOR BEAM ELEMENT
AND SHELL ELEMENT - MODE 2

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EXHIBIT 28
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COMPARISON OF MODAL DISPLACMENT FOR BEAM ELEMENT
 AND SHELL ELEMENT - MODE 3

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EXHIBIT 2.9
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COMPARISON OF MODAL DISPLACEMENT FOR BEAM ELEMENT
AND SHELL ELEMENT - MODE 4

COMPARISON OF MODAL DISPLACEMENT FOR BEAM ELEMENT
AND SHELL ELEMENT - MODE 5

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EXHIBIT 2.11
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SLAB-SHEAR WALL MODEL

EQUIVALENT SPACE FRAME MODEL

EXHIBIT 2.14

POWER PLANT CROSS SECTION

EXHIBIT 2.15

ANALYTICAL MODEL OF POWER PLANT

EXHIBIT 2.16

COMPARISON OF FLOOR ACCELERATIONS

EXHIBIT 2.17

COMPARISON OF MAXIMUM SHEAR FORCES

LUMPED MASS ANALYTICAL MODEL OF STRUCTURE

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COUPLED SOIL-STRUCTURE MODEL

MODEL 1 FOR ASPECT RATIO STUDY

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MODEL 2 FOR ASPECT RATIO STUDY

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EXHIBIT 3.4
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MODEL 3 FOR ASPECT RATIO STUDY

COMPARISON OF RESPONSE SPECTRA AT BASE SLAB FOR
THREE ASPECT RATIOS IN SOIL MODEL

COMPARISON OF RESPONSE SPECTRA AT SLAB 7
FOR THREE ASPECT RATIOS IN SOIL MODEL

INTERACTION SPECTRA AT FOUNDATION

COMPARISON OF RESPONSE SPECTRA AT FOUNDATION

INTERACTION SPECTRA AT JOINT 7

COMPARISON OF RESPONSE SPECTRA AT JOINT 7

HORIZONTAL STRUCTURAL MODEL

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EXHIBIT 3.10
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1-15-80

LEGEND

- Mode Shape (Detailed Model)
- Mode Shape (Equivalent Model)

NORMALIZED MODE SHAPE COMPARISON FOR THE DETAILED
AND EQUIVALENT MODEL (E-W MODEL)

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EXHIBIT 3.11
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1-15-80

LEGEND
Mode Shapes:
— Detailed Model
- - - Equivalent Model

NORMALIZED MODE SHAPE COMPARISON FOR THE DETAILED
AND EQUIVALENT MODEL (N-S MODEL)

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EXHIBIT 3.12
SL-3759
1-15-80

COMPARISON OF THE INTERACTION SPECTRA OF THE
2D PLANE STRAIN MODEL & THE 3D DETAILED MODEL

EFFECT OF CUT-OFF FREQUENCY
ON CALCULATED RESPONSE SPECTRA OF SLAB 7

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EXHIBIT 3.14
SL-3759
1-15-80

EFFECT OF CUT-OFF FREQUENCY ON
CALCULATED RESPONSE SPECTRA OF SLAB 15

EFFECT OF FREQUENCY INTERPOLATION INTERVAL
PARAMETER ON CALCULATED RESPONSE SPECTRA

EFFECT OF SIDE SOIL (SLAB 7)

EFFECT OF SIDE SOIL (SLAB 15)

RESPONSE SPECTRA FOR EARTHQUAKE OF EXHIBIT 4.2 AND 2% DAMPING

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EXHIBIT 4.2
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EARTHQUAKE TIME HISTORY

BAND-LIMITED WHITE NOISE FORCING FUNCTION

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EXHIBIT 4.4
SL-3759
1-15-80

FOURIER TRANSFORM OF BAND-LIMITED WHITE NOISE

Mode	MODAL FREQUENCIES	
	Frequency (c/sec.) System A $\frac{mK}{JK} = 1$	System B $\frac{mK}{JK} = 2$
1	1.000	1.000
2	1.056	1.422
3	2.919	2.919
4	3.083	4.153
5	4.602	4.602
6	4.860	5.911
7	5.911	6.546
8	6.244	6.742
9	6.742	8.410
10	7.122	9.592

ANALYTICAL MODEL FOR A SIMPLE TORSIONAL SYSTEM

HORIZONTAL STRUCTURAL MODEL

FLOOR RESPONSE SPECTRA (SLAB 3)

FLOOR RESPONSE SPECTRA (SLAB 7)

SPECTRAL RADII OF NUMERICAL INTEGRATION OPERATORS CASE $\xi = 0.0$

EXHIBIT 5.2

SECTIONAL ELEVATION OF THE AXISYMMETRIC
PLANT STRUCTURE

EXHIBIT 5.3

FINITE ELEMENT MODEL OF PLANT IN EXHIBIT 5.2

TIME HISTORY OF LOADING FUNCTION USED IN THE STUDY OF
INTEGRATION METHODS

EXHIBIT 5.5

TIME HISTORY AT THE CENTRE
OF THE BASE SLAB

EXHIBIT 5.6

RESPONSE SPECTRA AT THE CENTRE
OF THE BASE SLAB

EXHIBIT 5.7

TIME HISTORY AT THE JUNCTION
OF CONTAINMENT AND THE TOP OF
AUX. BUILDING

EXHIBIT 5.8

RESPONSE SPECTRA AT THE JUNCTION
OF CONTAINMENT AND THE TOP OF
AUX. BUILDING

SYSTEM DAMPING VALUES ASSOCIATED WITH PRIMARY
COOLANT LOOP EQUIPMENT
(From Reference 36)

a. YIELD IN TENSION AND COMPRESSION

b. YIELD IN TENSION, BUCKLING IN COMPRESSION

TRUSS ELEMENT BILINEAR BEHAVIOR INCLUDING STRAIN HARDENING
* AFTER REF. 52

a. ELEMENT IDEALIZATION

b. HINGE MOMENT - ROTATION RELATIONSHIP
FOR TAKEDA MODEL OF REINF. CONC. COLS.

REINFORCED CONCRETE BEAM ELEMENT WITH DEGRADING STIFFNESS
AFTER REF. 52

a. UNLOADING STIFFNESS

b. RELOADING STIFFNESS

HINGE MOMENT ROTATION RELATIONSHIP FOR EXTENDED
TAKEDA MODEL OF REINFORCED CONCRETE BEAM-COLUMNS
AFTER REF. 52

STRESS-STRAIN RELATIONSHIP FOR INFILL PANEL
AFTER REF. 52

a. SHAPE CODE = 1

b. SHAPE CODE = 2

SIMILARLY FOR B & D

c. SHAPE CODE = 3

INTENSITY FUNCTION TO CONSIDER NONSTATIONARITY OF
EARTHQUAKES - FOR DETAILS SEE REF. 55

a. Frames with Rigid Girders

b. Frames with Flexible Girders

EXPERIMENTALLY OBTAINED DAMPING VALUES
FROM LAS VEGAS HIGH-RISE BUILDINGS

STRUCTURAL CONSIDERED FOR THE EXAMPLE