NUCLEAR STANDARDS TRANSMITTAL

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<td>March 22, 1985</td>
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- **No later than**
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      - **Com.** (615) 574-6512
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<td>NE F 9-2T</td>
<td>Seismic Requirements for Design of Nuclear Power Plants and Nuclear Test Facilities</td>
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FOREWORD

This standard supersedes the January 1984 issue of NE F 9-2T and incorporates those changes to that issue of the standard that were approved for publication in this revision. These changes are identified by the following marginal notations:

C  Change  
D  Deletion  
N  Addition

Editorial changes that were made during preparation of this revision are not identified.
SEISMIC REQUIREMENTS FOR DESIGN OF NUCLEAR POWER PLANTS AND NUCLEAR TEST FACILITIES

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SEISMIC REQUIREMENTS FOR DESIGN OF NUCLEAR POWER PLANTS AND NUCLEAR TEST FACILITIES

1. INTRODUCTION

1.1 Scope. This standard establishes engineering requirements for the design of nuclear power plants and nuclear test facilities to accommodate the vibratory effects of earthquakes.

Revision 1 of F 9-2T, dated January 1984, incorporated four non-mandatory appendices which provide guidance for analysis in the areas of ground motion definition, pipeline supports and snubbers, soil/structure interaction, and composite modal damping.

Revision 2 of F 9-2T incorporates three new additional non-mandatory appendices that provide guidance in the areas of damping for insulated LMFBR pipe systems, dynamic decoupling in seismic analysis, and seismic analysis fast reactor core internal structures.

The appendices incorporated in the standard are the result of the DOE Seismic Base Technology Program through its Breeder Reactor Seismic Technology Plan.

1.2 Applicability. This standard applies to all systems, structures, and components of nuclear reactors and nuclear test facilities which, in consideration of public health and safety; of cost or time for replacement; or of requirements of local, state, or national codes, are required to be designed for earthquake-induced loads. For nuclear reactor facilities, this standard is applicable to the containment; to all vessels, piping, equipment, and appurtenances that constitute the reactor coolant pressure boundary; to engineered safety features or features whose continued functioning is important to safety; and structures, systems, and components that are required to permit continued operation or that are essential for maintaining support of normal operation.

2. APPLICABLE DOCUMENTS

The following documents are a part of this standard to the extent specified in Secs. 1 through 9. The issue of a document in effect on the date of the invitation to bid, including any amendments or other published changes also in effect, shall apply unless otherwise specified. Where this standard appears to conflict with the requirements of a referenced document, such conflict shall be brought to the attention of the purchaser for resolution.


10 CFR 50 Licensing of Production and Utilization Facilities
2.2 U. S. Nuclear Regulatory Commission (NRC) Regulatory Guides.

Reg. Guide 1.12 Instrumentation for Earthquakes
Reg. Guide 1.29 Seismic Design Classification
Reg. Guide 1.48 Design Limits and Loading Combinations for Seismic Category I Fluid System Components
Reg. Guide 1.57 Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components
Reg. Guide 1.61 Damping Values for Seismic Design of Nuclear Power Plants
Reg. Guide 1.92 Combining Modal Responses and Spatial Components in Seismic Response Analysis
Reg. Guide 1.100 Seismic Qualification of Electric Equipment for Nuclear Power Plants
Reg. Guide 1.122 Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components
Reg. Guide 1.124 Design Limits and Loading Combinations for Class 1 Linear-Type Components Supports

2.3 American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code).

Section III, Division 1, Nuclear Power Plants Components
Section III, Division 2, Concrete Reactor Vessels and Containment
Section VIII, Division 1, Pressure Vessels
Section VIII, Division 2, Pressure Vessels - alternative rules

2.4 American Concrete Institute (ACI) Publications.

ACI 318 Building Code Requirements for Reinforced Concrete
ACI 349 Code Requirements for Nuclear Safety Related Concrete Structures
2.5 Institute of Electrical and Electronics Engineers (IEEE) Standards.

IEEE 344 Recommended Practices for Seismic Qualifications of Class IE Equipment for Nuclear Power Generating Stations

2.6 American Institute of Steel Construction (AISC) Specifications.

Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings.

2.7 Other Documents.

Uniform Building Code (UBC)


3. DEFINITION OF TERMS

3.1 Assembly. An integrated subsystem which performs a specific function. It may contain one or more components and includes the structural supports for the entire subsystem. Examples of assemblies are electrical consoles and instrument panels.

3.2 Seismic Category I. Classification for those structures, systems, and components which are designed to remain functional for the vibratory ground motions of the Safe Shutdown Earthquake (SSE) and which are designed to remain in continuous operation for the vibratory ground motion of the Operating Basis Earthquake (OBE) without undue risk to the health and safety of the public. This classification is consistent with the requirements of 10 CFR 100, App. A, and Reg. Guide 1.29 which place in Category I all structures, systems, and components which are necessary to assure the integrity of the reactor coolant pressure boundary, capability to shut down the reactor and maintain it in a safe shutdown condition, or capability to mitigate the consequences of accidents which could result in potential off-site exposures comparable to the guideline exposures of 10 CFR 100.

3.3 Seismic Category II. Classification for those structures, systems, and components which are designed to remain in continuous operation for the vibratory ground motion of the OBE, fraction of the OBE or other seismic ground motion specified in the project seismic criteria (4.1).

3.4 Seismic Category III. Classification for those structures, systems, and components that are not included in Category I or II but are essential for maintaining support of normal plant operations.
3.5 Direct Integration Method. An analysis method for systems where the responses are obtained directly by integration of the equations of motion for each time increment of loading (3.21).

3.6 Floor Acceleration. The response acceleration of a particular building floor or plant equipment support resulting from given seismic excitation applied to the building.

3.7 Ground Motion. The motion of the earth at the nuclear plant site without the presence of any structures. This is sometimes called free-field motion and may refer to the surface, subsurface, or bedrock motion. However, the point of reference and method of obtaining the specified motion shall be clearly defined for the input motions required in 6.2.2.1 to ensure that the soil-structure interactions are considered as required in 6.2.1.3.

3.8 Interface. The connection points between seismic mathematical models (3.18) and the connection points between two or more physical systems, which may be of the same or different classifications.

3.9 Modal Superposition Method. An analysis procedure for multi-degree-of-freedom systems where the responses are obtained for each normal mode and then combined to arrive at the total response. This method may be used with either response spectrum (modal combination) or time-history procedures (3.15 and 3.21).

3.10 Operating Basis Earthquake (OBE). That earthquake which, considering the regional and local geology and seismology and the specific characteristics of local subsurface material, could reasonably be expected to affect the plant site during the operating life of the plant. It is that earthquake which produces the vibratory ground motion for which those features of the facility necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional.

3.11 Piping Systems. Those pipes, valves, fittings, and other components which perform a specific fluid handling function. Examples are heat transport system piping, off-gas piping, process water piping, radioactive waste piping, external piping from cooling water sources and piping-structure interfaces, etc. Piping systems include appurtenant features, such as hangers and supports.

3.12 Plant Equipment. Systems and components which when combined perform a safety function or service function necessary for reactor operation.

3.13 Primary Supporting Model. The seismic mathematical model (3.18) of structures, plant equipment, or both which is founded on ground and which provides support for one or more supported models.
3.14 **Reactor Coolant Pressure Boundary.** Those coolant-containing components of nuclear facilities, such as vessels (including enclosure systems), piping, pumps, and valves, which are part of the primary coolant system or which are connected to the primary coolant system up to and including the outermost containment isolation valves or safety and relief valves. For boiling and pressurized water-cooled nuclear power reactors, the reactor coolant pressure boundary is that defined in 10 CFR 50, 50.2(V). For Liquid Metal Fast Breeder Reactors (LMFBR) the primary and intermediate loops of the heat transport systems are included as part of the reactor pressure boundary.

3.15 **Response Spectrum.** A plot of the maximum response (pseudo-acceleration, pseudo relative velocity, or relative displacement) versus natural frequency (or period) of a family of idealized single degree-of-freedom damped oscillators subjected to a specified vibratory motion input at their supports.

3.16 **Safe Shutdown Earthquake (SSE).** That earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional.

3.17 **Secondary Containment.** The structure or system that houses the primary coolant boundary or portions thereof and which serves as a secondary leakage-limiting barrier in the event of an accident which may release radioactive material from the primary coolant boundary.

3.18 **Seismic Mathematical Model.** The idealization of structures, plant equipment, or both as a dynamic system and in a mathematical form suitable for detailed dynamic analyses.

3.19 **Structures, Systems, and Components.**

a. **Structures.** Buildings, stacks, towers, and similar items, including their foundations. Structural supports for systems and components are included.

b. **Systems.** Mechanical, electrical, and fluid systems, such as the reactor coolant system; radioactive waste treatment, handling, and disposal systems; and plant protection systems.

c. **Components.** Specific items within a system, such as pumps and valves in a piping system.

d. **Reactor Core.** That part of the reactor in which nearly all of the nuclear fission occurs. For LMFBR's the coolant, arrays of fuel, blanket, control, and reflector assemblies are considered part of the reactor core.
3.20 **Supported Model.** The seismic mathematical model of structures, plant equipment, or both which has its means of support furnished by the primary supporting model or other supported models.

3.21 **Time-History Procedures.** Those dynamic analysis methods having input vibratory motion which is a continuous or stepwise continuous function of time. Time-history methods may be used with either mode superposition or direct integration analysis methods to calculate the multi-degree-of-freedom response of the seismic mathematical mode involved (3.5 and 3.9).

4. **CLASSIFICATION REQUIREMENTS FOR DESIGN**

4.1 **Project Seismic Criteria.** A document shall be prepared, which shall contain specific seismic design criteria applicable to a given nuclear power plant or test facility and site. These criteria shall provide the basis for the seismic analyses and design of the given nuclear facility in accordance with this standard. The content of the project seismic criteria shall include, but not be limited to, the following:

1. Classification and listing of structures, systems, and components by category as set forth in the individual system design description (SDD). The category shall be as described in 4.2, and should be noted that the SDDs are the controlling documents. This list is provided for information only.

2. Applicable governing seismic input for the foundation, as defined by 3.10, 3.16, and Sec. 5 and required by 6.2.2.1.

3. Applicable site and geological features which may influence seismic design as required in 6.2.1.3.

4. A requirement for a design specification which provides the information needed for design and procurement of appropriate structures, systems, or components and which shall include operating conditions, loading combinations, respective categories, and selected methods of seismic analysis and design; mathematical models; input time histories or response criteria; and allowable stress, strain, and deformation criteria.

5. Designation of the organizations which will be responsible during the construction period for ensuring that the installation of structures, equipment, and supports is in accord with the data and assumptions of arrangement and configuration used in the design analyses.

4.2 **Category.** All structures, systems, and components which are to be seismically designed shall be classified as either Seismic
Category I, II, or III.* The selection of classification shall satisfy 3.2, 3.3, 3.4, and the requirements of Reg. Guide 1.29.

5. SEISMIC LOAD APPLICATIONS

Structures, systems, and components for a nuclear facility shall be designed to accommodate seismic loadings which may be produced by a range of earthquake intensities. Such loadings may be in lieu of or in addition to other postulated conditions and are dependent upon the category classification which establishes the required level of safety and functional assurance. Horizontal and vertical vibratory ground motion shall be considered. Unless otherwise provided in the project seismic criteria, the specified horizontal and vertical design response spectra shall conform to the requirements of Reg. Guide 1.60.

Recommended non-mandatory guideline procedures that can be used to define vibratory ground motion are given in App. XI. Design requirements and combinations of seismic loading with other loadings are specified in Sec. 7. The general descriptions of two reference seismic loadings are as follows.

5.1 Safe Shutdown Earthquake (SSE). The SSE shall be determined in accordance with 10 CFR 100, App. A. The magnitude, form of loading, duration, and other related earthquake load data shall be set forth in the project seismic criteria.

5.2 Operating Basis Earthquake (OBE). For nuclear reactor facilities, an OBE shall be selected in accordance with 10 CFR 100, App. A. For other facilities, the selection of the OBE is the responsibility of the facility owner or his designated agent. The magnitude, form of loading, duration, and other related earthquake load data shall be set forth in the project seismic criteria.

If vibratory ground motion which exceeds that of the OBE occurs (as determined by instrumentation meeting the requirements of Reg. Guide 1.12), shutdown of the plant is required. Prior to resuming operation, it shall be demonstrated that no functional damage has occurred to those features necessary for continued operation without undue risk to the health and safety of the public.

6. ANALYTICAL REQUIREMENTS FOR VIBRATORY MOTION

The effects on Seismic Category I and II resulting from the OBE and SSE seismic environment shall be determined by any of the seismic

*The inclusion of a Category II is optional. Its purpose is to provide a framework whereby equipment not easily replaced can be identified and protected against reasonably expected earthquake loads so as to protect plant investment and ensure continuance of priority programs in a given facility. Alternately, such items may be selectively included in Category I with safety-related items, and all remaining structures, systems, and components may be included as Category III.
analysis methods described herein. Appendix X7, "Seismic Analysis of Fast Reactor Core Structures," presents a series of recommendations associated with the analysis of LMFBR cores.

This standard does not exclude nor specify any of the available methods of seismic analysis. The following minimum requirements shall be met.

6.1 Simplified Analysis. Where analytical methods are used to ensure that the required function for Category I and II items are maintained, an equivalent static analysis may be used in lieu of a dynamic analysis method provided that it can be shown that the equivalent static analysis provides adequate conservatism and that the use of simplified analysis procedures is not specifically prohibited in the project seismic criteria. The justification for the use of simplified analyses shall show that the applicable site, structures, and plant equipment characteristics related to the specified earthquake were taken into account. Justification of the simplified analysis approach may include the analysis of representative examples of a given type of system or component, if it can be demonstrated that such analysis conservatively describes the behavior of such systems and components. The relative displacements occurring between structures and plant equipment and their bases or means of support and the loads produced by the displacements shall be taken into account. The responses computed in the separate analyses which account for inertial response and relative base displacement shall be combined absolutely.

6.1.1 Equivalent Static Analysis. Where equivalent static analysis is used, as justified in 6.1, the equivalent static forces for structures shall satisfy as a minimum, the requirements of the UBC. The determination of equivalent static forces for supported systems and components shall include consideration of the magnification of the supporting structure motion at the support point. This magnification will depend on the natural frequencies and damping of the supporting structure and the natural frequencies and damping of the supported systems or components. The method of determining the magnification factors shall be documented and justified. The following options may be applicable for determining the equivalent static forces for supported equipment and components.

1. Where response spectra are available for the support point, the equivalent static force for supported system or component may be taken as given in Eq. 1:

\[ F = 1.5 M_e A_s , \]  

where

\[ F = \text{equivalent static force distributed proportionally to the mass of the system or component.} \]
\( M_e = \) effective mass of system or component including liquid contents,

\( A_g = \) maximum acceleration from support point response spectrum for the appropriate system or component damping value.

2. Where response spectra are not available for the support point, the equivalent static force for the system or component may be taken as given in Eq. 2:

\[
F = 12.0 \, M_e A_g ,
\]

where

\( A_g = \) acceleration from the response spectrum for the component damping value.

6.2 Dynamic Analysis. The dynamic analysis method used shall be the modal superposition method, direct integration method, or any other analysis method which can be justified. If modal superposition methods are used, all significant modes shall be included in the analysis. If the response spectrum method is used, the modal combinations shall be at least as conservative as the combinations given in Reg. Guide 1.92. If alternate methods are used, these shall be documented and justified with the design analysis. The dynamic analysis shall include the following considerations.

6.7.1 Mathematical Modeling. Structures and plant equipment of physically connected systems may be idealized as a combined single mathematical model or, subject to the provisions of 6.2.3, as subdivided systems of one or more smaller mathematical models. In the latter case there shall be at least one primary supporting model. In all cases the dynamic effects of coupling at interfaces between the models shall be included in the dynamic analyses as specified in 6.2.3. Mathematical models of structures and plant equipment composed of different classification categories shall be modeled in a form which will represent those dynamic effects resulting from the different loadings which may be required for each category. In addition, the effects of different stress criteria and deformation limits between the classification categories shall be included. Dynamic loads and displacements at these interfaces of Category I and Category II systems which result from such above mentioned differences shall be accounted for in analyses of all models. Inelastic effects may be incorporated into the mathematical model provided that the results of the analysis are compatible with 7.6.1.1. Because of the complexities and large number of variables within nuclear systems, the seismic mathematical models should be varied where possible to give consideration to the effect of changes in the mathematical model on the computed response. Significant nonlinearities, such as gaps or clearances between components shall be considered and included in the mathematical model. In this case, a nonlinear time-history analysis is required which considers the impact forces generated at the gap locations.
6.2.1.1 Degrees of Freedom. Structures and plant equipment may be modeled as lumped spring-mass systems or by other techniques, such as the finite element methods, which adequately represent the mass and stiffness properties. A sufficient number of degrees of freedom shall be used so that the seismic response is adequately described. Translational, torsional, and rocking degrees of freedom shall be considered at foundations or points of support.

6.2.1.2 Damping. For mathematical convenience, damping is assumed to be viscous in nature and it is expressed as a percent of critical damping. For structures which are assumed to be homogeneous, the damping is assumed to be proportional, i.e., coupling between the modes of vibration does not exist. Damping coefficients for these structures are listed in Table 1.

Table 1. Equivalent Proportional Viscous Damping Coefficients.

<table>
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<tr>
<th>Structural System</th>
<th>Percent of Critical Damping</th>
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<tr>
<td></td>
<td>OBE</td>
</tr>
<tr>
<td>Reinforced concrete structures and foundations</td>
<td>4.0</td>
</tr>
<tr>
<td>Prestressed concrete structures</td>
<td>2.0</td>
</tr>
<tr>
<td>Steel structures</td>
<td></td>
</tr>
<tr>
<td>Welded steel structures</td>
<td>2.0</td>
</tr>
<tr>
<td>Bolted or riveted steel structures</td>
<td>4.0</td>
</tr>
<tr>
<td>Piping systems (welded)</td>
<td></td>
</tr>
<tr>
<td>Small diameter</td>
<td>1.0</td>
</tr>
<tr>
<td>Large diameter [&gt;12 in. (0.3048 m) nominal diameter]</td>
<td>2.0</td>
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BIn the dynamic analysis of active components as defined in Reg. Guide 1.48, these values should also be used for SSE.

Appendix X6 provides the basis for the following damping coefficients for small diameter heavily insulated LMFBR piping systems:

<table>
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<th>Percent Critical Damping</th>
<th>Pipe Size</th>
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<tr>
<td>8.0</td>
<td>1 in. or smaller</td>
</tr>
<tr>
<td>5.0</td>
<td>8 in.</td>
</tr>
<tr>
<td>Linear interpolation</td>
<td>&gt;1 in. &lt;8 in.</td>
</tr>
</tbody>
</table>
These damping values are applicable to small diameter piping, with ratios of insulation weight to total piping system weight specified in Appendix X6. For larger diameter piping or for less heavily insulated piping, Appendix X6 presents alternatives to the damping ratios given by Reg. Guide 1.61.

For systems or structures such as a reactor coolant loop-containment building-soil foundation the damping mechanism for one subsystem may vary considerably from the others. The term "composite modal damping" addresses the treatment of non-uniform or subregional modal damping. Appendix X4 discusses the methods used in the evaluation of the composite modal damping properties.

6.2.1.3 Soil/Structure Interaction. The effects of the soil on the structural response of the nuclear facility shall be considered. Several factors such as the extent of the embedment, depth of soil over rock and the layering of the soil strata, dictate the particular modeling approach to the soil/structure interaction problem.

Appendix X3 provides recommended nonmandatory guidelines that are considered when performing soil/structure interaction analysis for nuclear power plants structures.

6.2.1.4 Relative Displacement Effects. The effects of relative displacements of support points or foundations of all supported mathematical models shall be considered in the overall analyses. These relative displacements shall be imposed on the seismic mathematical model as a separate static displacement unless the appropriate time-history forcing function at each support point is used in the dynamic analysis, such that effects of these displacements are inherently included in the analysis. It should be noted that, in general, for input motions other than time-histories at each support point, the displacement responses may or may not be in phase. For the case which requires separate analysis for relative support displacements, any conventional analysis method may be employed to determine resulting forces or stresses which shall then be absolutely superimposed on those forces or stresses due to seismic-induced inertial loadings.

6.2.1.5 Torsional Effects. Parameters, such as nonsymmetric geometry, mass, and stiffness, that will produce torsional effects shall be included in the seismic mathematical model for any system.

6.2.1.6 Hydrodynamic Effects. The most important hydrodynamic effects in seismic analysis include dynamic fluid forces, vibration of submerged structures and components, and wave transmission. The effect of added or virtual mass for fluid filled and submerged components shall be included in the analysis. Also the added mass and wave effects shall be included for components containing fluids with a free surface. These and any other significant hydrodynamic effects shall be accounted for in the seismic mathematical model.
6.2.1.7 Discontinuities. The nonlinearities occurring at discontinuities, such as gaps or clearances, require special consideration. Impact forces may occur at these discontinuities. The location and magnitude of such impact forces shall be determined by analysis and justification for appropriate coefficients or restitution shall be presented.

6.2.1.8 Overturning Effects. Overturning moments due to seismic excitation shall be determined for all Category I and II structures supported on the ground and shall be considered in the overall analysis.

6.2.2 Analytical Load Characteristics.

6.2.2.1 Design Ground Motion. Appendix A to 10 CFR 100 states that the vibratory ground motion associated with the most severe ground motion at the site should be designated the Safe Shutdown Earthquake. This earthquake is defined by the response spectra at the elevations of the foundation of the nuclear power plant structure. The specific procedures for determining the design basis for vibratory ground motion are given in App. XI, "Definition of Design Earthquake Ground Motion."

In lieu of detailed evaluation of the appropriate site dependent response spectra, the design response spectra of Reg. Guide 1.60 may be used.

The response spectrum for large structures may be adjusted in some cases to reflect the filtering effect of large building foundations due to the finite transit time of seismic waves. If adjustments are made for this effect, justification of the adjustments shall be presented with the design analysis and appropriate allowance for torsion and tilting shall be included in the analysis.

6.2.2.1.1 Supporting Systems. The analysis of Category I primary supporting models, as defined in 3.13, may be required by 6.2.1 and 6.2.3 to generate input information for the subsequent analysis of supported systems. Where this applies, time-history input motion, recorded or artificially produced, may be used to represent the vibratory motion of both the SSE and OBE. If one time-history analysis is used, the scale factor for the time-history ground motion shall be selected such that the response spectrum for this input record envelopes the specified response spectrum as defined in the Standard Review Plan section 3.7.1.11.1.1.1.b, the floor response spectra must be widened as specified in Reg. Guide 1.122. Multiple time-history analyses may be used to permit smaller scale factors and still preclude errors which may result from variations in frequency content. If multiple time-history analyses are used, the scale factor for the ground motion of each time-history record shall be selected such that the average of the response spectrum values at each frequency in the range of interest exceeds the specified response spectrum.
6.2.2.1.2 Supported Systems. The input for supported structures, systems, and components shall be determined from the floor or support point accelerations computed by the time-history analysis of the supporting structure. If multiple time-history analyses of the supporting structure are performed, the floor or support point motion shall be characterized by one of the following options.

a. A response spectrum may be constructed for the floor or support point and used in the analysis of the supported structures, systems, and components. This response spectrum shall equal or exceed at all frequencies the floor or support point response spectra determined from all of the time-history analyses.

b. A single time-history for the floor or support point may be synthesized for use in the analysis of the supported structures, systems, and components. The response spectrum for this synthesized time-history shall not significantly depart from the response spectrum as determined in 6.2.2.1.a at each frequency in the range of interest.

c. The time-history floor or support point motions from each of the supporting structure analyses may be used for multiple time-history analyses of the supported structures, systems, and components. Each of these time-history analyses shall satisfy the design requirements of Sec. 7.

Category I supported systems, Category I systems represented as a single mathematical model supported on the ground, and Category II systems may be analyzed using response spectra inputs derived from ground motion of applicable support motions. For these cases, the procedures for combining the modal responses shall be set forth in the project specification and shall be consistent with Reg. Guide 1.92. In addition, the maximum displacement response from analyses of primary supporting models shall be used to derive the relative support displacements for applicable supported models. These relative displacements shall be considered as prescribed in 6.2.1.4.

6.2.2.2 Directional Combination Effects. Seismic analyses shall include vertical axis excitation concurrent with horizontal excitation along any axis in the horizontal plane. For design purposes, the most adverse combination of a horizontal axis motion combined with vertical motion shall be considered. Horizontal input motion may be applied coincident and orthogonal with one of the principal axis of the model; effects of torsion as prescribed in 6.2.1.5 shall be included.

6.2.2.3 Frequency Limit Effects. Horizontal and vertical dynamic amplification of motion shall be considered in the analysis. This is given by the applicable response spectra. Dynamic amplification is applicable to structural systems whose fundamental frequency is less than the frequency value at the start of the zero period acceleration on the response spectrum (about 33 Hz). Those structures, systems, and components whose fundamental frequencies fall in the zero period.
acceleration region of the response spectrum in any direction may be generally assumed to be rigid in that direction and designed for the maximum accelerations of their support (zero period acceleration) in the corresponding direction. For those cases where the natural frequencies of the structural system are very small, displacement may be the limiting factor and shall be considered in the analysis.

6.2.3 System Coupling and Resonance. The natural frequencies of all primary supporting models and the natural frequencies of the related supported models shall be determined. This determination shall consider all interaction effects at interfaces such as piping reactions and structural restraints.

6.2.3.1 Mathematical Uncoupling. When a large system is subdivided into smaller systems with a mathematical model for each subdivided system, the interfaces between such mathematical models serve considerations shall be required. Coupling between the mathematical models always exists; however, the mathematical uncoupling of such connected models is permitted when justified.

If $R_m$ and $R_f$ are defined as

$$R_m = \frac{\text{Total Mass of the Supported System}}{\text{Total Mass of the Supporting System}}$$

$$R_f = \frac{\text{Fundamental Frequency of the Supported System}}{\text{Frequency of the Supporting System}}$$

then the supporting and supported systems can be analyzed as follows:

1) If $R_m \leq 0.01$, decoupling can be done for any value of $R_f$.

2) If $0.01 \leq R_m \leq 0.1$, decoupling can be done if $R_f \leq 0.8$ or if $R_f \geq 1.25$.

3) If $R_m > 0.1$, an approximate model of the supported system model should be included in the supporting system model.

The fundamental frequency of the supported system is calculated with the interface point fixed and the frequency of the supporting system is calculated with the mass of the supported system removed.

The above specified criterion is based on mass and frequency relation of the subcomponents, this criterion is by necessity very general in nature, and the application to complex systems requires further explanation. Appendix X5 provides a quantitative evaluation of this criterion and provides guidelines for their application to system configurations found in nuclear plants.
6.2.3.2 Interaction Effects of Subdivided Systems. The analysis of mathematical models of subdivided systems as described in 6.2.3.1 will result in reactive forces at the model interfaces. These reactive forces are herein termed interaction effects. All such effects shall be included in the design requirements of 7.3.

6.2.4 Creep-Fatigue Damage and Progressive Deformation from Cyclic Loading. All Category I structures and plant equipment shall treat earthquake loads as cyclic loadings about a reference plane which may have been displaced as a result of deformations occurring due to structural yielding. A number of loading cycles for determining cumulative fatigue damage shall be determined using the natural frequencies of the mathematical model and the number, magnitude and duration of earthquakes specified in the project seismic criteria. This number of loading cycles shall be used in the evaluation of creep-fatigue damage and progressive cyclic inelastic deformation as required by the applicable codes. Where it can be justified, the number of loading cycles may be determined from probabilistic analyses which define the earthquakes in terms of a statistical distribution of magnitudes up to the SSE.

7. DESIGN REQUIREMENTS TO ACCOMMODATE VIBRATORY MOTION

7.1 General. All Category I structures, systems, and components shall be designed to withstand the effects of the SSE and remain functional (will not exceed the stress, deformation, and damage limits specified as appropriate for the SSE). All structures, systems, and components necessary for continued operation without undue risk to the health and safety of the public (Category I) shall be designed to withstand the effects of the OBE and remain functional (will not exceed the stress, deformation, and damage limits specified herein as appropriate for the OBE). Applicable concurrent functional and accident induced loads shall be considered as prescribed in 7.4. Category II structures, systems, and components shall be designed to withstand the effects of the OBE, any stated fraction of the OBE, or other seismic ground motion as required in the project seismic criteria. Category III includes all structures, systems, and components not included in Categories I and II, and are designed to withstand the ground motion specified in the project seismic criteria for this category.

7.2 Loading Components. The separate components of loading which are to be used in the loading combinations of 7.4, are in accordance with the ASME Section III, Nuclear Power Plant Components Code, and defined as follows.

7.2.1 Dead. Dead loads shall include all static weight which contributes to stresses and deformations. This may include structures, plant equipment, soil or hydrostatic loads, vessels and piping (including liquid content), and insulation.
7.2.2 Live. Live loads shall include nominal floor live loads where appropriate, impact loads from operating cranes or other machinery and movable or semipermanent loads such as snow.

7.2.3 Operating. Operating loads are those caused by operating pressure (internal and external); operating temperature and thermal effects, including shrinkage and creep; and other mechanical loads as specified in the project seismic criteria for the applicable operating conditions. Temperature effects and forces due to thermal expansion of piping are included.

7.2.4 Design. Design loads are those resulting from the design pressure, temperature, and mechanical loads as specified in the SDD or component specification and appropriate design specification for structures and plant equipment. The specified mechanical loads may include the OBE loading.

7.2.5 Earthquake Loads (SSE and OBE). Earthquake loads are those determined by the analyses set forth in Sec. 6. The SSE and OBE are defined in Sec. 5.

7.2.6 Accident Loads. Accident loads are those resulting from pressure, temperature, impact forces and reactions caused by jet impingement or fluid discharges and similar loadings which occur as a result of postulated accidents as specified in the project seismic criteria.

7.3 Design and Operating Conditions. The separate conditions which include the simultaneous occurrence of the various loading components for use in the loading combination requirements and design criteria of 7.4 shall be specified in the project seismic criteria.

7.3.1 Design Conditions. The design conditions include the design loads of 7.2.4 in accordance with the ASME Code, Sec. III, Div. 1, and shall be specified.

7.3.2 Operating Conditions. The operating conditions are classed as service limit levels A, B, C, and D in accordance with the ASME Code, Sec. III, Div. 1. In addition, the operating conditions also include general classifications conforming to the ASME Code, Sec. III, Div. 2 for Concrete Reactor Vessels and Containments. These classifications are termed service load conditions and factored load conditions. The intent herein is to parallel existing codes where possible when specifying loading and design criteria.

7.3.3 Non-Code Items. The design and operating conditions for items not covered by existing codes shall be specified.

7.4 Coincident Loading Combinations and Design Criteria. The minimum requirements for combining loads involving seismic along with their associated allowable stress and deformation criteria are set forth
in this section. Additional loading combinations for structures and plant equipment may be set forth in the project seismic criteria. These minimum load combination requirements are separately grouped for structures and plant equipment. Structures shall be in accordance with 7.4.1, and plant equipment shall be in accordance with 7.4.2. All loading combinations involving seismic shall be set forth in the project seismic criteria.

7.4.1 Structures. Category I and Category II structures shall be designed to accommodate postulated loading combinations which include seismic loading.

7.4.1.1 Non-Pressure Retaining Steel Structures. Category I steel structures shall satisfy the following Service Loads and Factored Load Conditions:

Service Load Condition: Dead + Live + Operating + OBE
Factored Load Condition: Dead + Live + Operating + SSE

Category II steel structures shall satisfy the Service Load Condition only.

Category I and II steel structures or portions thereof shall meet the design requirements and allowable stress and deformation limits specified by the AISC Specification with the following qualifications. Allowable stress limits for the Service Load Condition shall not include the one-third increase which is normally permitted by the AISC Specification for dynamic loading (seismic).

Category III steel structures shall satisfy the loading combinations and design requirements of the UBC in addition to the AISC Specification. Where design criteria, including stress and deformation limits are not covered by UBC or AISC Specifications, such criteria shall be set forth in the project seismic criteria.

Component supports are excluded from the requirements of this section and shall be designed as required by the ASME Code, Sec. III, Div. 1, Subsection NF.

Allowable stresses for the Factored Load Conditions are specified in the appropriate code or specific design criteria.

7.4.1.2 Non-Pressure Retaining Concrete Structures. Category I concrete structures shall satisfy the Service Load and Factored Load Conditions. Category II structures shall satisfy the Service Load Conditions.
The Service Load Condition shall include Eqs. 2, 7, and 10 of Sec. 9.3 of the ACI-349 Code. The Factored Load Condition shall include Eqs. 4 and 8 of Sec. 9.3 of the ACI-349 Code.

The design requirements and stress and deformation limits for Category I and Category II concrete structures shall be governed by ACI-349. Category III concrete structures shall satisfy the load combinations and design requirements of the Uniform Building Code in addition to ACI-318. Where design criteria, including stress and deformation limits are not covered by the UBC or ACI-318, such criteria shall be set forth in the project seismic criteria.

7.4.2 Systems and Components.

7.4.2.1 Systems and Components Under Jurisdiction of the ASME Code. The project seismic criteria shall define all design and operational loading conditions of Category I and Category II systems and components. The loading combination requirements and stress limit criteria shall be in accordance with the ASME Code, Sec. III, Div. 1 and Reg. Guides 1.48, 1.57, and 1.124.

7.4.2.2 Systems and Components Not Under Jurisdiction of the ASME Code. Systems and components whose design and fabrication are not subject to the ASME Code shall have their loading requirements and design criteria set forth in the project seismic criteria. Category I and Category II engineering safeguards systems, electrical instrumentation and control systems, and fire protection systems shall be subject to the same loading combinations required for ASME Code covered systems and components. Where such systems and components cannot be adequately analyzed, vibration testing shall be performed as called for in Sec. 8.

The loading combinations and design criteria for Category III systems and components shall be specified in the SDD.

7.4.2.3 Pressure Retaining Concrete Components. The design requirements, load combinations, allowable stress and deformation limits for concrete reactor vessel and containment structures shall be governed by the ASME Code, Sec. III, Div. 2.

7.4.2.4 Pressure Retaining Metal Components. The loading combinations and design criteria shall be set forth in the project seismic criteria and shall be in accordance with the applicable portion or portions of the ASME Code, Sec. III, Div. 1; ASME Code, Section VIII, Divisions 1 and 2; and in accordance with Reg. Guide 1.57.

7.5 Special Stress Intensity, Deformation, and Damage Criteria. The specific stress intensity, deformation, and damage criteria shall be specified for appropriate systems and components where they are required to preclude the loss of specific functions.
7.5.1 Deformation Limits. Any structure, component, or system for which excessive deformation will limit its performance, such as reactor internals or control rod drive systems, shall meet specified deformation limits for the applied loading combinations. The loss-of-function modes and limits on acceptable deformations shall be set forth in the system, component, or equipment specifications.

7.5.2 Damage Limits. For some components, operation and safety considerations may be governed by damage limits other than stress and deformation. In these instances damage limits shall be set forth in the applicable SDD, component design specification as required by the applicable code, or both.

7.6 Special Design Consideration. The nuclear facility shall be designed to preclude the loss of safety function for Category I structures, systems, and components resulting from seismic-induced movement of non-Category I structures, systems, and components. Where applicable, the design of the nuclear plant systems shall include, but not be limited to, the following special considerations.

7.6.1 Structures.

7.6.1.1 Inelastic Design. Design and analyses which incorporate provisions for inelastic deformation may be used. However, where such procedures are employed, justification shall be provided that all Category I and II structures perform their required safety and operational functions and that they do not induce failure or loss of function in related and/or supported Category I and II systems and components. Attention to design details, requirements for continuity, and related design factors shall be provided to ensure that the ductility requirements of inelastic action are provided without a loss of structural strength. Care must be taken that the allowed use of inelastic deformation to ease and benefit the seismic response of structures does not coincidentally reduce strength and ductility margins.

7.6.1.2 Anchorage. Anchor bolts and anchorage attachments shall be designed to accommodate the applied seismic loads and to restrict movement of equipment, supports, and framing within limits set forth in the design specification. The anchorage design shall be of sufficient detail to ensure that the required strength, ductility, and material quality are provided.

7.6.1.3 Support Framing. Structures and framing which provide support and/or bracing for equipment, such as heat exchangers, tanks, or similar items, shall be designed with the required strength and ductility to accommodate the imposed horizontal and vertical loads. The design analysis shall demonstrate that stresses in such support framing are within allowable limits and that buckling will not occur. Framing members shall be designed to accommodate shear reversals. This is a special design hazard in prestressed and reinforced concrete framing members.
7.6.1.4 Connections. The design of connections is fully as important as design of structural members. Connections shall, whenever possible, be designed so that yielding will occur in the members joined rather than in the connection itself.

7.6.1.5 Joints. Joints between different support systems or those within a system, such as expansion joints in reinforced concrete, shall be considered. Slippage at such joints shall be accounted for in the analytical model, if restraint is not provided in the design, fabrication, and construction.

7.6.1.6 Metallic Liners. Metallic liners for concrete reactor vessels and containments shall be designed and installed to accommodate the imposed seismic loadings and deformations within the requirements of the ACI-ASME Code.

7.6.1.7 Vibration Dampers. Sway braces or vibration dampers shall be used to control the movement of piping due to vibration. The restraint forces associated with these devices during nonseismic events shall be included in the analysis.

7.6.1.8 Seismic Snubbers. Mechanical or hydraulic devices that resist seismic induced pipe vibratory loads shall be included. These devices offer little resistance to relatively slow thermal motion and are not intended to carry the pipe weight. Appendix X2 provides recommended guidelines that address the modelling of seismic snubbers for the dynamic analysis of piping systems.

7.6.1.9 Nonstructural Elements. Nonstructural elements and components will participate in the dynamic response of the structure. They shall have sufficient flexibility to accommodate the deformations of the structure, and the design shall account for the behavior of the complete structure including the nonstructural elements.

7.6.2 Plant Equipment.

7.6.2.1 Fluid System Components. Unless otherwise permitted in the project seismic criteria, components, and equipment shall be anchored to their structural supports. If components must be suspended, lateral bracing shall be designed and installed to prevent uncontrolled displacements. Where such equipment must be allowed to move, its support system shall contain elements for damping or restricting the magnitude of movement within bounds. Where vibration isolation of components and equipment is necessary, suitable constraints shall be added to restrict movement within design limits, and the differing damping properties and amplification factors shall be included in the analyses. Where systems and equipment are allowed to sway, equipment connections require special design consideration and these conditions shall be included in the design analyses.
7.6.2.2 Piping Systems. Piping attachments to the supporting structure shall be designed to accommodate seismic induced vibratory loads. Recommended guidelines for the incorporation in the seismic analysis of the flexibilities of pipe clamps, extension struts, and civil structures, as well as the non-linearities associated with such elements, are given in App. X2.

Seismic induced loads shall be considered in the design of piping, piping supports and restraints as per the specifications outlined by the Power Piping Code B31.

The torsional and bending effects caused by valves, operators, etc., shall be considered in the design of piping systems.

Small diameter pipe less than 3 in. outside diameter may be analyzed using the simplified analysis approach of 6.1 unless prohibited by the piping specifications. Piping systems used for emergency core cooling are excluded from this group.

For LMFBR's, the piping associated with auxiliary cooling systems and auxiliary heat removal systems are also designed to accommodate seismic loads.

Buried piping requires special design consideration. Characteristics such as abrupt changes in stiffness at locations, such as building entrance points, valve blocks, and similar restraints, serve as special hazard or failure points. The design shall accommodate the seismic loads at such interfaces.

Piping systems that are required to operate at elevated temperatures are of special concern because the flexibility requirements for thermal expansion may be incompatible with the stiffness requirements to accommodate seismic loadings.

8. SEISMIC QUALIFICATION TESTING

Where specifically permitted by the project seismic criteria and as approved by the purchaser, verification testing may be used to satisfy the requirements for design analyses of applicable structures, systems, and components provided such verification testing is based upon the use of acceptable testing procedures and load levels that confirm the structural integrity and performance function. Structures, systems, and components of the nuclear plant, with the exception of primary supporting structures and systems, may be tested in accordance with a purchaser approved procedure in conjunction with design analysis.

Acceptable justification, procedural descriptions, and final results of any such testing program shall be documented and reviewed by the responsible designer. The minimum requirements for testing shall be as described in Sec. 6 of IEEE 344 and Reg. Guide 1.100 and supplemented as follo ws.
8.1 Testing Procedures. The applicable components, equipment, and assemblies shall be connected to the vibration generator in a manner that represents the service conditions expected for the item. Sufficient monitoring equipment shall be used to thoroughly evaluate performance during and after testing. Furthermore, monitoring equipment shall be located on the component or assembly so that the maximum response is always obtained for the item. To ensure acceptable testing, the tests shall be performed at the natural frequencies determined in 8.2, at predetermined frequencies, or at all frequencies within the range to which the item is to be qualified.

8.2 Resonance Search and Testing. Components and assemblies shall be subjected to an exploratory vibration test to determine the presence, if any, and location of natural frequencies within the frequency range to which the item is to be qualified. Where this is not specified, a range of 1 to 33 Hz should be used. An approved procedure shall be used to determine the resonance conditions. It is suggested that the search include a minimum of two continuous sweeps from 1 to 33 Hz using a maximum sweep rate of no greater than two octaves per minute and a minimum acceleration of 0.1 g. Single frequency resonance testing, as applicable, shall be performed at each natural frequency found by the resonance search and at other frequencies at intervals of one-half octave within the frequency range to which the item is to be qualified.

8.3 Testing Requirements. The test methods, requirements and criteria for single and multiple frequency testing, and for single and multi-directional testing shall be described in IEEE 344.

8.3.1 Cases in Which No Resonant Frequency is Found. If it can be shown that no natural frequency occurs below 33 Hz, the test input motion may be a single frequency motion with a dwell time of no less than 30 seconds duration. Alternately the test input motion may be as given in 8.3.2. The input level shall equal the maximum floor or support point acceleration resulting from the SEE for Category I and resulting from the OBE for Category II.

8.3.2 Cases in Which One or More Resonant Frequencies Are Found. The test input motion shall have a broad frequency content covering the range from 1 to 33 Hz. The motion may be a simulated earthquake ground motion time history, random motion, combination random and sine beat or complex wave motion provided that the test response spectrum envelopes the design response spectrum. The level of the input shall be such that the maximum acceleration is at least equal to the zero period acceleration on the design response spectrum. The testing method shall be as described in Sec. 6.6 of IEEE 344.

8.4 Performance Criteria. The performance of components and assemblies during and following vibration tests shall demonstrate their ability to accommodate specified seismic motion.
8.4.1 Mechanical Strength. The mechanical strength of the component or assembly may be determined by test or by extrapolation of the test data and analysis. Where applicable, the results of the tests and analysis shall demonstrate that stress, deformation, and special damage limits, as set forth in 7.4, 7.5, and the project seismic criteria, are not violated by seismic motion acting alone or in combination with other loads.

8.4.2 Functional Capability. Components or assemblies shall be capable of performing their intended function during and following vibratory tests equivalent to specified seismic motion. With approval of the purchaser, assemblies may be tested without components being in operation provided the components are performance tested following the test of the assembly. In these cases, the input vibratory motion for the component test shall account for any amplification of motion caused by the assembly.

9. DOCUMENTATION

The selected methods of seismic analysis for design; mathematical models and their natural frequencies; input time histories and their corresponding response spectra; damping values; and allowable stress criteria used in the design analysis shall be documented.

The documentation for each type structure, system, or component shall give details which demonstrate that the item meets the stated requirements when subjected to the seismic motion for which it is to be qualified. Where proof of performance is obtained by analytical means, it shall be presented in a form which is readily auditable by persons knowledgeable in such analyses. The analyses and test results obtained shall be certified to be in accordance with this standard, and such certification shall be made by the responsible professional engineer.
APPENDIX - NONMANDATORY

XI. DEFINITION OF DESIGN EARTHQUAKE GROUND MOTION

XI.1 SCOPE

This appendix provides recommended guideline procedures that can be used to define the earthquake vibratory ground motion required for the design of nuclear power plants and test facilities.

Strong earthquake vibratory ground motion is characterized for design of engineering facilities by its strength, frequency content, and duration. This appendix provides a review of published procedures for predicting the strength of vibratory ground motion as a function of earthquake magnitude, source distance and site-soil conditions, procedures for predicting the frequency content of ground motion in the form of response spectra, and for predicting the duration of strong ground motion.

This appendix was prepared by Agbabian Associates (AA) for the Materials and Structures Program sponsored by the Department of Energy, Office of Breeder Technology Projects.

XI.2 SUMMARY

The necessary steps associated with the deterministic or probabilistic definition of the design earthquake ground motion are outlined in the sections of this appendix that follow. Sections XI.2.1 and XI.2.2 are not within the scope of the recommended guidelines and are presented here as background information only.

XI.2.1 Geological and Seismological Investigation. A thorough and complete geological and seismological investigation of the site and region is required in order to identify capable faults and source areas that contribute to the seismic hazard associated with a proposed site. The nature and details of these investigations have not been considered within the scope of this study. However, the importance of these investigations should not be overlooked. Seismic and geologic siting criteria are set forth in 10 CFR 100, App. A, which is enclosed as an appendix to AA (1).* Complete familiarity with this document is assumed.

XI.2.2 Prediction of Maximum Earthquake Magnitude. The maximum earthquake magnitudes [or epicentral Modified Mercalli Intensities (MMI)] must be established for each capable fault or source area that can contribute in a significant manner to the seismic hazard associated with the proposed site. This also is not within the scope of this study but is provided as a part of the geological and seismological investigation performed in accordance with the siting criteria set forth in 10 CFR 100, App. A. It is an essential and difficult task and when done completely deterministically generally results in the assignment of overly

*The numbers in parentheses refer to the list of references at the end of this appendix.
conservative magnitudes. The earthquake magnitudes selected must be rational and believable events. As a general statement, the maximum magnitudes (or epicentral MMI) for each contributing source or source area should be based upon a careful consideration of the following factors:

a. the seismic history of the relevant fault zones, source areas, and host tectonic province

b. geologic and geomorphic evidence of recent fault movements, e.g., offset strata, scarps, scarp-like features

c. comparison of the seismic history of the zones and province with similar structural and tectonic environments elsewhere

d. empirical fault length-fault displacement-magnitude relationships (5).

An additional important guideline recommendation is that the maximum magnitudes (or epicentral MMI) selected should be plotted on the recurrence curves described in Sec. XI.2.5 to help establish that the maximum values are rational and believable events.

XI.2.3 Earthquake Ground Motion Attenuation Relationships. Relationships are required which express the rate at which the strength of the vibratory ground motions attenuate with distance from the epicenter, or center of energy release, of earthquakes. These relationships are required for deterministic as well as for probabilistic prediction of design ground motion. Ground motion attenuation relationships are normally expressed in terms of mean peak ground acceleration, velocity, and displacement, respectively, as a function of earthquake magnitude and source distance. The effect of site subsurface conditions on ground motion strength parameters should also be considered. In regions where strong ground motion acceleration records are not available, the distance attenuation relationships are generally expressed in terms of MMI. One of the major areas of emphasis in this study has been the review and evaluation of ground motion attenuation relationships. Guideline recommendations follow.

XI.2.3.1 Peak Horizontal Ground Acceleration. Peak horizontal ground acceleration-distance attenuation relationships by Bernreuter (4), Donovan (6), Housner (9), McGuire (11), Schnabel and Seed (15), Trifunac (18), Werner, Ts'ao, and Rothman (21), and AA (2), are reviewed and compared in AA (1). The following guidelines are recommended.

a. Based on the scatter in the available data, the limited number of records available for rock sites and the uncertainties in the classification of site subsurface conditions at recording sites, site subsurface conditions have not been demonstrated to be a significant correlation factor in regression analyses of peak ground acceleration as a function of magnitude and source distance. Therefore, for rock and
stiff soil sites it is recommended that peak ground acceleration be
assumed to be independent of site subsurface conditions. Adjustments
in peak ground acceleration for soft clay and deep cohesionless soil
sites may be made if justified by special studies.

b. The (adjusted) Werner relationship given below in Eq. XI.1 with
$E = 1$ is recommended as a guideline for regions of reverse faulting in
the western United States where focal depths of 10 to 15 km are
anticipated, accompanied with surface manifestations of faulting. This
relationship should be assumed to be equally applicable (at the ground
surface) to rock and to stiff soil sites.

$$\ln a' = 12.38 - \frac{32.85}{M} - 1.01 \ln R$$
$$+ 1.72 E - 0.38 (\ln R)E$$

(XI.1)

Here

$\ln a'$ = Natural logarithm of the peak horizontal
ground acceleration in in./sec$^2$

$M$ = Richter magnitude

$R$ = Hypocentral distance in km

$E$ = A constant based on fault conditions

c. In regions of normal and strike-slip faulting in the western
United States, lower peak ground accelerations than indicated by
Eq. XI.1 with $E = 1$ may be justified. However, it is recommended that
values not be used that are less than given by Eq. XI.1 with $E = 0.545$
unless supported by studies based on regional acceleration records from
historical earthquakes.

d. In regions where earthquakes have significantly different
focal depths than typical of California earthquakes, and in regions
where MMI attenuation relationships are significantly different than
experienced in California, the direct applicability of peak horizontal
ground acceleration/magnitude/distance relationships based essentially
on California earthquakes is questionable. In these regions, the
MMI attenuation procedures given in Sec. XI.2.3.6 are recommended.

XI.2.3.2 Peak Horizontal Ground Velocity. Peak horizontal
ground velocity/distance attenuation relationships by McGuire (11),
Triplunac (18), Seed et al. (16), and Werner et al. (21), are reviewed
and compared in AA (1). The following guidelines are recommended.
a. The (adjusted) Werner relationship given below in Eq. XI.2 with $E = 1$ is recommended as a guideline for regions of reverse faulting in the western United States where focal depths of 10 to 15 km are anticipated, accompanied with surface manifestations of faulting. The effect of site subsurface conditions should be considered in peak ground velocity predictions. All predictions should be based on a careful study of the local geology and site surface conditions that determine the susceptibility of the site to surface wave motions.

$$\ln v' = 8.60 - 31.99/M - 0.66 \ln R + 2.98E \quad (\text{XI.2})$$

$$-0.66 (\ln R)E + 0.07 (\ln R)S_D$$

$$- 0.15 (\ln R)S_R$$

Terms not defined previously in Sec. XI.2.3.1, relative to Eq. XI.1, are defined as follows:

$$\ln v' = \text{Natural logarithm of the peak horizontal ground velocity in in./sec}$$

$S_D = \text{Deep soil site classification factor (equals 1 for deep soil sites and 0 for all other sites)}$

$S_R = \text{Rock site classification factor (equals 1 for rock sites and 0 for all other sites)}$

b. In regions of normal and strike-slip faulting in western United States where strong surface waves are not anticipated, lower peak velocities than given by Eq. XI.2 with $E = 1$ may be justified. However, it is recommended that values not be used that are less than given by the Werner (adjusted) relationship given in Eq. XI.2 with $E = 0.545$. When strong surface waves are anticipated, it is recommended that $E$ be assumed equal to unity in all cases.

c. In regions where earthquakes have significantly different focal depths than typical of California earthquakes and in regions where MMI attenuation relationships are significantly different than experienced in California, direct applicability of peak horizontal ground velocity/magnitude/distance relationships based on California earthquake data is questionable. In these regions, MMI attenuation procedures given in Sec. XI.2.3.6 are recommended.

XI.2.3.3 Peak Horizontal Ground Displacement. Peak horizontal ground displacement attenuation relationships by McGuire (11), Trifunac (18), and Werner et al. (21), are reviewed and compared in AA (1). The same guidelines are recommended for peak ground displacement as given in Sec. XI.2.3.2 for peak ground displacement as given in
Sec. XI.2.3.2 for peak ground velocity except the relationships given below in Eq. XI.3 should be used in place of Eq. XI.2.

\[ \ln d' = 5.89 - 26.87/M - 0.36 \ln R + 3.11 \text{E} \]
\[ -0.68(\ln R)\text{E} + 0.06(\ln R)\text{S}\_\text{D} - 0.17(\ln R)\text{S}\_\text{R} \]

All terms in the above equation are as previously defined in Secs. XI.2.3.1 and XI.2.3.2, except \( \ln d' \), which is defined here as the natural logarithm of the peak horizontal ground displacement in inches.

XI.2.3.4 Peak Vertical Ground Motions. The mean peak vertical ground acceleration, velocity, and displacements can be assumed to be two-thirds of the respective mean peak horizontal components of motion, or the correlation equations given above in Secs. XI.2.3.1, XI.2.3.2, and XI.2.3.3 can be used with the following adjustments in the constant of each equation:

In Eq. XI.1, 11.71 should be substituted for 12.38,
In Eq. XI.2, 7.85 should be substituted for 8.60,
In Eq. XI.3, 5.30 should be substituted for 5.89.

XI.2.3.5 Mean Plus One Standard Deviation Ground Motions. Mean attenuation relationships for peak horizontal ground acceleration, velocity, and displacement have been recommended above for predicting site ground motions as current response spectra amplification factors are based on mean peak ground motions. However, mean plus one standard deviation (\( M + \sigma \)) peak ground motions are frequently required for other purposes. It is recommended that Eqs. XI.1, XI.2, and XI.3 given in Secs. XI.2.3.1, XI.2.3.2, and XI.2.3.3, respectively, be used as guidelines to obtain (\( M + \sigma \)) ground motions with the following adjustments in the constant of each equation:

In Eq. XI.1, 12.95 should be substituted for 12.38,
In Eq. XI.2, 9.13 should be substituted for 8.60,
In Eq. XI.3, 6.50 should be substituted for 5.89.

XI.2.3.6 Modified Mercalli Intensity (MMI) Attenuation Relationships. In regions where there are no strong motion acceleration records available, which is typical of the eastern two-thirds of the United States, isoseismal maps of historical regional earthquakes may be used to establish the rate of attenuation of earthquake ground motion with distance. In these regions, it is generally more convenient to express historical and design earthquakes in terms of epicentral MMI. Regional MMI/distance attenuation relationships should be developed from regional isoseismal maps based on the average distances from the epicenter to the isoseismal lines. Since there are many factors other than strength of ground motion that may have affected the assigned MMI values of historical earthquakes, some adjustment in MMI
may be required to better reflect the rate of attenuation of strength of motion. These factors are discussed in AA (1).

If design earthquakes for a proposed site are expressed in terms of MMI and represent values that have been derived from average regional MMI/distance attenuation curves, the following guidelines are recommended for estimating the mean peak ground acceleration, velocity, and displacement for the design earthquake. Because of lack of recorded ground motion at epicentral distances less than 20 km, the MMI/mean peak ground motion relationships have been inferred above MMI VIII.

Based on present knowledge, the relationship for mean peak horizontal ground acceleration given in Fig. XI.1 is recommended as a guideline for isoseismal MMI values of VIII and below. Guideline recommendations for MMI/mean peak horizontal ground velocity and MMI/mean peak horizontal ground displacement are provided in Figs. XI.2 and XI.3, respectively. These curves are based on Trifunac and Brady (19) data and the mean peak ground motion/distance relationships given in Secs. XI.2.3.2 and XI.2.3.3. These guideline relationships should be assumed to be representative of average conditions in California (i.e., comparable to the relationships evaluated for E = 0.545). When the regional MMI/distance attenuation relationships are significantly different than California MMI/distance relationships, some adjustment in the MMI/peak ground motion guideline relationships may be required to better represent the regional attenuation characteristics. These adjustments must be based on special studies for each region.

In many regions of the eastern United States, present deterministic procedures of 10 CPR 100, App. A require that nuclear facilities be designed for epicentral ground motions, and MMI/distance attenuation relationships are not required. When the design earthquake can be specified in terms of magnitude, the mean peak ground motions resulting from Eqs. XI.1, XI.2, and XI.3 for E = 0.545, with R set equal to the estimated focal depth, are recommended as guidelines for regions where there is no evidence of surface faulting. When the design earthquake must be specified in terms of epicentral MMI values, the MMI/mean peak ground motion guideline relationships given in Figs. XI.1, XI.2, and XI.3 may be used as guidelines if the anticipated focal depths are 20 km or more and there are no surface manifestations of faulting. It is recommended that the peak vertical motions be taken as two-thirds of the peak horizontal motions as a guideline in regions where MMI data must be used as a basis for ground motion predictions.

The \((M + \sigma)\) peak ground motions are ordinarily not used to scale response spectra but may be of value in meeting other design requirements. Standard deviation values have been given by Trifunac and Brady (19) for peak ground acceleration, velocity, and displacement, but these values have been computed for each individual MMI area. The standard deviation for an MMI area should be greater than for an isoseismal MMI since the former has a distance variation effect (i.e.,
Fig. XI.1. Guideline isoseismal MMI/ Mean peak horizontal ground acceleration relationships.
Fig. X1.2. Guideline isoseismal MMI/Mean peak ground velocity relationships.
Fig. XI.3. Guideline isoseismal MMI/Mean peak ground displacement relationships.
variation in peak motion with distance across the MMI area). Additional study is needed to evaluate this effect quantitatively. For the present, it is recommended as a guideline that the logarithm of the mean values for the peak ground acceleration, velocity, and displacement found from Figs. XI.1, XI.2, and XI.3, respectively, be increased by the standard deviation values given in Table XI.1 to obtain the logarithm of design \((M + \sigma)\) values.

**Table XI.1. Standard Deviation Values**

[Werner et al. (21)]

<table>
<thead>
<tr>
<th>Ground Motion</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration</td>
<td>0.57</td>
</tr>
<tr>
<td>Velocity</td>
<td>0.53</td>
</tr>
<tr>
<td>Displacement</td>
<td>0.61</td>
</tr>
</tbody>
</table>

XI.2.4 **Probabilistic Assessment.** The probabilistic assessment requires two additional steps that are the development of a stable recurrence relationship for earthquakes as a function of magnitude or epicentral MMI for each source or source area, and the performance of probability calculations. These procedures are reviewed in AA (1).

The most important requirement in the development of the recurrence relationship for earthquakes for a site assessment is that careful attention be given to the historical record of earthquakes. Even though historical accounts of large earthquakes may extend over a period of nearly 200 years, the population density has not been sufficient until the last 50 to 100 years to guarantee that all intermediate magnitude earthquakes have been noted. Also, adequate instrumentation to detect and record low magnitude earthquakes in most regions of California has been in existence for less than 50 years and does not exist in most regions of the United States today. Extreme care must therefore be exercised in interpreting the historical data.

Mathematical models for performing the probability calculations are described in AA (1). There have been great advances in modeling techniques recently. Relationships between magnitude and fault length, and fault location and focal depth can all be considered. In general, the models are sophisticated and are limited only by present knowledge of earthquake phenomenology. The probability calculations should be based on mean peak ground motion attenuation curves to be consistent with the amplification factors used to scale \((M + \sigma)\) design response spectra, which are also based on mean peak motions.

XI.2.5 **Design Response Spectra.** If the design ground motions are to be established in a completely deterministic manner, the mean peak ground acceleration and velocity should be predicted for the site based on the design earthquakes derived from the steps described above in Secs. XI.2.1 and XI.2.2, and the distance attenuation relationships described in Sec. XI.2.3. These values should then be used to scale
design response spectra. However, this study recommends that the probability of the site experiencing these mean peak values of ground motion first be determined so that overly conservative or overly unconser-vative values can be detected and corrected at an early date. At the present time, acceptable probability values for the design earthquake ground motions are not available. However, values having an annual probability of occurrence of \( 1 \times 10^{-4} \) are recommended as a guideline for the SSE when the calculations are based on mean peak ground motion distance attenuation curves.

A review and comparison of published procedures for developing design response spectra have been provided in AA (1) (3). Procedures reviewed include those by Housner (8); Newmark and Hall (12) (13); Reg. Guide 1.60 (20); Mohraz (10); Seed et al. (16); Guzman and Jennings (7); and Werner et al. (21). At the present time, Reg. Guide 1.60 response spectra for horizontal response are recommended as guidelines for site independent response spectra. The Newmark and Hall (13) procedures when based on an acceleration : velocity : displacement ratio consistent with the Standard Earthquake ratio of 1.0 g : 48 in./sec : 36 in., give comparable site-independent horizontal response spectra.

The most important improvement that can be made to procedures for developing design response spectra at this time is to use the site-dependent mean peak ground motions given in Sec. XI.2.3 as scaling factors rather than the Standard Earthquake ratio of peak values. For deterministic design criteria, it is recommended that the amplification factors given in Fig. XI.4, which have been taken from Newmark and Hall (13), be applied to site-independent design values of peak horizontal ground acceleration and velocity obtained by the procedures given in Sec. XI.2.3 to obtain guideline horizontal response spectra. The procedure is illustrated in Fig. XI.4. A significant reduction in response will usually result for rock and intermediate (or firm soil) sites below frequencies of about 2 Hz from that given by Reg. Guide 1.60 or Newmark and Hall procedures that are based on Standard Earthquake peak ground motion ratios. Higher response than obtained by these site-independent procedures will frequently be obtained below frequencies of about 2 Hz for deep soil sites when site-dependent ground motions are used. The same procedures are recommended as guidelines for the construction of vertical response spectra. The vertical ground motions may be taken as two-thirds of the horizontal ground motions, or the vertical motions can be computed as indicated in Sec. XI.2.3.4.

Response spectra can be constructed using peak ground accelerations and velocities that have been computed from a probabilistic assessment similar to that described in Sec. XI.2.4. The guideline report points out that when response spectra are based on mean peak ground motions that have been selected probabilistically to represent values that have an annual probability of \( 1 \times 10^{-4} \) of occurrence,
Percent of Critical Damping | Spectrum Amplification Factors
---|---|---
0.5 | 5.10 | 3.84
2.0 | 3.66 | 2.92
5.0 | 2.71 | 2.30
7.0 | 2.36 | 2.08
10.0 | 1.99 | 1.84

(a) Amplification factors

Fig. X1.4. Comparison of site-dependent guideline response spectra and Newmark and Hall (13) response spectrum.
the resulting spectra should be corrected for frequencies above about 20 Hz to the \((M + \sigma)\) peak ground acceleration in order to maintain a uniform level of conservatism in response at all frequencies (Fig. XI.4). The difference between the mean and \((M + \sigma)\) peak horizontal ground acceleration is comparable to the difference between the peak and "effective peak" ground acceleration that has confused communication between geological engineers and seismologists on one hand, and geotechnical and structural design engineers on the other [Page et al., (14)]. Geological engineers and seismologists are well aware that peak ground accelerations may occur at a proposed site that are significantly greater than the peak ground accelerations frequently used by the geotechnical and structural design engineer to scale design response spectra. The reason that lower (mean) values are correctly used by the latter, however, is that the amplification factors used to scale \((M + \sigma)\) response spectra are based on the lower (mean) values. However, current procedures for scaling design response spectra yield response above 20 Hz that is less conservative than the response below 20 Hz unless the above correction is made.

When a probabilistic assessment is used to establish the peak design ground motions, the results should normally be presented in terms of the annual, or service life, probability of occurrence of the mean peak ground acceleration (and velocity), or in terms of the annual, or service life, probability of occurrence of the isoseismal MMI. Fig. XI.5 provides an example for a California site in which mean peak ground acceleration is expressed in terms of annual probability, and Fig. XI.6 provides an example in which the service life probability of occurrence of the isoseismal MMI is provided for a site in the southern Appalachian seismic zone. Hopefully, in the future, acceptable annual or service life probabilities will be specified by regulatory agencies for design earthquakes. When this is done the appropriate design acceleration and velocity, or isoseismal MMI, can be selected, based on the required probability level. Only in this way can nuclear plants all be designed to the same level of seismic risk. Until this has been done, the results of the probabilistic assessment can only be used to check the probability level of the values estimated for design by present deterministic procedures.

Based on the discussion in the guideline report (1), values having an annual probability of not being exceeded of 99.99% are recommended as reasonable values for the Safe Shutdown Earthquake (SSE). This is equivalent to a value that has a 50-year service life probability of 99.5% of not being exceeded and to an annual probability of occurrence of \(1 \times 10^{-4}\). Using this probability level, the SSE for the California site could be based on a peak horizontal ground acceleration of 0.5 g (Fig. XI.5) and the SSE for the southern Appalachian seismic zone site could be based on an isoseismal MMI VII + 1/2 (Fig. XI.6).

For the California site considered above, the mean peak ground acceleration (and velocity) having an annual probability of occurrence
Fig. XI.5. Probability of annual occurrence of mean peak ground acceleration at hypothetical site in California.
Fig. X1.6. Modified Mercalli Intensity probability relationships for hypothetical site in the southern Appalachian seismic zone. Firm site conditions.
Fig. XI.7. $(M + c)$ Response spectrum on $1 \times 10^{-4}$ annual probability
mean peak ground motions – hypothetical site in California.
of $1 \times 10^{-4}$ have been used to scale the design response spectra given in Fig. XI.7. The spectrum has been adjusted for frequencies above about 20 Hz to the $(M + \sigma)$ peak ground acceleration that is a value of 0.9 g. It is of interest to note that nuclear power plants in the coastal region of California are currently based on Reg. Guide 1.60 spectra scaled to peak ground accelerations of 0.67 to 0.75 g.

For the site in the southern Appalachian seismic zone considered above, the isoseismal MMI VII + 1/2 can be used to select the mean peak ground acceleration and velocity needed to scale the design response spectra. Figs. XI.1 and XI.2 indicate a mean peak ground acceleration and velocity for rock or firm soil sites of 0.15 g and 6.5 in./sec, respectively. Assuming these values would be applicable to the site, the spectrum given in Fig. XI.8 for intermediate site conditions would apply, except for frequencies above 20 Hz. For frequencies above about 20 Hz, the response spectrum has been raised to the $(M + \sigma)$ peak ground acceleration. This spectrum is compared in Fig. XI.8 with a Reg. Guide 1.60 spectrum scaled to 0.25 g, which represents the level of response currently being provided for several nuclear plants in this region.

It should be apparent that when deterministic peak ground motion values are used to scale design response spectra that are higher than obtained from a probabilistic analysis based on mean ground motion attenuation curves, overly conservative response will be obtained, except for frequencies above about 20 Hz. It is noted that currently only the deterministic procedure is an acceptable regulatory procedure.

XI.2.6 Design Acceleration Records and Duration of Strong Motion. Guideline procedures for developing design acceleration records that envelop design response spectra and guideline procedures for estimating the duration of strong motion are given in AA (1).

REFERENCES


Fig. X1.8. \((M+\sigma)\) Response spectrum from \(1 \times 10^{-4}\) annual probability mean peak ground motions - hypothetical site in southern Appalachia Seismic zone.


APPENDIX - NONMANDATORY

X2. SEISMIC ANALYSIS OF PIPELINES SUPPORTED BY SEISMIC SNUBBERS

X2.1 SCOPE

This appendix provides recommended guidelines and procedures for use in the analysis of pipelines supported by seismic snubbers.

This appendix condenses the knowledge gained from the Fast Flux Test Facility (FFTF); consequently, the guidelines are associated with thin wall piping, the kind found in the main heat transport of Liquid Metal Fast Breeder Reactors (LMFBR).

This appendix was prepared by Hanford Engineering Development Laboratory (HEDL) for the Materials and Structures Program sponsored by the Department of Energy, Office of Breeder Technology Projects. The source document for this appendix is Anderson (1).*

X2.2 SUMMARY

X2.2.1 Pipeline Analyses with Seismic Snubbers.

1. Seismic snubbers may introduce additional support flexibility and certain non-linear characteristics into pipe support systems. The effect of these characteristics is to usually reduce system natural frequencies. This may either increase or decrease pipe system stress and support loads depending on the relationship of the modified system natural frequencies to the predominant seismic forcing frequencies. These characteristics are described in Sec. X2.3.2.

2. Many snubber types and sizes are produced by several manufacturers. No one analytical model can adequately represent all of these. Several types of analytical models are presented in Sec. X2.3.3. Both linear and non-linear types are described.

3. Seismic snubbers are attached to pipe systems in series with other elements. These may include pipe clamps, extension struts, and civil support structure. Additional flexibilities and non-linearities introduced by such additional elements must be incorporated into the pipe support model as described in Sec. X2.3.4.

4. Seismic dynamic motion at a specific plant site may be defined as input response spectra or acceleration time-histories. This input must be further modified to account for any additional flexibility between the defined point and the pipe support attachment points as described in Sec. X2.4.1.

*The numbers in parentheses refer to the list of references at the end of this appendix.
5. Pipeline finite element models must be sufficiently detailed to reproduce the dynamic response of the full scale system. Modeling concerns are described in Sec. X2.4.2.

6. Most pipeline seismic analyses are conducted using linear response spectrum techniques. Results are evaluated against screening limits. The less conservative and more mathematically accurate non-linear methods are more costly and are usually utilized when designs cannot be shown acceptable by the simpler methods. Screening limits and analysis comparisons are discussed in Sec. X2.4.3.

X2.2.2 Seismic Support Spacing.

1. Prior to conducting detailed seismic analyses, it is necessary to select seismic support spacing. Sec. X2.5.1 presents guidelines for selecting preliminary support spacing for straight pipe, pipe with concentrated loads, crossover pipe and vertical risers. Support spring rates are also recommended for various pipe sizes.

2. Support loads for small diameter pipe systems, operating at low temperatures, may be established without extensive analysis. Sec. X2.5.2 presents guidance on establishing these loads and discusses limits of applicability.

X2.3. SEISMIC SNUBBER CHARACTERISTICS

X2.3.1 Snubber Types.

X2.3.1.1 Hydraulic Snubber. Recent designs of this device use a hydraulic fluid filled cylinder with a double acting piston to control seismic loads. Under slowly applied motions, such as thermal expansion or contraction, the hydraulic fluid passes through a poppet valve or control orifice allowing the piston to move with little resisting force usually, on the order of 1 to 2% of rated load. Under higher velocities, such as induced by a seismic event, fluid flow is restricting causing the snubber to act as a load carrying strut. Both tension and compression loads are resisted by the double acting piston.

X2.3.1.2 Mechanical Snubber. This device generally uses a mechanism to convert translational motion to rotary motion of an inertia mass. This device also allows relatively unrestricted slowly applied motions. However, under more rapid seismic motions, the inertia mass activates a braking mechanism which again causes the snubber to act as a load carrying strut. These devices also resist both tension and compression loads.

X2.3.2 Dynamic Characteristics.

X2.3.2.1 Dynamic Stiffness. The seismic response of a pipe system is directly related to the proximity of the system eigen frequencies to the predominant input frequencies of the imposed seismic
The dynamic stiffness of the snubbers has a major influence on the resulting eigen frequencies of the pipe system when modeled in dynamic analyses. Thus, snubber dynamic stiffness must be accurately determined and modeled to reliably predict the pipe system seismic response. The effective stiffness of a snubber may be load and frequency dependent, and is a function of clearances, snubber internal mass properties, and of acceleration or velocity lockup threshold.

X2.3.2.2 Damping. Snubber damping has been shown to have a significant effect in attenuating the seismic response of small diameter pipe systems (2)(3). It tends to be less important on larger diameter pipe on a system basis, but may have a significant localized effect in attenuating individual support seismic loads. Snubber damping has been found to be relatively insensitive to applied load level but appears to be very sensitive to frequency. Snubber damping is a function of the snubber effective mass and stiffness, freeplay which affects impact damping, and of internal friction.

X2.3.2.3 Freeplay. Freeplay results from clearances within the snubber mechanisms and from the linkage system between the snubber and the supporting structure and between the snubber and the pipe clamp. The freeplay modifies the piping/support response characteristics by producing an effective nonlinear stiffness. Freeplay effects can either amplify or attenuate the system response depending upon the set of circumstances. Support loads can be amplified due to impacting of relatively hard structures in combination with relatively large clearances. The load amplification in this case will increase approximately as the square root of the increase in the structural stiffness and the clearance between the impacting structures. In the case of relatively soft structures, freeplay may have the effect of attenuating the system response through impact damping. Yet another possible effect of freeplay is to detune the piping system from a resonant or near resonant frequency condition. Large reductions in seismic response can occur due to the inability of the system to sustain a response in concert with the forcing frequency.

X2.3.2.4 Typical Snubber Characteristics. Typical test load and displacement data for a mechanical snubber, with an alternating load excitation frequency of 3 Hz, is shown in Fig. X2.1. Note that the maximum force occurs at the time of zero displacement while zero force occurs at maximum displacement. This implies that the force-time displacement characteristics could be modeled by a simple viscous damper. Fig. X2.2 shows the same test data at an excitation frequency of 9 Hz. Here, the force and displacement are more nearly in-phase and indicate that a combined spring and damper model would be needed.

Figures X2.3 and X2.4 show the Lissajous patterns for the same snubber at 3 Hz and 9 Hz respectively. The area within the curves is a measure of the effective snubber damping. The displacement at zero load is a measure of the snubber dynamic freeplay.
Fig. B-1. Typical mechanical snubber load and displacement as a function of time at 3 Hz.

Fig. B-2. Typical mechanical snubber load and displacement as a function of time at 9 Hz.
Fig. B-3. Typical mechanical snubber Lissajous pattern at 3 Hz.

Fig. B-4. Typical mechanical snubber Lissajous pattern at 9Hz.
Similar test data at other frequencies and load levels demonstrate that snubber dynamic characteristics are both frequency and load magnitude dependent (4)(5).

From the preceding it is apparent that seismic snubbers exhibit certain nonlinear characteristics. It is necessary to perform dynamic tests to define these characteristics for all types and sizes of snubbers to be used in a pipe system. These tests should be conducted over a range of load levels and test frequencies sufficient to envelop the intended application.

Once these test data are available, dynamic analysis models can be generated to duplicate the test data. These models can then be used to perform the pipe system seismic analysis.

X2.3.3 Snubber Analytical Models. Many pipeline seismic analyses have been used on the assumption that pipe supports are rigid compared to the pipe system as shown in Fig. X2.5. This assumption is valid only if it can be demonstrated that the pipe support structure does not amplify the seismic input accelerations. Seismic snubbers have a certain amount of freeplay which may lead to impact load amplification and a reduction in their effective dynamic spring rates. For this reason, the rigid support assumption is not usually valid for pipe lines supported by seismic snubbers and may result in under prediction of seismic loads (6). This is particularly true when one considers that snubbers are usually attached to pipelines using pipe clamps which may themselves have relatively soft spring rates. Since the clamp and snubber spring rates are in series, the effective support train spring rate is likely to be much less than any of its elements. This problem is discussed further in Sec. X2.3.4.

In order to generate an accurate estimate of the loads and stress levels induced in a pipe system by a seismic event, it is usually necessary to incorporate some form of snubber model into the pipeline analytical model. There are two primary requirements for such models: first, they must accurately reproduce the major dynamic characteristics of the snubber; second, they must be reasonably simple so as not to overload computer capacity when incorporated with a typical pipeline model.

No one snubber model may be sufficient to reproduce the seismic characteristics of the many types and sizes of snubbers made by the various manufacturers. The following paragraphs describe several snubber models which have been successfully applied to pipe seismic analyses. The reader should bear in mind that modification to these models or entirely different models may be required to properly represent a given snubber design.

X2.3.3.1 Linear Spring Model. The most commonly used snubber model is the linear spring as shown in Fig. X2.6. Sufficient data are often available from snubber vendors to determine the equivalent
Fig. X2.5. Rigid support assumption.

Fig. X2.6. Linear spring assumption.
Fig. X2.7. Average spring rate and dynamic freeplay from snubber test data.

\[ \text{Avg } k = \frac{\text{Deflection Range}}{\text{Load Range}} \]
\[ \text{Deflection Range} \approx 2700 \times 0.038 = 6550 \text{ lb/in.} \]

\[ \text{Freeplay} = 0.013 + 0.009 \]
\[ \text{Freeplay} = 0.011 \text{ inch} \]
dynamic linear spring rate of the snubber for finite number of frequencies and load levels. This spring rate is usually determined from snubber test data Lissajous patterns as shown in Fig. X2.7.

Although this snubber model includes free travel displacement in determining the equivalent linear spring rate, it cannot account for nonlinear impact load amplification and damping. Nor can it account for viscous or Coulomb damping or for spring rate changes caused by frequency or load level effects.

However, if its limitations are understood, the linear spring model can be a useful aid in estimating seismic loads and piping stresses. The model is usually used with the linear response spectrum analysis technique. Initial support spring rates are estimated and an analysis conducted. The resulting support load levels and predominant contributing frequencies are determined and compared with available snubber data. A set of snubbers is selected and the spring rates adjusted to match their characteristics. Several iterations may be required to achieve an acceptable support system design.

Analytical results obtained using spring models with the response spectrum technique have generally been found to be conservative when compared with those obtained using more complex models and time-history seismic analysis (2). One reason for this is that nonlinear impact amplification effects are usually compensated by impact damping for the small freeplay usually present in modern snubbers. However, as freeplay increases, load amplification can occur as described in Brussalis (6) and Barta (7).

It is also clear that much of the apparent conservatism is simply due to the difference between using an enveloping response spectrum analysis and a more exact time-history analysis. Excess conservatism is not necessarily beneficial in the design of a nuclear pipe system. It requires the designer to install more seismic supports than may be absolutely essential. This increases initial costs and surveillance and in-service inspection costs over the plant lifetime. It can also lead to reduced system reliability since more component parts are required. To better understand the seismic response of a pipe system and to minimize the number of supports, the analyst may be required to use more sophisticated analytical models together with nonlinear time-history analyses.

X2.3.3.2 Voigt Model. The linear Voigt model represents the snubber with a dashpot and spring in parallel as shown in Fig. X2.8. Nonlinear characteristics can be simulated by incorporating a gap element as shown.

The model produces an elliptical Lissajous pattern with the two halves displaced if a gap element is included. The model will adequately duplicate snubber test data at a given frequency and load level. The dynamic spring rate, freeplay, and damping can easily be extracted from tests data Lissajous patterns.
\[ C = \frac{E}{\pi x_0^2 w} \]
\[ K = \sqrt{\frac{F_0^2}{x_0^2} - w^2 c^2} \]

where

- \( F_0 \) = maximum force - lbs.
- \( x_0 \) = maximum displacement - in.
- \( \Delta X \) = freeplay - in.
- \( E \) = energy dissipation per cycle = Area of Lissajous pattern lb-in.
- \( w \) = excitation frequency rad/sec

Fig. X2.8. Voigt model.
The model has the advantage of being relatively simple and can be utilized with most computer codes capable of pipeline dynamic analysis. It does incorporate snubber damping and, with the gap element, accounts for freeplay.

It has the disadvantage that it does not duplicate detailed snubber dynamic response in that dynamic spring rates, at points other than maximum force and maximum displacement, are not represented. It is not frequency or fully load level sensitive since only one set of damping, spring rate and freeplay values can be input.

The model can be successfully used to perform pipeline seismic analysis if an iteration procedure, similar to that recommended for linear springs, is employed. Such a technique was utilized in Barta et al. (2) to qualify FFTF pipe systems.

X2.3.3.3 Maxwell Model. The Maxwell model represents the snubber with a dashpot and spring in series as shown in Fig. X2.9. As in the Voigt model the Lissajous pattern is represented by an ellipse. Nonlinear characteristics can be handled with a gap element.

For the Voigt model, damping is linearly proportional to frequency, while in the Maxwell model damping is maximum at specific frequencies and less at other frequencies.

With the above exceptions, the Maxwell model has the same advantages and limitations as the Voigt model.

X2.3.3.4 Resonant Mass Model. Resonant mass models consist of a spring, lumped mass, and dashpot which may be arranged in several ways. A gap element can be included to model non-linear effects (Fig. X2.10).

The Lissajous patterns for these models are also elliptical. However, the model parameters may not easily be determined from snubber test data using simplified methods. More sophisticated data matching techniques, such as described by Hulbert and Schott (8), may be required to select appropriate mass, spring rate, and damping values. However, the resonant mass model can successfully reproduce snubber dynamic characteristics including both frequency and load dependent effects.

X2.3.3.5 Mechanistic Model. A snubber may be modeled by considering the force transfer characteristics and constitutive relations for each component part of its mechanism. Each kinematic state which interconnects these parts is considered. Equations of motion can then be written. Such a mechanistic model was successfully developed by Agbabian Associates for a particular snubber design as shown in Fig. X2.11 and further described in AA (9).
where

\( F_0 \) = maximum force - lbs

\( X_0 \) = maximum displacement - in

\( \Delta X \) = freeplay - in

\( E \) = energy dissipation per cycle = Area of Lissajous pattern = lb/in

\( w \) = excitation frequency rad/sec

Fig. X2.9. Maxwell model.
Fig. X2.10. Resonant mass models.

Fig. X2.11. Mechanistic model
This complex model successfully duplicated snubber dynamic test data. It was an excellent tool for appraising the effect of snubber internal mechanisms on the snubber response characteristics. However, such a model requires a large effort to define the many parameters and contains a large number of degrees of freedom. It is unwieldy for use in normal pipe system analysis.

X2.3.4 Support Structure Effects. Seismic snubbers are usually part of a pipe support system consisting of several additional elements which may include a pipe clamp, extension strut, and civil structure as shown in Fig. X2.12. These elements may be in series with the seismic snubber. Once an analytical model is selected for the snubber, additional modifications may be required to account for the flexibility of these other support elements before detailed pipe seismic analysis can proceed.

X2.3.4.1 Pipe Clamps. A wide range of pipe support clamp designs are utilized on nuclear piping. It is necessary that the spring rate characteristics of these clamps be considered in the pipe seismic analysis.

A typical load-deflection curve is shown in Fig. X2.13 for an insulated pipe clamp used on LMFBR thin-walled Sodium pipe. Note that the spring rate of this clamp is relatively soft and is also dependent on the loading direction with respect to the split line. Note also that the clamp exhibits non-linear characteristics in that it is stiffer in compression than in tension.

An additional complication arises when two seismic snubbers are attached to the same clamp. Load at one location will cause deflection at the second location and vice-versa. This cross-coupling causes a constantly changing spring rate since the loads are changing in both magnitude and direction with time. This effect appears to be best handled by assuming a range of high and low spring rates for the particular location and conducting sensitivity seismic studies over the range as was done in Barta et al. (2).

Pipe clamps often have substantial mass and this can affect both the dead weight and seismic analyses of the pipe system. Therefore, the clamp masses should be included in the pipeline analytical model.

X2.3.4.2 Pipe Strut Extensions. When pipe strut extensions are attached to seismic snubbers, they also add increased flexibility to the combined pipe support system. This increased flexibility should be incorporated into the combined support system analytical model. Care must also be exercised to ensure that snubber load ratings are not excessively reduced through buckling or parametric resonance conditions as described in Anderson and Entz (10).
Fig. X2.12. Typical seismic snubber installation.

Fig. X2.13. Typical insulated pipe clamp load-deflection curves - FFTF 28-inch pipe.
X2.3.4.3 Civil Structure. Civil support structures are usually designed after the pipeline analysis is completed. The pipe designer may then specify the required support stiffness at each seismic support attach point. It is then necessary to ensure close coordination between the pipe designer and the civil support structure designer to ensure that the requirements are clearly understood and effectively implemented. Special concerns related to civil structure design and flexibility are detailed in Refs. (11), (12), and (13).

X2.3.4.4 Composite Support Structure Model. A typical composite pipe support structure analytical model is shown in Fig. X2.14. This model is based on the Voigt nonlinear snubber model. It incorporates additional springs in series to represent the pipe/clamp and civil structure spring rates.

The model is further simplified, in the finite element model of Fig. X2.15, to a single composite spring with dashpot and gap element for each load direction. The composite spring stiffness is calculated from the series combination of each support element.

The snubber damping is represented by a single damper element acting in parallel with the equivalent spring. In theory, the snubber damping should act in parallel with only the snubber stiffness so the energy dissipation by the damper would coincide with the kinetic energy across the snubber. Since the energy dissipation is proportional to the square of displacement, the model damping coefficient should be equal to the square of the ratio of total stiffness to snubber stiffness multiplied by the snubber damping coefficient.

Once a composite support structure model is available for each pipe support, detailed seismic analysis of the pipe system can proceed.

X2.4 PIPELINE SEISMIC ANALYSIS

Specific considerations which must be addressed when performing seismic analyses on pipelines containing seismic snubbers are addressed in the following paragraphs.

X2.4.1 Earthquake Motion Definition. A complete discussion of the methods and techniques used to define earthquake vibratory motion is contained in App. A of this standard, and AA (14).

Earthquake vibratory motions are usually defined as surface or subsurface response spectra or as acceleration time-histories at the nuclear facility site. Before these motions can be used to analyze piping systems, they must be further modified to account for the flexibility of structures between the defined point and the attachment points for the pipe support structures. Methods of modifying earthquake motions to account for intervening structures are described in Weiner (15).
Fig. X2.14. Composite support structure model.

Fig. X2.15. Composite finite element model.
X2.4.1.1 Earthquake Response Spectra. Typical nuclear facility pipeline analyses are often conducted using linear analytical techniques with response spectra as the input seismic motion. The three components of the earthquake are defined by response spectra at the nuclear facility site. A typical example is shown in Fig. X2.16.

Critical pipelines are usually contained within a structure such as a containment building. The methods used to account for intervening structure flexibility, between the base location and the pipe support attachment points, are generally based on time-history analytical techniques. If a time-history matching the base spectrum is available, this is used; if not, an artificial time-history is generated which matches the specified base spectrum. With this time-history as input to a structural model, a response time-history is generated at the location of interest. A response spectrum can then be generated for this location, such as shown for Floor 1 in Fig. X2.17.

The calculated response spectrum will have many peaks and valleys. This response spectrum is usually linearized with an envelope, as is also shown in Fig. X2.17. As an added conservatism, the peaks of the calculated response spectra are broadened + 15% to ensure that minor shifts in system natural frequencies do not cause significant errors in predicted response (14).

Pipelines are often attached to locations with different response spectra within a structure. In this event, the applicable spectra may be overlaid and a single envelope spectrum used for input to the seismic analysis as shown in Fig. X2.18.

As an alternate, multiple spectra may be input using specialized computer codes as described in (17). However, as pointed out in (17), there is no apparent advantage for utilizing multisupport input over traditional common base envelope spectra in the extremely stiff building structures commonly used in nuclear facilities. The advantage of multiple spectra input comes when the pipe system has major differences in support motions (i.e., different buildings).

The linear response spectrum analytical technique is not explicitly capable of accounting for damping associated with individual seismic snubbers. Typically a single equivalent viscous damping coefficient is utilized in generating response spectra. This value represents the total damping available in the pipe system including snubbers, insulation, pipe contents, and pipe material effects. Guidance on selecting damping values may be obtained in Reg. Guide 1.61 (18).
Fig. X2.16. Base spectrum horizontal north-south.

Fig. X2.17. Floor 1 spectrum horizontal north-south.

Fig. X2.18. Envelope spectrum horizontal north-south.
To determine total system response for a seismic event (16) requires that the modal responses of each directional component of the earthquake be combined by the Square Root of the Sum of the Squares (SRSS) method. Any closely spaced modes are required to be combined by the absolute sum method. All three earthquake directional components must then be combined by the SRSS method.

The response-spectrum analytical technique provides an economical means of estimating support loads and stress levels in a pipe system subjected to seismic dynamic loads. However, since it is a linear technique, only linear snubber models can be incorporated into the system analytical model. The nonlinear freeplay, spring rate, and damping characteristics of the snubber usually cannot be explicitly incorporated. Evaluation of the effect of these characteristics on pipe system response generally requires the use of nonlinear time-history analytical techniques.

X2.4.1.2 Earthquake Time-Histories. In order to evaluate the non-linear snubber characteristic effects on a pipe system, it is necessary that the earthquake input be defined in the form of acceleration-time history curves. These curves are often specified along with the corresponding response spectra for the specific plant site. If they are not, artificial time histories may be generated to match the specified response spectra as described in (15) and App. B of (17).

The site base time-histories must again be modified to account for intervening structure flexibility and to develop input time-histories for the pipe support mount points. These input time-histories are then used as forcing inputs to the pipe model without further modification. Most computer codes, capable of large-scale pipe system nonlinear analysis, will accept multiple inputs for each support location, no enveloping is required, see App. D of (17).

The analysis techniques involve time-marching integration of the equations of motion, and the shape of the time-history curve is followed for each time increment. No peak broadening is employed to account for minor errors in pipe frequency prediction. For this reason, it is recommended that time-history analysis be conducted with the nominal time scale and with a modified time scale expanded and contracted 15% to ensure that peak response is identified as described in the Standard Review Plan (19).

When the maximum seismic responses for each earthquake direction are calculated independently, (16) requires that peak response be determined by combining the three maximum directional responses using the SRSS method. When the directional responses are combined algebraically at each time step, the maximum response can be obtained from the combined time solution.
X2.4.2 Pipe System Modeling. The main ingredients of the pipe system dynamic models are the (1) pipeline mass and stiffness models, (2) interfacing large component models, and (3) piping seismic support/snubber models.

X2.4.2.1 Pipeline Models. As discussed by Lin (20), the lumped mass finite element model must meet three conditions: (1) contributing natural frequencies of the lumped mass system must closely duplicate those of the continuous mass system, (2) moments calculated in the piping should at least be on the conservative side, and (3) reactions at anchors and intermediate supports should be realistic and conservative. Reference (20) provides guidance on selecting node spacing and mass distributions which will meet these conditions. Typical examples of pipeline analytical models are shown in Refs. (2) and (17).

Nominal pipe dimensions are usually used to establish pipe flexibility. Flexibility factors, such as those in Par. NB 3687 of the ASME code (21), must be incorporated for elbows and pipe fittings.

For large system models, it may be necessary to limit the number of dynamic degrees of freedom so as not to exceed computer core capacity. In this case, it is recommended that all dynamic degrees of freedom which represent stretching and contraction of straight sections of piping be eliminated. The total piping mass may be preserved by lumping the entire axial coordinate mass for each straight section at one node point.

X2.4.2.2 Interfacing Large Components. Many pipelines attach to components which are in themselves quite flexible. These components include reactor vessels, pumps, and heat exchangers. When the component natural frequency is less than the frequency at which the applicable response spectra reaches the zero period acceleration, typically 30 Hz, the component should be modeled into the pipeline analytical model.

Some large components may be satisfactorily represented with as few as 3 to 6 dynamic degrees of freedom. However, care should be taken to use a sufficient number to represent all significant vibration modes. It is sometimes prudent to perform a separate vibration modal analysis of the component, and this may lead to retention of 30 or more dynamic degrees of freedom to adequately represent the component. Care must be exercised to ensure that the component support structure spring rates are considered. For very large components, even a relatively stiff support structure can cause natural frequencies below 30 Hz.

Typical examples of large components modeled for pipeline analyses are shown in (2) and (22).

In modeling large components, the flexibility of the nozzle joining the pipe and component should also be considered. Flexibility factors for such nozzles are often available or may be derived from the detailed stress analysis.
These flexibility factors may also be approximated by assuming that the nozzles are branch connections in pipe. The flexibility factors may then be calculated using the methods outlined in (23).

X2.4.2.3 Seismic Support/Snubber Models. In the usual case, seismic analyses are conducted separately from dead weight analyses. Constant and variable spring supports are not normally included in the seismic analysis model. However, that portion of the mass of such supports which moves with the pipe should be included in the pipeline model. Strut or anchor type dead weight supports will also carry seismic loads and must be included in the analytical model along with snubbers.

In determining spring rates for these supports, the effects of the entire support train, including clamps and civil structure must be considered. Seismic snubbers are incorporated using either linear or non-linear composite support structure models as described in Par. X2.3.4.4.

When evaluating vertical support adequacy, it should be remembered that the total load on the support is that caused by the dynamic seismic loads plus any contribution from dead weight.

X2.4.3 Structural Evaluations.

X2.4.3.1 Screening Limits. Pipelines for many nuclear facilities operate at design temperatures above 800°F. These high temperature pipelines are usually under the jurisdiction of ASME Code Case N-47, which modifies the ASME Pressure Vessel Code, Section III, Division 1, to account for creep and ratchetting effects. A primary concern in the design of such pipelines is to avoid configurations which will require inelastic analyses to demonstrate adequacy.

The usual practice is to establish allowable stress screening limits, such as those developed in (24) and (25), for preliminary analysis. Allowable stress levels are established for the particular operating condition to be evaluated. Apportioned values are then selected for dead weight, pressure, thermal, and seismic induced pipe stress. The pipe configuration and support system is then modified until the calculated pipe stress meets the allowable seismic apportioned value.

In most cases, an acceptable configuration can be evolved to satisfy these screening limits. Occasionally, space restrictions will not allow the screening limit to be met and inelastic analysis will be required.

It should also be noted that the OBE seismic condition is often more critical than the SSE condition. This is due to the fact that higher damping and higher allowable stresses are usually allowed for evaluating the SSE condition.
X2.4.3.2 Linear and Nonlinear Analysis Comparison. The linear response spectrum analysis technique is widely used throughout the nuclear industry. The technique can be readily handled by most pipe analysis finite element computer codes presently in use. The method is economical to conduct and provides generally acceptable estimates of support loads and pipe stresses provided that support spring rates are adequately modeled.

The use of linear or non-linear time history analyses is much less common. Since a time-marching integration process is used, computer costs are much larger and much more complex computer codes are required. However, the time-history methods eliminate some of the conservatisms imposed on the response spectrum method, particularly, as regards the direct addition of closely spaced mode contributions and the SRSS addition of other modal contributions. Analyses conducted using time-history methods typically result in much reduced support load and smaller pipe stresses than those resulting from response spectrum analyses (2), (7), and (17). In addition, seismic test results on a prototype pipe system have also shown that non-linear time-history calculated support reactions are much closer to measured reactions than results obtained with response-spectrum techniques (26).

It is recommended that the more economical response spectrum techniques continue to be utilized for both preliminary and final design of most pipelines in nuclear facilities. Consideration for the support train flexibilities must be included. The more sophisticated and more costly time history methods should only be utilized when the increased cost is warranted to prevent extensive redesign. Typically, the time history methods have been found most useful for analysis of 8-in. diameter and larger thin-walled sodium pipelines in LMFBRs.

X2.5. GUIDELINES FOR LOCATING SUPPORTS

Preliminary selection of hanger and snubber spacing for piping is generally based on previous experience and/or general guidelines formulated by various pipe codes and architect-engineering firms. Typically, hanger locations are selected, based on these general guidelines. Once completed, a computer analysis is undertaken to demonstrate code compliance for all loadings (i.e., deadweight, thermal, seismic, etc.). If the computer analysis is successful, often no further design optimization is undertaken. This process may lead to undue conservatisms and additional hardware costs being introduced into the design. This is especially true when dealing with seismic design of piping.

This process has been generally accepted and successful in the past in the nuclear industry. However, as labor and material costs escalate, a more detailed engineering evaluation of the hanger/snubber location process is justified.
Experience with FFTF pipe design provides an example of the extent of this problem. The initial designs for high temperature piping required the installation of 4,000 seismic snubbers. During plant construction, a decision was made to replace the originally specified snubbers with a new design.

An analytical effort was undertaken to determine if the number of replacement snubbers required could be reduced. It was found that 20%, some 800 snubbers, could be deleted while still satisfying the project seismic requirements. The majority of these deleted snubbers were located on pipe 4 in. and smaller in diameter on which the original snubber spacing had been based on conservative guidelines.

Although some of these snubbers were replaced with rigid struts, most were simply deleted. The savings to the project in initial costs, installation costs, and future in-service inspection costs were substantial.

The purpose here then, is to present a seismic support spacing guideline which will stress the supported pipe to levels closer to its design limits than those methods presently in use. Use of the guideline should result in fewer seismic supports on piping while still maintaining structural integrity. A detailed derivation of the guidelines is presented in (27) along with examples and verification documentation.

X2.5.1 Support Spacing. This guideline is based on computing a maximum allowable span for a straight section of pipe. Due consideration is given to operating temperature, material allowable based on the ASME Code, dead weight, and seismic accelerations.

Given this maximum straight span, support location criteria were developed for crossover piping, concentrated load, and vertical risers. The basic piping configurations are shown in Fig. X2.19. The nomenclature used is as follows:

\[ a = \text{Distance from crossover or riser to first support, in.} \]

\[ A_s = \text{Resultant peak acceleration of horizontal and vertical response spectra using SRSS rule, g's.} \]

\[ g = \text{Acceleration of gravity, 386.4 in./sec}^2. \]

\[ L = \text{Length of crossover or height of riser, in.} \]

\[ g_{\text{max}} = \text{Maximum pipe span for straight piping, in.} \]

\[ g_{\text{max}}' = \text{Maximum pipe span for straight piping with concentrated load, in.} \]
NOTE: These Guidelines are based on a fixed support assumption which has been found to be more accurate than the usual guided cantilever assumption.

Fig. X2.19. Basic piping configuration.
$s_s$ = Seismic restraint pipe span, usually referring to $2l_{\text{max}}$, in.

$S_{\text{dwt}}$ = Maximum allowable deadweight bending stress, psi.

$S_s$ = Maximum allowable seismic stress, psi.

$w$ = Distributed load of pipe, insulation, contents, and clamp hardware, lb/in.

$W$ = Weight of concentrated load, lb.

$z$ = Section modulus of pipe, in.$^3$.

The stiffness of pipe supports can have significant impact on the magnitude of support loads. This effect has been considered in establishing these guidelines. The minimum value of support stiffness recommended for each pipe size is shown in Fig. X2.20. Higher values will not invalidate the guideline provided that pipe system natural frequencies are larger than the frequency corresponding to the peak of the pertinent pipe response spectra. Other values may also be used if detailed analysis is conducted to determine loads.

Figure X2.20 shows a maximum stiffness value for pipe supported with clamps. This limit is based on the insulated clamp design used on FFTF high temperature thin-walled pipe; other clamp designs may allow greater stiffness. The cutoff for pipe supported on structural steel frames is also based on FFTF experience and is believed to be a realistic upper bound.

It should be noted that support stiffness refers to overall effective stiffness of the support. Because of this fact, all components of the support train should be considered in computing stiffness. These components include pipe clamps, snubbers (or struts), support steel (including base plates), and any other source of significant flexibility between the pipe and a hard point. Neglecting any single component can invalidate the analysis of the piping.

X2.5.1.1 Straight Piping. Deadweight and vertical seismic restraints are positioned at distances no greater than $l_{\text{max}}$, which is computed as follows:

\[
l_{\text{max}} = \begin{cases} 
\left( \frac{12zS_{\text{dwt}}}{w} \right)^{1/2}, & A_s < 0.4735\left( \frac{S_s}{S_{\text{dwt}}} \right), \text{ in.} \\
2.384\left( \frac{s_s z}{A_s w} \right)^{1/2}, & A_s \geq 0.4735\left( \frac{S_s}{S_{\text{dwt}}} \right), \text{ in.}
\end{cases}
\]
Fig. X2.20. Recommended support stiffness.
Lateral seismic restraints are located at distances no greater than 2\(\bar{\xi}_{\max}\). Detailed derivation of these equations may be found in (27).

X2.5.1.2 Concentrated Loads. Deadweight and vertical seismic restraints are positioned on both sides of the concentrated load with the maximum span not to exceed:

\[\bar{\xi}_{\max} = \frac{1}{2} \left[ -\frac{12 \cdot WAB^2}{w} + \sqrt{\frac{12 \cdot WAB^2}{w} + 4 \cdot L^2_{\max}} \right], \text{ in.} \quad (X2.1)\]

The concentrated load should be located no further than 25% of \(\bar{\xi}_{\max}\) from one support.

Lateral seismic restraints should be located at the adjacent vertical supports on each side of the concentrated load.

X2.5.1.3 Crossover Piping. Deadweight and vertical seismic restraints are positioned at distances no greater than 85% of straight piping [i.e., \((A + L/2) < 0.85 \bar{\xi}_{\max}\)]. The straight piping requirement is applicable on the crossover leg for lengthy crossovers. Restraints should not be positioned any closer than approximately 2 ft. to the crossover (i.e., \(a \geq 24\) in.).

Lateral seismic restraints are placed at the first vertical supports on either side of the crossover. Lateral supports are also required on crossover when the crossover length \(L\) exceeds \(\bar{\xi}_{\max}\). Additional lateral supports on crossover may also be required if restraint of the piping orthogonal to the crossover is required.

X2.5.1.4 Vertical Risers. Deadweight and vertical seismic restraints are positioned at a distance from riser of:

\[a < \frac{1}{2} \left[ -L + \sqrt{L^2 + (8 \cdot S_{dwt} \cdot z)/w} \right], \text{ in.} \quad (X2.2)\]

If dimension "a" is less than approximately 2 ft., an in-line deadweight support should be provided on the riser. Vertical seismic restraints are positioned beyond the weld area of the elbow. Lateral seismic restraints are placed at the first vertical supports on either side of the riser. Lateral supports are also required on riser when the riser height exceeds \(\bar{\xi}_{\max}\).

X2.5.1.5 Additional Considerations. It should be recognized that there may exist other constraints on the placement of hangers which must be taken into consideration. These constraints will be
primarily external to the piping. For example, a vessel or pump nozzle may have load limitations that override normal hanger/snubber spacing requirements.

With these guidelines and recommendations, pipe hangers and snubbers can be located to give a more optimum design than what generally is produced by traditional means. However, pipe hanger/snubber design relies to a great extent on the experience and intuition of the designer. Because of this fact, the utilization of these guidelines will be greatly enhanced when the proper judgment and experience are present.

X2.5.2 Pipe Support Loads. Special considerations must be employed in the design of pipe systems operating at temperatures much above room temperature. Thermal expansion induces large motions at support points or, if restrained, develops large support loads and piping stresses. Such piping is often supported with variable spring or constant support hangers to prevent excessive loads. Seismic restraint is provided by snubbers which allow movement during relatively slow thermal motions but provide restraint during rapid seismic acceleration.

Both total motion and restraint loadings at these support points must be accurately determined to properly size support hardware. FFTF experience was that such piping should be analyzed using computer flexibility analyses to determine these deflections and loads.

Support load guidelines generally did not provide sufficient accuracy for this task. Those which did entailed extensive hand calculations which proved more cumbersome than performing the detailed analysis.

The FFTF Project utilized guidelines only for that piping which operated at design temperatures less than 300°F. Support loads for piping with 300°F or higher design temperatures were determined using detailed flexibility analyses.

The 300°F temperature was selected since most pipelines operating above this temperature are sodium lines with a minimum design temperature of 400°F. Those pipes with operating temperatures less than 300°F are typically argon, nitrogen water, Mobiltherm, and H&V pipes with a design temperature typically about 150°F.

In application, these guidelines were found to be very effective for 2-in. and smaller piping. The total loads induced on supports of these small pipes are relatively low and can be enveloped by a standard load value of reasonably small magnitude.

For larger size pipe, the guidelines incorporated a hand weight-balance calculation to determine deadweight and seismic loads. Thermal loads were hand calculated using a simplified method which did not account for elbow or support flexibility. Both a vertical and
horizontal contributory weight calculation were required for seismic loads. The thermal expansion analysis resulted in excessively conservative load predictions. This method proved to be both inaccurate and time consuming in application. It was found that much more accurate thermal expansion and seismic loads could be generated faster using computer programmed detailed flexibility analyses. As a result, FFTF reverted to using this detailed analysis techniques for determining support loads for all pipes with diameters larger than 2 in. 

X2.5.2.1 Two-Inch and Smaller Straight Piping. Pipe supports in nuclear power plants are often used as stepping stones by construction personnel during the plant construction phase. To avoid damage from this activity, no pipe support should be designed for a service load of less than 200 lb. Recommended design loads for pipe sizes up to 2-in. diameter are shown in Fig. X2.21. Calculations demonstrating the adequacy of these recommended load magnitudes are included in (27).

X2.5.2.2 Larger Diameter and Elevated Temperature Pipe. Support loads for pipe operating at temperatures greater than 300°F or larger than 2-in. diameter should be determined by detailed flexibility analysis.

X2.5.2.3 Concentrated Loads. Concentrated loads such as valves or flow meters cause additional deadweight and seismic support loads which must also be considered. The designer may elect to provide additional horizontal and vertical supports at these concentrated loads to eliminate this effect on adjacent pipe supports.

In this event, the seismic loads will be equal to the zero period acceleration (zpa) of the appropriate base structure spectra times the mass, provided that the support stiffness is set sufficiently large. To this should be added the deadweight of the item in the vertical direction and the horizontal and vertical seismic loads from the attached pipe.

If supports are not provided on the concentrated mass, the reduced span approach of Par. X2.5.1.2 may be used. The fundamental frequency of the reduced span may then be calculated. The spectral acceleration factor, at this frequency, can then be determined from the appropriate horizontal or vertical response spectra. The mass can then be multiplied by these accelerations, times a 1.5 factor to account for higher modes, to determine the seismic loads. Both the deadweight and seismic loads may then be distributed to the adjacent supports using simple statics.

X2.5.2.4 Crossovers. Crossovers will induce additional in-plane seismic loads in a direction parallel to the crossover centerline. This load should be treated as above under concentrated load and should be added to the loads on the first two supports adjacent to the crossover shown in Fig. X2.22.
Fig. X2.21. Support loads - 1 and 2 in. φ pipe and gas or fluid filled to 150°F.

Fig. X2.22. Crossovers.
X2.5.2.5 *Vertical Runs.* Vertical runs will also induce additional deadweight and vertical seismic loads. These loads must be added to the adjacent vertical supports in a manner identical to that used for crossovers.

X2.5.2.6 *Additional Sizing Considerations.* Support loads determined by linear and non-linear analysis techniques are dependent on the natural frequencies and modes of the piping system and their relation to the frequency distribution of the input seismic motions. Tolerance effects on pipe section properties, insulation density and support flexibility can cause minor shifts in the pipe system modes and natural frequencies. These shifts are generally small and economic considerations do not often allow detailed analyses of all possible tolerance combinations.

A well-designed piping system will have a fairly uniform load distribution on seismic supports in a given direction. Design loads for these supports can be increased by some constant factor to account for these tolerance effects. Other factors, such as interference problems, may preclude a uniform spacing of seismic supports. In this event, a wide variation in seismic loads can result.

The FFTF experience indicates that tolerance effects do not cause a large variation in the higher loads, but can cause substantial changes in the smaller loads. Such variation can occur when the higher load is the result of a natural frequency at or near the peak of the response spectra while the lower load is the result of a frequency near the spectra zpa or a location that is near a "natural node" of one or more of the significant modes. Tolerance shift will have little effect on the large load but may have a significant effect on the small load. As an example, a 1,000 lb variation in a 20,000 lb load is not substantial but a 1,000 lb variation in a 1,000 lb load may cause failure.

For this reason, it is strongly recommended that all the support hardware be sized for no less than one-fourth the average load in those systems that exhibit a wide variation in calculated seismic loads. As a further example, one or two supports rated at 5 kips, along with many supports rated at 15 kips and one or two 50 kips supports would be acceptable for a large diameter (24-in.) pipeline. However, no 1 or 2 kip supports should be used.

It is also recommended that every effort be made to eliminate any support with exceptionally low loads in such a system. This can usually be accomplished by respacing the supports.

For the 1- and 2-in. pipe sizes, use of the recommended design loads of Par. X2.5.2.1 will result in seismic support loads of constants. Load capability. Adequate margin is included to cover tolerance effect.
REFERENCES


APPENDIX - NONMANDATORY

X3. SOIL/STRUCTURE INTERACTION ANALYSES

X3.1 SCOPE

This appendix provides recommended guideline procedures that are applicable to the soil/structure interaction analysis for nuclear power plant structures.

The basic problem of soil/structure interaction involves the determination of the motions of one or more structures at a given site from the knowledge of a given motion (control motion) at a specified point (control point) of the site prior to construction (free field). Soil/structure interaction is a complex phenomenon leading to the conclusion that the problem cannot be solved exactly; however, acceptable predictions are possible.

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X3.2 SUMMARY

The main elements of a typical soil/structure interaction (SSI) analysis are the description of the methods of analysis, modeling techniques, and the calculation of the system response. Guideline procedures which address these topics are presented in the sections that follow.

X3.3 CHARACTERISTICS OF ANALYSIS TECHNIQUES

Available analysis techniques for handling soil/structure interaction problems can be broadly classified on the basis of the type of information desired: (1) deterministic methods in which specific response values are calculated for a particular seismic excitation, or (2) probabilistic methods in which statistical estimates of the response under a (theoretically) infinite ensemble of seismic excitations are determined.

X3.3.1 Deterministic Methods. Within the class of deterministic methods, various techniques can be classified according to the method of approach (Fig. X3.1).

a. Direct methods and
b. Substructure methods

X3.3.1.1 Direct Methods. In this approach, a set of free-field ground motion is applied to the subsurface boundaries of a discrete model, and then the response of the combined soil/structure system is computed. Discretization techniques can involve finite element or finite difference approaches of any desired degree of sophistication,
although finite element methods are much more widely used in current practice. The responses of the soil as well as that of the structure are determined simultaneously. Due to practical cost and time considerations, only the overall dynamic behavior of the structure is typically determined in the direct method of analysis. A subsequent structural analysis utilizing a refined model subjected to soil interface loadings computed from the soil/structure interaction analysis must be performed if detailed structural response information is needed. Figs. X3.1(a) and X3.1(b) correspond to the direct method.

X3.3.1.2 Substructure Method. Unlike the direct method, where the dynamic response of the soil and structure is handled in one step, this approach reduces the soil/structure interaction problem to the solution of a sequence of three simpler problems [(1) solution of the kinematic interaction problem, (2) calculation of the foundation impedance functions, and (3) solution of the coupled soil/structure system] and then superposing the resulting solution to obtain the total solution. This method offers considerable cost and time savings over finite element methods when fully three-dimensional analyses are required; however, due to a reliance on the principle of superposition, only systems that can be adequately modeled as equivalent linear systems can be handled by this approach. This method has not been widely used for carrying out actual SSI analyses for nuclear plant structures except as a check on other solutions in some special cases. It may be more widely used in the future, however, when current research efforts directed toward incorporating embedment effects for arbitrarily shaped foundations are completed. Figure X3.1(c) corresponds to the substructure method.

X3.3.2 Probabilistic Method. Of the various methods that can be used to calculate the seismic response of structural systems, those based on random vibration techniques offer the most rational approach for handling the response of soil/structure systems under stochastic excitations. However, due to the mathematical difficulties that arise in the solution of complex SSI problems under nonstationary random excitation, the state of the art of random vibration analysis methods is such that numerous simplifying assumptions (including many assumptions of dubious validity) have to be made before an approximate probabilistic solution can be obtained.

Due to the different limitations on the range of validity of the analysis methods discussed above, it is recommended that, unless compelling reasons are presented to justify the use of other methods, the direct approach discussed above should be used at this time to analyze SSI problems that arise in the design of nuclear power plant systems. Further details that form the basis of this conclusion are given in the recent AA (5)* report.

*The numbers in parentheses refer to the list of references at the end of this appendix.
X3.4 MODELING OF SOIL/STRUCTURE SYSTEMS

The deformational theories to be used in the modeling of the soil/structure system can vary from relatively simple linear two-dimensional models to general nonlinear three-dimensional models of the soil and the structure. The following deformational formulations are commonly used in SSI analyses:

a. Plane-strain formulation, which is suitable for modeling structures whose dimension normal to the plane of the model is very long.

b. Axisymmetric formulation, which can be used if the structure as well as the soil has axisymmetric geometry and material properties, and if the loads are separable into Fourier harmonics expanded in the circumferential direction [Fig. X3.2(a)].

c. Pseudo-three-dimensional formulation which uses conventional two-dimensional plane-strain formulation that is augmented by out-of-plane viscous dampers acting in the third dimension to simulate the three-dimensional effects of radiation damping [Fig. X3.2(b)].

d. General three-dimensional formulation which can handle arbitrary geometries for the soil and structure including their non-linear properties.

It should be borne in mind that regardless of which formulation is used, care must be exercised in using a refined second-stage analysis for the detailed three-dimensional response of the structure to two uncoupled horizontal excitation components and one vertical component.

On the basis of cost/benefits ratios, it is recommended that for cases of SSI analysis that are not research oriented, the SSI investigation should use a 2-D or axisymmetric formulation.

X3.4.1 Soil Configuration and Behavior.

Soil configurations have significant effects on the response of soil/structure systems. Among the commonly used configurations are:

a. The uniform elastic or viscoelastic half-space, which is an appropriate model for a foundation resting on a relatively thick and uniform soil layer.

b. A single layer of soil of uniform thickness and infinite horizontal extent overlaying a stiff uniform half-space.

c. A sequence of uniform elastic or viscoelastic layers of uniform thickness and infinite horizontal extent.
(a) Schematic view of an axisymmetric model.

(b) Schematic view of a simplified 3-dimensional model.

Fig. X3.2. Some soil-structure interaction formulations.
d. A discrete model (usually finite element) of a soil profile of arbitrary topography and elastic, viscoelastic, or inelastic soil layers of arbitrary orientation and geometry.

When using the direct method for SSI analyses, a common but unnecessary assumption, particularly for plane-strain or pseudo-three-dimensional representation, is that the soil configuration consists of horizontally stratified soil layers.

In dealing with the direct method of analysis, two approaches are available to analyze a SSI system with nonlinear soil properties:

a. Equivalent linear analysis involving the iteration of the properties of a linear viscoelastic material model characterized by two elastic parameters and a damping factor.

b. Nonlinear analyses using suitable nonlinear constitutive soil models. Such procedures are still in a research and development stage and are not appropriate for structural design purposes.

Since the actual state of stress in the soil under earthquake excitation is three-dimensional, the generation of three-dimensional nonlinear constitutive models is an active area of research. The nonlinear models that are currently most widely used include:

a. The cap model, which is a continuum model based on the classical incremental theory of plasticity.

b. The multisurface plasticity model that simulates hysteretic effects.

c. The endochronic model, an alternative to the plasticity models, that uses intrinsic time as a basis for measuring the memory of past deformation history.

X3.4.2 Foundation Properties. For soil/structure interaction, the main characteristics of concern in modeling foundations are the configuration, mass, and stiffness properties and their proximity to one another.

A summary of equivalent spring constants, mass ratios, and damping ratios for rigid foundations and for embedded rigid circular foundations on elastic half-space is given in Tables X3.1 and X3.2. The values in these tables are useful in the evaluation of the SSI effects of different foundation shapes.

Actual nuclear power plant structures possess extremely complicated geometries whose authentic detailed three-dimensional representation while theoretically possible, involves prohibitive computer costs when finite element direct methods of analyses are used. Therefore, simplified two-dimensional representations of these structures are typically
Table X3.1. Equivalent stiffness and damping constants for embedded rigid circular foundations (as developed from theory of Novak and colleagues).

<table>
<thead>
<tr>
<th>Vibration</th>
<th>Component</th>
<th>Stiffness and Damping Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical translation:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right)$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}} \cdot \sqrt{\frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2} \cdot \sqrt{\frac{e_3}{c_3}}}}$</td>
</tr>
<tr>
<td>Uncoupled horizontal</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right)$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}}$</td>
</tr>
<tr>
<td>Uncoupled rocking</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right) + \frac{c_0}{\sqrt{\frac{\rho}{G_0}}} \cdot \frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}} \cdot \sqrt{\frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2} \cdot \sqrt{\frac{e_3}{c_3}}}}$</td>
</tr>
<tr>
<td>Coupled horizontal-rocking</td>
<td>$k_{xx} = k_{xx} + c_0 \sqrt{\frac{\rho}{G_0}} \cdot \sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
<td>$c_{xx} = c_0 \sqrt{\frac{\rho}{G_0}} \cdot \sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
</tr>
<tr>
<td>Rocking rotation:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rocking rotation</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right)$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}}$</td>
</tr>
<tr>
<td>Uncoupled torsion</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right) + \frac{\rho}{G_0} \cdot \frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}} \cdot \sqrt{\frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2} \cdot \sqrt{\frac{e_3}{c_3}}}}$</td>
</tr>
<tr>
<td>Coupled rocking-torsion</td>
<td>$k_{xx} = k_{xx} + \rho \cdot \frac{e_1}{c_1} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
<td>$c_{xx} = c_0 \sqrt{\frac{\rho}{G_0}} \cdot \sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}$</td>
</tr>
<tr>
<td>Torsional rotation:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Torsional rotation</td>
<td>$k_{xx} = G_0\left(\frac{e_1}{c_1} \cdot \frac{e_2}{c_2} \cdot \frac{e_3}{c_3}\right)$</td>
<td>$c_{xx} = \frac{\sqrt{\frac{\rho}{G_0}}}{\sqrt{\frac{e_1}{c_1}} \cdot \sqrt{\frac{e_2}{c_2}} \cdot \sqrt{\frac{e_3}{c_3}}}$</td>
</tr>
</tbody>
</table>

**Note:**
1. Soil material properties are defined as follows:
   - $\rho, G$ = mass density and shear modulus of soil layer along sides of foundation
   - $\rho, G$ = mass density and shear modulus of soil beneath base of foundation
2. $\alpha$ = embedment ratio $= \frac{h}{D}$
Table X3.2. Summary of equivalent spring constants, mass ratios, and damping ratios for rigid foundations on elastic half-space.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Soil</th>
<th>Spring Constants</th>
<th>Damping Values (Circular footing only)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m = Mass</td>
<td>Circular Footing</td>
<td>Rectangular Footing*</td>
</tr>
<tr>
<td>Vertical</td>
<td></td>
<td>$k_x = \frac{4 G r}{1 - v}$</td>
<td>$k_x = \frac{G}{1 - v} 8 \sqrt{k cd}$</td>
</tr>
<tr>
<td>Horizontal</td>
<td></td>
<td>$k_y = \frac{32(1 - v) G r}{1 - v}$</td>
<td>$k_y = k(1 + v) G b \sqrt{cd}$</td>
</tr>
<tr>
<td>Rocking</td>
<td></td>
<td>$k_z = \frac{8 G r^2}{3(1 - v)}$</td>
<td>$k_z = \frac{G}{1 - v} 8 \sqrt{b cd}$</td>
</tr>
<tr>
<td>Torsion</td>
<td></td>
<td>$k_\theta = \frac{16 G r^3}{3}$</td>
<td>—</td>
</tr>
</tbody>
</table>

*Note: For rectangular footing, the quantities: $\beta_x$, $\beta_y$, and $\beta_z$ are given in terms of the aspect ratio $d/c$ in the curves shown below. (Richart and Whitman, 1967)
used in current practice, although neither systematic analytical or experimental studies have been conducted to the degree necessary to evaluate the effects of SSI analysis. The nature of these simplifications requires careful evaluation on a case-by-case basis.

When representing three-dimensional foundations by two-dimensional models, care should be exercised in selecting an appropriate cross section. No systematic procedure is currently available to select an equivalent cross section for foundations with complicated geometrical shapes.

X3.4.3 Structural Models. Structural models used in SSI analysis can be relatively crude and designed to represent only the dominant features of the structural dynamic response. This phase of the analysis should be followed by another dynamic analysis in which a detailed 3-D model of the structure is subjected to one vertical component and two uncoupled horizontal components of foundation motion resulting from the earlier SSI analysis.

Modeling of complicated multi-degree-of-freedom structural systems, particularly if they are nonlinear, requires careful attention in order to obtain valid results. Where appropriate test data are available, modern methods of system identification should be used in determining the significance of various modes and phenomena that should be included in an appropriate model.

X3.5 CONSIDERATIONS THAT ARISE IN THE CALCULATION OF THE RESPONSE

X3.5.1 Specification of Free-Field Ground Motion. Factors that define earthquake motions at a site are the size and location of the earthquake, its transmission path, and local site properties. The usual procedure for characterizing earthquake ground motion is to specify the earthquake intensity, spectral characteristics, and duration.

Because of computational convenience, it is common to assume that the wave propagating mechanism at a site consists of vertically incident P- and SH-waves. Such an assumption leads to (1) vertical motion that is entirely transmitted by P-waves, (2) horizontal motion that depends only on SH-waves, (3) all points on the soil surface undergo identical free-field deformations, and (4) no torsional component of the input motion is generated. This assumption may not be conservative, particularly for very stiff sites and significant surface wave components.

When using the direct method for SSI analysis, a reference (control) motion and a reference (control) point must be specified, in addition to the free-field ground motion which must be specified at the boundaries of the model. The location of a control point should be a free surface and should be either (a) an assumed outcropping at depth or (b) at the surface of the soil.
X3.5.2 Solution Procedure. Analytical solution techniques can be separated into three basic groups of methods: (1) modal analysis, (2) direct integration, and (3) frequency-domain. In the modal analysis methods, a choice has to be made between response-spectrum and time-history analysis. It is noted that modal or frequency-domain methods of analyses are valid for equivalent linear systems only, whereas direct integration methods can be used either for linear or nonlinear systems.

X3.5.3 Mesh Size. The mesh size used in discrete mathematical models (whether finite element or finite difference) affects the solution in a way similar to the effects of time or frequency increments. For accurate reproduction of a frequency of interest in the solution domain, the mesh size must be sufficiently small; however, if the mesh size is excessively small, prohibitive computer costs will result. Decisions regarding the optimum mesh size should adhere to the following recommendations:

a. The mesh size should not exceed about one-seventh of the shortest wave length of interest.

b. The mesh size near the edge of the foundation should be smaller than those in the middle.

c. Higher derivatives of the response, whether spatial or temporal, require a finer mesh if they are to be calculated with the same accuracy as the basic response quantity.

d. A mesh size of at least ten elements over the width of the foundation mat is needed for accurate calculation of stress distribution under the structure.

e. If the primary response is due to the vertical propagation through the soil of P-waves or shear waves, it is acceptable to keep the height of the soil elements constant but to change their horizontal size.

f. When, for the sake of economy, a fine mesh around the foundation is enlarged as the distance from the foundation increases, the change in mesh size should be gradual.

g. When dealing with general wave fronts, or if both the horizontal as well as the vertical dimensions of the mesh size are changed, calibration checks should be conducted on the accuracy of the results over the frequency domain of interest.

X3.5.4 Size of Soil Region. The location of soil boundaries in a SSI analysis using a finite element direct method of analysis is influenced by static (e.g., pressure under foundation) and dynamic (e.g., wave reflection at artificial boundary) effects. It is noted that for substructure methods, radiation damping is represented exactly;
i.e., only for finite element methods does the possibility of unwanted wave reflections from artificial boundaries of the soil grid require careful assessment of the locations of these soil grid boundaries. For accurate results the following rules should be followed:

a. The dimension of the soil core region should be such that viscous or elementary boundaries are placed at a distance of 5 to 10 radii (or a half-foundation width) from the foundation edge.

b. The location at the bottom boundary should coincide with a sharp transition in the soil properties.

c. Unless rock is present at a shallower depth, the lower boundary should be at least 2 to 4 radii below the foundation.

X3.5.5 Integration Time Step. Time-wise integration schemes used in direct methods of analysis can be conveniently separated into two basic groups:

1. Implicit methods, due to their unconditional stability, can tolerate a relatively larger time step.

2. Explicit methods which require a relatively small time step for stable solution, but which do not involve the inversion of large matrices at every time step.

As in the case of mesh size, care should be exercised in selecting the time step size so that it is small compared to the shortest period (highest frequency) of interest. Normally, when the time step is less than one-fourth the shortest period of interest, satisfactory results are obtained. Careful attention should be directed to the selection of time step size and mesh size in nonlinear systems to assure that true components of motion are not filtered out.

X3.5.6 Frequency Specifications. When using SSI solution procedures that employ the frequency domain approach, the frequency range used should be sufficiently wide to encompass the significant frequency components of both the excitation and response. Transfer function calculations should use at least 150 frequency points, and Fourier transform calculations should use at least 2048 frequency points. Typically it is satisfactory to limit the upper bound of this frequency range to 20 Hz.

X3.5.7 Damping Values. Energy losses that must be considered include material damping and hysteretic losses within the structure or in the surrounding soil materials, frictional losses at structure connections, equipment mounts or at soil/structure interfaces; separation of the soil and structure along their interface; and the radiation of energy away from the structure foundation and into the surrounding soil
(radiation damping). The magnitudes of these energy losses depend on such factors as the amplitude and frequency content of the applied excitations, operating temperatures, material properties, site conditions, soil/foundation interface condition, and configuration and design details of the nuclear plant.

For purposes of computational expediency, the various energy losses in nuclear plants are typically represented in dynamic analyses as equivalent viscous (i.e., relative velocity dependent) damping factors. However, the implications of this type of approximation, and the basis for selection of equivalent viscous damping factors for purposes of design, must always be carefully evaluated. Detailed discussion of this topic is available in Agbabian Associates (4).

X3.5.8 Excitation Characteristics. When using random vibration techniques, the format of the input should include a probabilistic characterization of the loads expressed in terms of appropriate statistical quantities that represent the earthquake loads as nonstationary random processes.

If a deterministic analysis is to be conducted, then for the cases that need a time history of the excitation, available computer codes should be used to generate an artificial earthquake record compatible with any specific design spectra.

It is usually assumed that the wave propagation mechanism at a site consists of vertically incident P- and SH-waves. The significance of nonvertically incident propagating waves on the structural response depends on the nature of the earthquake motion, characteristics of the site, and design of the structure. At present, no general guidelines are available to assess a priori the importance or unimportance of this phenomenon. Most SSI analyses are based on the assumption that the input consists of vertically propagating waves.

For solution procedures that use the design response spectra directly, the excitation is completely specified by the design spectra that are applicable to the site.

X3.5.9 Soil Properties. Accurate representation of realistic soil configurations encountered in SSI analyses should account for the nonlinearity of soil properties as well as the three dimensionality of the stress/strain relationship in a manner consistent with the type of data generated by the field and laboratory soil tests.

X3.5.10 Structure-Structure Interaction. If the separation of some structures is relatively small, then reliable estimates of the extent of coupling through the soil in the dynamic response between the closely spaced structures requires 3-D analysis. Simplified 2-D analyses may over-emphasize this effect.
X3.5.11 Geometrical Analysis. Commonly used deformational formulation in SSI analysis are: (1) plane-strain, (2) axisymmetric, (3) two-dimensional, (4) pseudo-three dimensional, and (5) general-three dimensional. Details regarding the appropriateness and limitations of each of these formulations are available in reports SAN-1011-111R (3) and DOE/SP/01011-130 (4).

The state of the art of computational techniques is such that for conventional SSI analysis (i.e., not of a research nature), a 2-D or axisymmetric formulation will yield the optimum cost/benefit ratio.

X3.5.12 Boundary Conditions. Due to the inherent limitations of finite element models to generally represent a semi-infinite medium, there is a need to eliminate the physically incorrect effects of artificial boundaries along the sides and bottom of the discretized model of finite dimensions. This is usually accomplished by introducing boundary elements that tend to eliminate the reflection caused by regular boundaries, with the aim of making the response of the soil at distances that are far from the structure essentially the same as the free-field conditions in the physical system.

It is common practice to assume that the vertical radiation of energy below the bottom fictitious boundary is negligible. In dealing with boundary conditions involving the lateral boundaries, several types of mathematical boundaries are available: (1) elementary, (2) viscous, and (3) consistent and transmitting.

Further details regarding this subject are available in (2).

X3.5.13 Inclined Rock Strata. Only limited information is currently available regarding the effects of inclined rock strata on the site response analyses of nuclear power plant sites. In a recent study (1) two rock site examples were considered; in one example, the sequence of layers of sedimentary rocks were horizontal, while in the other, example of layers were inclined (i.e., dipping). On the basis of this study it was concluded that (1) ground surface records can adequately represent vertical interior motions at the foundation level, and (2) that horizontal ground surface records far exceed the corresponding response at the foundation for these sites.

X3.5.14 Nonlinearities. Sources of nonlinearities encountered in SSI analysis can be conveniently divided into two groups: (1) material and (2) geometric. Material nonlinearities include those associated with the soil and structure, while geometric nonlinearities involve phenomenon such as debonding between soil and structure, impacts between adjacent structures or elements, sliding of structures, etc. Nonlinearities may be distributed throughout the solution domain (e.g., soil properties) or be localized (closing of gaps at a certain point).
The area of nonlinear dynamic analysis and characterization of nonlinear processes is an active area of research in which numerous practical problems still await adequate treatment.

X3.6. AVAILABLE COMPUTER CODES

A list of commonly used computer codes for SSI analyses together with their basic features are given in (4).

Computer codes that perform linear or equivalent nonlinear analyses include (1) SAP, (2) FLUSH, (3) CLASSI, (4) STARDYNE, (5) TRI/SAC, (6) STRUDL, and (7) CAST1. Codes capable of performing nonlinear analyses include (1) ANSYS, (2) TRANAL, (3) MARC, (4) NASTRAN, (5) STEALTH, (6) NONSAP, and (7) ADINA.

Major features of nonlinear and linear computer codes are summarized in Tables X3.3 and X3.4. Further details regarding this subject may be found in Werner and Huang (5).

A code for performing a probabilistic SSI analysis on linear or nonlinear systems that can be treated as piecewise linear is PLUSH.

Codes that are acceptable, providing their limitations are observed, are CLASSI and CAST1.

REFERENCES


Table X3.3. Nonlinear finite element analysis procedures.

<table>
<thead>
<tr>
<th>Items for Comparison</th>
<th>ANSYS (Bathe, 1979)</th>
<th>TANAL (Belytschko et al., 1979)</th>
<th>MARC (Marc, 1979)</th>
<th>NASTRAN (Marc, 1979)</th>
<th>AQUA (Bathe, 1978)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static Analyses</td>
<td>Linear, thermal, plastic, buckling, creep Nonlinear material properties and geometry</td>
<td>None</td>
<td>Linear, thermal Nonlinear material properties and geometry</td>
<td>Linear, buckling, thermal Nonlinear material properties and geometry</td>
<td>Linear, thermal Nonlinear material properties and geometry</td>
</tr>
<tr>
<td>Eigenvalues and Eigenvectors</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes (6 methods, restartable, complex roots)</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>Transient</td>
<td>Explicit</td>
<td>Implicit</td>
<td>Explicit</td>
<td>Explicit</td>
</tr>
<tr>
<td></td>
<td>Harmonic response</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Element Types</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Flat and Flat Shell</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Curved Shell</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Sphere</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
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<tr>
<td>Two-Dimensional</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
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<tr>
<td>Special</td>
<td>Pipe and fluid elements</td>
<td>No</td>
<td>Concrete pipe and fluid elements</td>
<td>Substructures Pipe and fluid elements</td>
<td>Fluid elements</td>
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<td>Loading</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model Point</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Member</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Gravity</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Initial Stress/Strain</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Kinematic Boundary Conditions for Dynamic Analysis</td>
<td>Displacement, velocity, and acceleration</td>
<td>Displacement</td>
<td>Displacement</td>
<td>Displacement, velocity, and acceleration</td>
<td>Displacement, velocity, and acceleration</td>
</tr>
<tr>
<td>Maximum Number of Nodes Points</td>
<td>Dynamically allocated (D.A.)</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
</tr>
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<td>Maximum Number of Elements</td>
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<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
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</tr>
<tr>
<td>Maximum Half-Bandwidth</td>
<td>Wavefront technique</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
</tr>
<tr>
<td>Maximum Number of Load Cases</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
</tr>
<tr>
<td>Maximum Number of Materials</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
</tr>
<tr>
<td>Maximum Number of Cross Sections</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
<td>D.A.</td>
</tr>
<tr>
<td>Graphic Output</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strain or Strain Plot</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Mode Shapes</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Time History</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Response Spectra</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Contour Plot</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Automatic Mesh Generation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bandwidth Minimization</td>
<td>Wavefront technique</td>
<td>Not applicable</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Constrained DOF (Steering)</td>
<td></td>
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<tr>
<td>Special Features</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Early conversion of cartesian to cylindrical or polar coordinates</td>
<td>Bonding and debonding capability</td>
<td>Extensive output graphics</td>
<td>Sparse matrix methods</td>
<td>Extensive output graphics</td>
<td>Executive control</td>
</tr>
</tbody>
</table>
Table X3.4. Nonlinear and equivalent linear computer codes used for SSI analysis.

<table>
<thead>
<tr>
<th>Code</th>
<th>Soil Properties</th>
<th>Type</th>
<th>Element Types</th>
<th>Large Strain Capability</th>
<th>Energy-Absorbing Boundary</th>
<th>Seismic Input</th>
<th>Strongly Nonlinear Interface</th>
<th>3-D Effects</th>
<th>Structure-Structure Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRAMAL</td>
<td>Nonlinear</td>
<td>Explicit, Time Domain, Finite Element</td>
<td>Continuum</td>
<td>Yes</td>
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<td>General</td>
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<td>Yes</td>
<td>General</td>
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<td>No</td>
<td>General</td>
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<td>No</td>
<td>General</td>
<td>Yes</td>
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<td>FLUSH</td>
<td>Equivalent, Linear</td>
<td>Implicit, Frequency Domain, Finite Element</td>
<td>Continuum, Structural</td>
<td>No</td>
<td>Yes</td>
<td>Rigid Bedrock Shaking, Vertically Propagating Body Waves</td>
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<td>No</td>
<td>Yes</td>
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<td>Yes</td>
<td>General</td>
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<td>Yes</td>
<td>Yes</td>
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APPENDIX - NONMANDATORY

X4. COMPOSITE MODAL DAMPING

X4.1 SCOPE

This appendix provides recommended guidelines on the methods used to calculate modal damping when individual components and substructures of a nuclear power plant exhibit pronounced differences in component damping.

Since the late 1960s a number of papers on composite damping have appeared in the literature. A number of approximate approaches to the problem have been proposed. No single method has emerged as the obvious best solution. In Standard Review Plan 3.7.2 (1)*, the Nuclear Regulatory Commission lists two acceptable methods for treating "element-associated damping." Appendix N of Section III of the ASME Code (2) discusses several approaches for treating "subregional modal damping" but states no position on a preferred method.

Due to terminology inconsistencies in the literature, it is important to clearly define the phrase "composite modal damping" as it is used in this appendix. The term "composite" refers to systems composed of subsystems or components which may have unique damping properties. The available methods for treating composite damping utilize mode frequency analyses which explain the term "modal." It also implies a limitation to linear systems. Thus, composite modal damping can be defined as the analytical treatment of damping within a linear system when the damping varies in magnitude at the subsystem level. Using this definition, all of the approaches mentioned in the previous paragraph could be considered to be composite modal damping methods.

This appendix has been prepared by Hanford Engineering Development Laboratory for the Materials and Structures Program sponsored by the Department of Energy, Office of Breeder Technology Projects.

X4.2 SUMMARY

Damping in actual structures is complex and is seldom precisely known. An analyst faced with obtaining the seismic response of a particular structure is typically provided with a tabular set of equivalent viscous damping values expressed in terms of percent of critical damping. These values generally have some experimental basis and usually are expected to be on the conservative side.

Considering the imprecise nature of damping and the approximations involved in developing the seismic ground motion, a highly rigorous

*The numbers in parentheses refer to the references listed at the end of this appendix.
treatment of component varying damping is seldom warranted. In the majority of cases involving variable damping, it is probably adequate to assume a uniform damping equal to the minimum of the range of values. If bounding solutions are desired, the maximum damping solution can also be obtained.

There are occasions when a composite modal damping approach may be highly beneficial. In Sec. X4.8, an example is given where a large reduction in seismic response is obtained by considering variable damping vs a uniform minimum damping. If composite damping is built into a dynamic analysis computer program, very little extra effort may be required to include variable damping. Also, if a conservative uniform damping approach fails to meet specified limits, additional runs may be required.

Another situation with large payback potential is a generic analysis which will be providing input to a number of subsequent analyses. A good example of this situation is a soil-structural model which is used to develop floor spectra for piping and components.

Various composite modal damping methods are presented and compared in Secs. X4.5 and X4.7. All methods involve some approximations. No evidence was found which indicated that any of the presented methods should obviously be eliminated from further consideration. However, based upon the conclusions presented in Sec. X4.7, the weighted energy ratio methods described in Sec. X4.5.1 are preferred. The weighted energy methods are simple to apply and require no judgment decisions by the analyst. They also performed well in composite damping and seismic response predictions.

X4.3 THE NATURE OF DAMPING

Damping can be defined as the means by which the response motion of a structural system is reduced as the result of energy losses. Proper accounting for this reduced response can be very practical since a small increase in damping can potentially produce large reductions in the seismic stresses. However, damping in real structures is very complex and a precise analytical treatment is generally not feasible. A good summary of the current state-of-the-art of damping fundamentals is found in Nelson and Grief (4).

Energy loss mechanisms in structures are generally categorized into (1) material damping, associated with hysteresis losses during stress cycling; (2) Coulomb damping due to dry friction between sliding surfaces; (3) impact damping, usually associated with the dynamic closing of a gap; and (4) viscous damping due to vibrations in fluid or time-dependent material resistance. Energy dissipation in vibrating structures generally involves a combination of mechanisms and a precise definition is difficult. For mathematical convenience, damping is usually assumed to be viscous in nature and is expressed as a percent of critical damping.
Typical equivalent viscous damping values are given in Table X4.1 (Table 1 on the main body of this standard). Most of these damping coefficients originated in Reg. Guide 1.61 (8). Note that stress amplitude dependency is incorporated by allowing higher damping for an SSE than for an OBE. In this simple indirect fashion, the stress dependency of material (hysteretic) damping is incorporated into the equivalent viscous values. Some authors prefer to separate damping mechanisms in an analytical model. For example, in Rossett et al. (5) a viscous damping is assumed for soil and hysteretic damping for structure in a soil/structure model.

Table X4.1 Equivalent viscous damping coefficients

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Percent of Critical Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OBE</td>
</tr>
<tr>
<td>Reinforced concrete structures and foundations\textsuperscript{A}</td>
<td>4.0</td>
</tr>
<tr>
<td>Prestressed concrete structures\textsuperscript{A}</td>
<td>2.0</td>
</tr>
<tr>
<td>Steel structures\textsuperscript{A}</td>
<td></td>
</tr>
<tr>
<td>Welded steel structures</td>
<td>2.0</td>
</tr>
<tr>
<td>Bolted or riveted steel structures</td>
<td>4.0</td>
</tr>
<tr>
<td>Piping systems\textsuperscript{A} (welded)</td>
<td></td>
</tr>
<tr>
<td>Small diameter</td>
<td>1.0</td>
</tr>
<tr>
<td>Large diameter [\textgreater\textgreater 12 in. (0.3048 m)\textsuperscript{nominal diameter}</td>
<td>2.0</td>
</tr>
</tbody>
</table>


\textsuperscript{B}In the dynamic analysis of active components as defined in Reg. Guide 1.48, these values should also be used for SSE.
The experimental basis for the currently recommended equivalent viscous damping coefficients is discussed in Stevenson (6). It should be understood that the damping values prescribed in Reg. Guide 1.61 are interim values. These values are generally considered to be conservative. The author in (6) anticipates some increase in these values as more data become available.

It is emphasized that the complex nature of damping in real structures does not lend itself towards precise treatment. This point should be kept in mind when determining the degree of rigor desired when the added complexity of variable damping is considered.

X4.4 DYNAMIC EQUILIBRIUM EQUATIONS

To provide a basis for understanding the composite modal damping methods, a summary of the fundamental equations of motion is presented in this section. Considerable variation in notation exists in the literature. For this appendix, a notation consistent with the methods evaluation report (2) is used.

The basic dynamic equilibrium equation for an elastic multi-degree-of freedom system can be expressed as:

\[ [m] \{\ddot{u}\} + \{d\} + [k]\{u\} = \{p\}, \quad (X4.1) \]

where

- \([m]\) = mass matrix,
- \([k]\) = stiffness matrix,
- \([d]\) = damping factor vector,
- \([u]\) = displacement vector,
- \([\ddot{u}\]) = acceleration vector,
- \([p]\) = applied force vector,

The damping force vector, \([d]\), can take many forms depending upon the assumed damping mechanism. Assuming viscous damping,

\[ [d] = [c] \{\dot{u}\} \quad (X4.2) \]

where \([c]\) is the viscous damping matrix and \([\dot{u}\]) is the velocity vector. Alternative forms of \([d]\) can be found, for example, in Appendix N of (2).

If the damping is light, the eigenvalues (natural frequencies), \(\Omega_r\), and eigenvectors (natural modes), \([\phi]\), can be found with reasonable accuracy by considering the undamped eigenvalue problem associated with Eq. (X4.1).
If $\Omega^2$ is defined as a diagonal matrix of the squared natural frequencies and $[\phi] = \{[\phi]_1, [\phi]_2, \ldots [\phi]_n \}$, then the coordinate transformation

$$\{u\} = [\phi] \{q\} \quad \text{(X4.3)}$$

converts Eq. (X4.1) into

$$\{\ddot{q}\} + [\phi]^T [c][\phi]\{\dot{q}\} + [\Omega^2]\{q\} = [\phi]^T \{\dot{p}\} = \{p\} \quad \text{(X4.4)}$$

where $[c]$ is defined by Eq. (X4.2) and the eigenvalues are normalized so that

$$[\phi]^T [m][\phi] = 1. \quad \text{(X4.5)}$$

If the damping matrix $[c]$ is diagonalized by the undamped normal modes (i.e., $[\phi]^T [c][\phi]$ is a diagonal matrix), then the damping matrix $[c]$ is termed "proportional"; otherwise, it is called a nonproportional damping matrix. The most common proportional damping matrix is obtained by assuming that $[c]$ is proportional to $[m]$ and $[k]$ (Rayleigh Damping):

$$[c] = \alpha [m] + \beta [k], \quad \text{(X4.6)}$$

where $\alpha$ and $\beta$ are constants.

A more general form of Eq. (X4.6) is presented in Caughey (7).

$$[c] = [m] \sum_{j=0} a_j ([m]^{-1}[k])^j \quad \text{(X4.7)}$$

$$= a_0 [m] + a_1 [k] + \ldots$$

For proportional damping, the equations of motion may be uncoupled into a set of independent one-degree-of-freedom systems. Considering the $r$th equation from Eq. (X4.4):

$$\ddot{q}_r + C_r q_r + \Omega^2_r q_r = [\phi]^T_r \{\dot{p}\} = P_r \quad \text{(X4.8)}$$

where

$$C_r = [\phi]^T_r [c][\phi]_r.$$  

By comparing Eq. (X4.8) with a single degree-of-freedom equilibrium equation, it can be rewritten as

$$\ddot{q}_r + (2 \Omega_r \Omega_r) q_r + \Omega^2_r q_r + P_r.$$  

(X4.9)
where $\eta_r$ represents the critical damping coefficient for mode $r$. The
damping coefficients can be determined experimentally for a given
structure or obtained from experiments on similar structures.

The approach of uncoupling the dynamic equilibrium equations using
a coordinate transformation is commonly referred to as the "modal
superposition" technique. As indicated above, this technique generally
involves solving the undamped eigenvalue problem. When damping is
"light," the effect of damping on the natural frequencies and modes
shapes is negligible. From Table X4.1, the maximum damping listed is
10% of critical. For 10% damping in a single degree-of-freedom
problem, the damped natural frequency is only about 1% less than the
undamped value [see, for example, pages 2-4 of Harris and Crede (14)].
Thus, in the context of this appendix, the damping can be considered
light and the effect of damping on the modal superposition approach is
not significant.

The concept of diagonalizing the damping matrix and having a
critical damping ratio associated with each mode is important with
respect to composite modal damping. In general, if the damping is not
uniform within a structure, the composite damping matrix will not be
diagonalized by the normal modes. The resulting damping matrix is
termed nonproportional and requires special treatment as will be
discussed in the next section.

Equation (X4.9) bears further discussion with respect to the
damping coefficients given in Table X4.1. Most of the Table X4.1
states that these values "should be used for viscous modal damping for
all modes." Thus, for uniform damping, the Eq. (X4.9) uncoupled
expressions can be solved using a single damping coefficient, $\zeta =$
constant. For an analysis approach using Rayleigh damping, Eq. (X4.6),
the constants can only be selected for two frequencies, and the
damping varies with frequency. Also, for a composite structure, even
though the damping is constant with respect to frequency for each com-
ponent, the composite damping varies with each mode. This is demon-
strated in the next section.

X4.5 COMPOSITE MODAL DAMPING METHODS

As discussed in the introductory section, there is some inconsis-
tency in the literature in the terminology used in the subject of
composite modal damping. Different names are often used for the same
method. In the evaluation report Julyk and Winkel (3) the methods were
categorized into three groups: (1) weighted energy ratio methods; (2)
component proportional damping methods; (3) component-modal synthesis
methods. A summary of each method follows below.

The composite modal damping problem divides into two major tasks:
(1) the development of the system composite damping properties; (2) the
solution of the resulting dynamic response equations, which, in
general, are not uncoupled. This section deals with the first task. The dynamic equation solution techniques are covered in Sec. X4.6.

X4.5.1 Weighted Energy Ratio Methods. If an analyst is faced with a structural model composed of components having different damping properties, a reasonable first approach is to bound the problem by assuming uniform damping at the two extremes of the damping range. If this approach is not adequate, the next logical step is to use a weighted average of the component damping values. The weighted energy approach assumes that the weighting should correspond to the relative amount of strain energy in a component for a particular mode:

\[ \zeta_r = \frac{\sum_{i=1}^{nc} \zeta_i E_{ir}}{\sum_{i=1}^{nc} E_{ir}} \]  

(X4.10)

where

- \( \zeta_r \) = composite damping coefficient for the rth system mode
- \( \zeta_i \) = damping coefficient for the ith component
- \( E_{ir} \) = elastic strain energy of the ith component vibrating in the rth mode
- \( [k]_i \) = ith component stiffness matrix expanded to system degrees-of-freedom
- \( n_c \) = number of components.

A less common alternate expression involving the kinetic energy of the mass can also be used:

\[ \zeta_r = \frac{\sum_{i=1}^{nc} \zeta_i T_{ir}}{\sum_{i=1}^{nc} T_{ir}} \]  

(X4.11)

where

- \( T_{ir} \) = kinetic energy of the ith component vibrating in the rth system mode.

\[ = \frac{1}{2} \Omega_r^2 [\phi]^T_r [m]_i [\phi]_r . \]
Equations (X4.10) and (X4.11) are referred to as the "stiffness weighted" and "mass weighted" energy ratio methods. They correspond to Eqs. (X4.3) and (X4.4) in (1). Once the damping coefficients are obtained for each mode, either a time history or response spectra analysis may be performed.

Implicit to Eqs. X4.10 and X4.11 is the assumption that the damping ratio for a given mode is dependent on the energy in that mode only. A more general approach is provided in the guideline report Julyk and Winkel (3), which includes cross coupling of modes. This results in a damping ratio matrix with off diagonal terms. The significance of these off diagonal terms is discussed further in Sec. X4.6.

Some variations to Eqs. (X4.10) and (X4.11) exist in the literature. In (5) an effort is made to develop Eq. (X4.10) as a combination of viscous and hysteretic damping. The resulting equation is:

\[
\tau_r = \frac{n_c}{\sum_{i=1}^{nc}} \left( \tau_i^v \frac{\Omega_r}{\omega_i} + \tau_i^H \right) E_{ir}
\]

(X4.12)

where

- \( \tau_i^v, \tau_i^H \) = viscous damping coefficient and hysteretic damping coefficient, respectively, for the ith component
- \( \omega_i \) = "reference frequency" for the ith component
- \( \Omega_i \) = rth system frequency.

Some difficulties arise in applying this equation. The first is that available damping coefficients are generally not broken down into viscous or hysteretic categories. The second problem is that it is not clear how to obtain "reference frequencies" for each component of the system, e.g., should the component interfaces be fixed or free? Also, for a given \( \omega_i \), the viscous damping increases with \( \Omega_r \) and the higher modes will be damped out.

X4.5.2 Component Proportional Damping Methods. This method was suggested by Idriss et al. (9). It is based upon the assumption that a proportional damping matrix can be constructed for each component of the system model. For example, if Rayleigh damping, Eq. (X4.6), is assumed,

\[
[c]_i = \alpha_i [m]_i + \beta_i [k]_i
\]

(X4.13)
where the subscript of \( i \) represents the \( i \)th component within a system. Using standard procedures [Appendix N of (2)], the component proportional damping constants can be found as a function of the component damping coefficient \( \zeta_i \), and frequencies:

\[
\alpha_i = \frac{2\omega_i \omega_i}{\omega_i + \omega_i} \zeta_i \]

\[
\beta_i = 2\zeta_i \frac{1}{\omega_i} \]

where \( \omega_i \) and \( \omega_i \) are "reference frequencies" of the \( i \)th component.

The system damping matrix is obtained from a global assemblage of the individual component damping matrices. Since the system damping matrix is not related to the system mass and stiffness matrices by a single \( \alpha \) and \( \beta \), it is not, in general, a diagonal matrix. Various options for handling the off-diagonal terms are discussed in Sec. X4.6.

The composite damping ratios for the system are derived in Par. N-1233.1 of (2). A basic assumption of the derivations is that the off-diagonal terms of the composite damping matrix can be neglected. From Eq. (X4.15) of the referenced paragraph, the effective damping of the \( r \)th mode is given as:

\[
\eta_r = \frac{\sum_{i=1}^{nc} (\Omega_r / \omega_i) \zeta_i [\phi]_r^T [k]_r [\phi]_r}{\sum_{i=1}^{nc} \beta_i [k]_r [\phi]_r} \]

It is interesting to consider the "stiffness proportional" case \( \alpha_i = 0 \). For this case, only a single component "reference frequency," \( \omega_i \), can be utilized and \( \beta_i = \frac{2\zeta_i}{\omega_i} \). Substituting into Eq. (X4.15),

\[
\eta_r = \frac{\sum_{i=1}^{nc} (\Omega_r / \omega_i) \zeta_i [\phi]_r^T [k]_r [\phi]_r}{[\phi]_r^T [k]_r [\phi]_r} \]
Note that Eq. (X4.17) corresponds to Eq. (X4.12) if \( \zeta_i = 0 \). Also note that Eq. (X4.17) is identical to Eq. (X4.10) if the "reference frequency" of the component is set equal to the rth system frequency, \( \Omega_r \). Thus, for this special case \( (\alpha_i = 0, \omega_i = \Omega_r) \), the stiffness weighted energy and component proportional methods are equivalent.

One difficulty with the component proportional method is in the selection of the reference frequencies for each component. In Idriss et al. (9), a single fundamental system frequency was used for all components. In Julyk and Winkel (3), "free interface" component frequencies were used. Based upon the experience described in Sec. X4.7, the system frequencies appear to give better results.

The above development of the component proportional method is based upon Rayleigh damping. For a more general approach using Eq. (X4.7), the reader is referred to page 8 in Julyk and Winkel (3).

X4.5.3 Modal Synthesis Method. Of the methods discussed in this appendix, the modal synthesis method is considered to be the most rigorous. The procedure involves synthesizing the system vibration modes from the unconstrained components (or subsystem) modes. The modal matrices of all the subsystems are assembled into system matrices and displacement compatibility at component interfaces is satisfied.

Two basic approaches appear in the literature. The first, the "free interface method," examines each subsystem independent of the remaining structure. In the second approach, the stiffness and inertia of the connecting structure is taken into account in an approximate manner. For purposes of identification, this approach is referred to as the "K, M loaded interface" method. The development of these methods is relatively complex and lengthy. For development details and background references, see the methods evaluation report Julyk and Winkel (3).

X4.6. DYNAMIC EQUATION SOLUTION PROCEDURES

In the general case, all three composite damping methods discussed in Sec. X4.5 will produce dynamic equations which will not be uncoupled by the undamped modes. For such a case, the damping matrix is termed as nonproportional. The solution of the coupled system equations can be obtained by integrating these equations simultaneously. Various step-by-step numerical integration schemes can be used.
An alternative procedure is to solve the complex eigenvalue problem. The original system of N equations is transformed into an equivalent 2N-space system which is uncoupled. Once the complex eigenvalues are obtained, the solution can proceed in a parallel fashion to the case of proportional damping by modal superposition.

A third approach is to neglect the off-diagonal terms in the composite damping matrix. On page 613 of Nelson and Grief (4), it states that this approach should be "reasonably accurate if the damping is light." Based upon limited experience described in Julyk and Winkel (3) which considered damping up to 14% of critical, the effect of neglecting the off-diagonal damping terms produced a maximum error of only 4%. In (15), Warburton and Soni proposed a criterion for quantifying the error associated with neglecting the off-diagonal damping terms. This criterion was found to be conservative in (3).

One advantage of obtaining a single composite damping ratio for each mode, i.e., no off-diagonal damping terms, is that it permits a response spectrum analysis of the system model. Once the modal composite damping magnitudes are determined, e.g., Eqs. (X4.10) or (X4.15), then the corresponding spectral values can be obtained for each mode.

X4.7 COMPARISON OF METHODS

In order to gain experience with the various composite modal damping methods and provide some basis for comparing the methods, a composite model was developed and analyzed. The details of the analysis and evaluation are contained in the evaluation report (3). A brief summary follows below.

The structural model selected consisted of three components; an in-line large and smaller diameter pipe simply supported at an intermediate location by a support beam. A sketch is shown in Fig. 4.1. The seismic ground motion excitation time-history and response spectra are shown in Fig. X4.2. Analyses of this problem were carried out using each of the methods discussed in Sec. X4.5. Time-history analyses were performed using a modified version of CAL78, a computer analysis language for the static and dynamic analysis of structural systems (10). Comparison runs using ANSYS (11) were also made. The purpose of ANSYS runs was to demonstrate the use of an existing capability which is readily available. Both response spectra and time-history runs were made be with ANSYS (Sec. X4.8).

Many parameter variations of this problem are reported in (3). A portion of these results is reported here to illustrate trends.

A comparison of the composite damping predictions is shown in Figs. X4.3 and X4.4. Figure X4.3 illustrates what can happen if the composite modal damping methods are used for a case where all components are assigned the same damping. The weighted energy methods
predicted the 2% system damping for all modes. Since the component proportional methods assumed a two-constant (Rayleigh) damping, higher damping for the higher modes was anticipated. However, on the component level, the modal synthesis methods assumed a 2% damping for all modes. Therefore, the significant deviation from 2% was surprising, especially for the K, M loaded interface approach. Note that the free interface modal synthesis underpredicted the damping while the K, M loaded interface method significantly overpredicted the 2% uniform damping.

Figure X4.4 illustrates the composite damping predictions for the case of 3 and 2% pipe damping and 14% support beam damping. All methods predicted the same trend of damping vs frequency. Analogous to the uniform damping case, the free interface modal synthesis method tended to predict low damping and the K, M loaded interface method produced high damping values. For the fifth frequency, the component proportional, system frequency approach predicted high damping which is due to the two-constant Rayleigh damping.

Acceleration predictions for the uniform 2% damping case are shown in Fig. X4.5. Peak accelerations are provided for each method for both the pipe and support beam. Since no "exact" solution is available, only deviations about the average of the predictions are shown. For the pipe accelerations, the predictions ranged from 8% below to 5%
Fig. X4.2. Seismic ground motion description.
Fig. X4.3. Predicted damping ratios, uniform system damping.
Fig. X4.4. Predicted damping ratios, 3, 2, 14% component damping.
above the average. For the beam, the range was 21% below to 7% above. The high damping produced by the K, M loaded interface modal analysis method is evidenced by the low acceleration predictions.

The acceleration predictions for the 3, 2, 14% nonuniform damping case are summarized in Fig. X4.6. As indicated, the pipe accelerations ranged from 8% below to 5% above the average. For the support beam, the values varied from 4% below to 9% above the average. It is interesting to note that the ANSYS predictions appear to be adequate even with the added uncertainty of the response spectra method.

X4.8. TWO-DEGREE-OF-FREEDOM PARAMETRIC STUDY

A parametric study was performed on the two-degree-of-freedom model shown in Fig. X4.7. Using the ground motion described in Fig. D-2, various combinations of damping mass ratios and stiffness ratios were considered. The primary objective was to evaluate the degree of conservatism in a minimum uniform damping approach vs variable damping. For all cases, the weighted energy response spectra approach available in ANSYS (11) was used.

A summary of the results is shown in Figs. X4.8 and X4.9. The contours shown are based upon limited data and are therefore shown only to illustrate trends.

It is also emphasized that the results are based upon a specific response spectra (Fig. X4.2). However, it does illustrate the sensitivity of results to mass and stiffness ratios. By accounting for a 2%, 7% variable damping, reductions in maximum stress of up to 38% were found. This illustrates the potential payback in considering variable damping vs a minimum uniform damping approach.

X4.9. COMPUTER PROGRAMS

On page 619 of (4), a survey of damping capabilities in the more common dynamic analysis programs is summarized. Of the twelve programs listed, only STARDYNE (12) is mentioned as having incorporated one of the Composite Modal Damping methods discussed in this appendix. STARDYNE utilizes the strain energy weighting approach to composite damping.

A number of programs are reported to have proportional damping capability. It is not specified if the proportionality constants can be input as material or element dependent. Based upon a survey of a limited number of User's Manuals at the time of the writing of this appendix, only ANSYS (11), was found to have this type of capability. ANSYS permits \( \beta \), Eq. (X4.6), to be material dependent. This ANSYS capability was successfully utilized as discussed in Sec. X4.7.
Fig. X4.5. Comparison of acceleration predictions for 2% uniform damping.
Fig. X4.6. Comparison of acceleration predictions for a 3, 2, 14% nonuniform damping.
Fig. X4.7. Two-degree-of-freedom variable damping model.
Fig. X4.8. Two-degree-of-freedom parametric study results for variable damping case of $\zeta_1 = .07$, $\zeta_2 = .02$. 
The recently released Revision 4 version of ANSYS permits strain energy weighted damping to be used in conjunction with a response spectrum analysis. It is incorporated into the new "Complete Quadratic Combination" method of combining modal responses [see page 2.12.6 of (11)]. The use of this new capability was included in the Sec. X4.7 comparison.

A number of computer programs permit the use of discrete damper elements. This permits composite damping within a structure, provided the appropriate discrete damper coefficients can be obtained.

For soil/structure interaction problems, the LUSH series of computer programs are available. The latest of the LUSH series is PLUSH (13). Damping ratios on PLUSH can vary from element to element. The analysis approach used in the LUSH series is not considered to be within the class of composite modal damping and is therefore not discussed in earlier sections of this report.

X4.10. CONCLUSIONS

The complex nature of damping in real structures does not lend itself to precise analytical treatment. Even when the damping is uniform over the entire structure, the effects of damping are generally accounted for in an approximate manner.

All of the methods available for treating nonuniform damping in structural dynamics involve some approximations. Therefore, no exact solution was available for evaluating the approximations. However, based upon the experience described in Sec. X4.7, some conclusions were made.

The most surprising finding in the methods comparison of Sec. X4.7, was the poor showing of the modal synthesis methods in handling the case of uniform damping. The modal synthesis, free interface approach underpredicted the damping for each mode, while the K, M loaded interface method highly overpredicted the modal damping. Since the modal synthesis methods appeared to be mathematically most rigorous, a better performance was expected.

The overprediction of higher mode damping for the component proportional methods was due to the limitation of the two-constant (Rayleigh) proportional damping assumption. Since the lower modes dominate the response, the acceleration predictions for the pipe/beam problem were reasonable. Better damping predictions would be expected if the more general proportional damping, Eq. (X4.7), was used. The biggest difficulty with the component proportional approach is in selecting the reference frequencies to use in calculating the proportionality constants. Based upon the experience described in Sec. X4.7, it is recommended that the reference frequencies be based upon the system frequencies. This recommendation is also influenced by the fact...
that, as demonstrated in Sec. X4.5.2, the component proportional method is mathematically equivalent to the weighted energy approach when the reference frequency is set equal to the system frequency.

Based primarily upon the above mentioned problems with the modal synthesis and component proportional methods, it was concluded that the best available method for accounting for nonuniform damping is the weighted energy approach. The weighted energy method is straightforward in application and is easily adapted to existing structural dynamics computer codes. It is currently available in at least two general purpose finite element codes using both time-history and response spectra methods.

A common alternative to a variable damping approach is to conservatively assume a uniform minimum damping. The degree of conservatism of this approach is a function of the stiffness, mass and damping distributions. Based upon a two-degree-of-freedom study, significant stress reductions can be realized by accounting for the variable damping within a structural system.

REFERENCES


APPENDIX - NONMANDATORY

X5. GUIDELINES FOR DYNAMIC DECOUPLING IN SEISMIC ANALYSIS

X5.1 SCOPE

Dynamic decoupling is an important feature of seismic analysis of nuclear power plant components. Its use can significantly affect completing seismic qualifications in an economic and timely manner.

This standard, as well as the NRC Standard Review Plan (1)* present criteria for dynamic decoupling which may be overly restrictive and general in nature. It is the purpose of this appendix to evaluate the present criteria, compare it with error bounds, and to provide guidelines for their application to system configurations found in nuclear plants.

Material presented in this appendix has been directly obtained from (2). This reference presents the results of work performed by Hanford Engineering Development Laboratory (HEDL) for the Materials and Structures Program sponsored by the DOE Office of Breeder Technology Projects.

X5.2 SUMMARY AND RECOMMENDATIONS

There are a number of papers in the literature on the subject of dynamic decoupling. All of the proposed decoupling criteria for the seismic analysis of nuclear power plants appear to be primarily based upon natural frequency errors in a two-degree-of-freedom system. Based upon the current state of the art, there is insufficient evidence to support a change in the current F 9-2T decoupling criteria. However, information is available which can provide assistance in applying the criteria. This information is summarized below in the form of guidelines and conclusions. Details are provided in the body of the report.

1. Decoupling procedural details should be system dependent. As indicated in Sec. X5.4.1 for secondary/primary frequency ratios greater than one, it is recommended that the secondary mass be lumped with the primary mass for the decoupled primary model. For frequency ratios less than one, the secondary mass should not be included in the decoupled primary model.

2. Decoupling complex multi-degree-of-freedom systems require judgment and caution. As discussed in Sec. X5.3.3, the concept of "modal mass" can be used to estimate the secondary/primary mass ratio for a particular pair of primary and secondary frequencies. The frequencies are usually fundamental, but higher modes should be considered if participation is significant.

*The numbers in parentheses refer to the list of references at the end of this appendix.
3. A decoupling interface is often conveniently based upon jurisdictional boundaries, e.g., an equipment/building interface. Care should be taken to assure that such interfaces are selected and treated properly. For example, a building analyst needs some understanding of floor and equipment stiffness in order to properly set up a decoupled primary model. The equipment analyst also needs some understanding of the building model to assure proper application of the generated equipment spectra.

4. Based upon experience with both multi- and two-degree-of-freedom systems, a decoupled analysis generally produces conservative responses in both the primary and secondary systems. An analyst should be aware that such conservatisms can be excessive, particularly for a supported system with secondary/primary frequency ratios close to 1.0. Compounding of this conservatism can occur if a multiple sequence of decoupling occurs.

5. If there is a need for coupled analysis, the analyst should be aware of two points. First, detail modeling of the secondary system may not be necessary. Techniques such as modal synthesis, dynamic substructuring or coarse modeling may simplify the modeling task. Second, primary and secondary systems with nearly equal natural frequencies can be quite sensitive to the masses and stiffnesses used in the coupled analysis, and the analyst should exercise special care in such cases.

A decoupling criteria based primarily upon response rather than frequency error is more reasonable. The existing criteria could probably be improved if a response based criteria was pursued. Some attention has been given to the coupled/decoupled response to frequency independent white noise. More insight can be derived by considering ground motion more representative of seismic spectra.

X5.3 BACKGROUND

X5.3.1 Decoupling Criteria. Seismic analysis of a nuclear power plant typically begin with a transient analysis of a building model with soil-structure interaction included. The building model is not fully detailed because of cost considerations and well known numerical problems that can arise from extreme differences in mass or stiffness. Furthermore, details may not be known at this step. Decoupling criteria should influence the extent of detail modeled. After a primary model of the building and soil is analyzed, the resulting motions at various points in the building can be used to analyze attached equipment later. This is called decoupled or cascaded analysis, because the equipment base motions are prescribed. Intuitively, there should be a level of equipment mass that will not influence the building motion when omitted. However, the concept of a vibration absorber, in which a small spring-mass system will stabilize a much larger one, should lead one to believe that such a mass level might be rather low in a resonance situation.
Several proposed decoupling criteria have been found in the literature. They are mostly based on the two-degree-of-freedom model shown in Fig. X5.1, and the criteria are expressed in terms of the mass ratio \( \mu = \frac{M_2}{M_1} \) and frequency ratio \( \rho = \frac{\omega_2}{\omega_1} \).

Pickel (3) provided background work for the 1974 version of F 9-2T criteria as shown in Fig. X5.2. Decoupling is permitted to the left of the criterion curve. When done at a high frequency ratio, \( \rho \geq 2 \), the secondary mass (M_2) should be lumped with the primary mass in the analysis of the decoupled primary system. Conversely, the secondary mass should be omitted at a low frequency ratio \( \rho \leq .5 \). This criterion was based on a 2 to 3% permissible natural frequency error in the primary system. In view of subsequent work reported below, this seems to be a rather stringent requirement. Figure X5.2 also shows the NRC criterion (1) which was adopted in the current F 9-2T standard. Here, no distinction is made as to how the secondary mass is to be lumped into the primary system. Pal, et al., (4) discusses the large differences between the two criteria. There appears to be no background material to the NRC or later F 9-2T criterion.

Fig. X5.1. Two-degree-of-freedom model.
Hadjian (5) considers the frequency errors induced by analyzing the primary and secondary systems separately with fixed bases. The frequency errors of both systems control the coupling criterion. If a 5% error governs, the criterion is shown in Fig. X5.3. More importantly, Hadjian considered Crandall and Mark's (6) random vibration results. Now, the model of Fig. X5.1 is subjected to white noise accelerations, and the expected RMS values of spring stretch and mass accelerations are given in terms of frequency, mass and fractional damping ratios. Thus, the problem is carried through to stretch and mass acceleration instead of just frequency error. Hadjian considers 15% unconservative and 25% conservative errors in the random vibration results as significant. Corresponding boundaries are also shown in Fig. X5.3 for equal damping fractions of 5% in the primary and secondary systems. It is of interest to note that only the conservative boundaries penetrate into the low mass ratios. The only unconservative error is the 15% primary spring stretch. This is believed to be the price of excluding the secondary mass from the decoupled primary model, and its effect would be much less severe if the secondary mass were lumped with the primary mass at the higher
Fig. X5.3. Random vibration results for $\zeta_1 = \zeta_2 = 0.05$.

frequency ratios. The 25% conservative error in the primary mass acceleration reflects the role of the secondary system acting as a vibration absorber in the coupled system. Finally, the 25% conservative error in the secondary spring stretch, $y_2 = x_2 - x_1$, reflects the price of cascading. It is assumed here that the decoupled secondary system is subjected in a prescribed way to the motion of the primary mass which, in turn, is the result of the primary system being subjected to the ground motion.

The random vibration results described above introduce damping into the problem. The equal 5% fractional values of critical damping used in Fig. X5.3 coincide with that used in the main body of the Crandall and Mark text, and they represent some of the higher values in Reg. Guide 1.61 (7). The problem was reworked with $\zeta_1 = .04$ and $\zeta_2 = .02$ which is motivated by small piping in the secondary role and a welded steel structure as the primary system in a Safe Shutdown.
Earthquake. The results are shown in Fig. X5.4. The change is not significant. The 25% conservative error in primary acceleration no longer shows on the figure.

Aziz and Duff (8) give detailed results for the two-degree-of-freedom oscillator. Their proposed criterion for decoupling is the 5% frequency boundary on Fig. X5.3.

Finally, there is an ASCE effort, (9) on decoupling that is summarized in Fig. X5.5. These curves appear to reflect about a 10% frequency error. Similar curves based on a 5% error are shown in Fig. X5.6. The solid curve is that of Hadjian (5) and Aziz and Duff (8). The dashed curve represents the effect of lumping the secondary mass with the primary system when analyzing the primary system alone. At the higher frequency ratios, it is obtained by comparing the smaller frequency root of the coupled system with that of the primary system including the secondary mass. At the lower frequencies, the comparison is made with the larger frequency root.

![Diagram](image.png)

**Fig. E-4.** Random vibration results for $\zeta_1 = .04$, $\zeta_2 = .02$. 
Fig. X5.5. ASCE criteria under development.

Fig. X5.6. 5% frequency error bounds.
X5.3.2 Floor Amplification. Floor amplification concepts differ from decoupling in that an equipment designer is faced with analyzing an equipment model without adequate knowledge of the supporting structure as a floor. Typically, he has knowledge of the excitation beyond the floor in the form of a response spectra. He also may have an estimate of floor and equipment masses and perhaps natural frequencies. The procedure is to use the response spectrum as if it applied to the equipment base, but modal loads are increased by floor amplification factors to account for support flexibility. In a resonance situation, which may have to be assumed, the factors can become quite large.

Gelman (10) provides floor amplification factors which are reproduced in Fig. X5.7. The model is the same as in Fig. X5.1. These factors were obtained by RMS summation of modal response and assumptions tantamount to a flat spectrum. No provision was made for closely spaced modes (11) nor is account made for the reductions in near-resonant response reported by Aziz (12) and Lin and Esselman (13). In spite of this, the factors around \( \omega_2/\omega_1 = 1.0 \) are not drastically different than those obtained by Suzuki (14). The latter factors were obtained by transient analysis with various seismic histories, and the effects of damping are included.

![Fig. X5.7. Gelman's floor amplification factors.](image-url)
The random vibration results of Crandall and Mark (6) are easily interpreted as floor amplification factors by normalizing the primary spring response in the coupled problem by the primary single-degree-of-freedom response; the result is shown in Fig. X5.8. The main difference between this and Gelman's result is that the random vibration results show the expected vibration isolator character at the higher frequency ratios.

Maximum floor amplification factors from various sources are plotted in Fig. X5.9 against mass ratio. Included is the simple formula \( (\frac{1}{\omega_1^2} + \frac{1}{\omega_2^2} + \sqrt{\frac{M_2}{M_1}})^{-1} \) given by Newmark (16). The Gelman factors continue rising indefinitely as the mass ratio tends to zero, probably because no damping enters into the problem. Overall, the three results are remarkably close.

X5.3.3 More Complex Models. A few models appear in the literature that go beyond the two-degree-of-freedom model of Fig. X5.1.

Scavuzzo and Lam (15) consider the case in which the secondary mass consists of several spring-mass systems in series. Time history analysis without damping provides responses of the various models. The results show that decoupling generally leads to conservatism, and that frequency errors can be related to modal mass defined by

\[ M = \frac{(\sum \omega_i^2 M_i)^2}{(\sum \omega_i x_i)^2} \]  

(X5.1)

Here, \( x_i \) is the modal deflection of the \( M_i \) mass in the isolated secondary system. Knowing the frequency errors associated with the decoupling of the two-degree-of-freedom system, one can estimate the frequency error caused by decoupling the more complex model by summing the percentage errors (with due regard to sign) based on the secondary system modal masses and frequencies. The concept of modal mass was used by Newmark (16) in a multi-degree-of-freedom primary system to bound the secondary response with the square root of the mass ratio. The attachment point of the secondary mass into the primary system does not enter into the problem.

Aziz (17) discusses coupling effects with a model having a secondary system with two spring-mass systems in series. The conservatism of decoupled analysis is noted, and bounds to the response of each secondary mass are given. It is interesting to note here that mention is made of possible nonlinear effects on the extremely large amplifications of the outermost secondary mass system.
Fig. X5.8. Floor amplification based on white noise excitation.

Fig. X5.9. Maximum floor amplification factors.
Ibrahim and Callahan (18) consider a simple header branch piping configuration with parameter techniques. It is noted here that the point of attachment of the branch to the header is important. As this point is moved closer to a header boundary constraint, the header response change due to decoupling would become negligible.

X5.4. ANALYSIS PROCESS

X5.4.1 Cascading. Decoupled analysis by cascading starts with analyzing some primary system and adding secondary masses to the primary system or ignoring them. Adding the secondary masses is probably the first inclination, but the practice should be preferred for relatively stiff secondary systems. Flexible secondary systems are better ignored in the primary system analysis. Once the primary system is analyzed, the secondary system can be treated with the resulting motion or response spectrum at the attachment location.

The stage of the design process has a strong influence on the decoupling process. Initially, the analyst may only have mass information on equipment and structure. Furthermore, it appears that analysts expect fairly rigid equipment, and this is probably the reason secondary mass inclusion is the first tendency in the decoupling process. If the frequencies of the primary and secondary systems are ignored, Lin and Liu (19) indicate that omitting the secondary mass provides better results overall in view of the results of Pickel (3). However, much is to be gained from the frequency ratio. If the secondary system is relatively stiff, its mass will follow the primary mass, and secondary mass inclusion in the primary model is desirable. If the secondary system is relatively flexible, however, its mass would remain still as if it were protected with a vibration isolator. Exclusion of the secondary mass from the primary model would be preferred. Secondary stiffness, including intermediate supports, plays a role in the primary modeling decision process.

Another reason for lumping secondary masses into the primary model may be to mentally avoid an accumulation of mass errors by ignoring many secondary systems that are attached independently to the primary system. If all these secondary systems are relatively flexible, it seems that at worst they would act together as a single flexible system.

Consideration should be given to the higher ratio of total secondary mass to primary mass if the natural frequencies of the secondary systems are nearly the same. Then the question would be the necessity of coupled analysis and not the need to lump the secondary masses into the primary mass.

Analysts should be aware of the strong conservatism that can result from cascaded analysis of systems with resonances. This is especially true for multiple cascades. Consider a containment
structure with an appurtenance such as an airlock. Usually the containment structure is analyzed as a stick model or axisymmetric structure with soil-structure interaction included. The motion at the airlock elevation may be used as input to a model of the airlock and containment in which non-axisymmetric deformation of the containment shell is included; this is one cascade. Then an operator on the airlock may be tested to a response spectrum derived from the ensuing airlock motion. This amounts to a double cascade which could be very conservative if natural frequencies line up.

Figure X5.10 shows this conservatism with the secondary spring stretch ratio of expected cascaded response to coupled response with white noise excitation. Some of this conservatism may be masked out if the coupled analysis is carried out with a response spectrum method with SRSS or effectively absolute summation of modal responses. The effect (13 or 17) seems of importance with very low mass ratios and high damping values so that the masking does not appear to be significant.

![Fig. X5.10. Conservatism of cascaded response over coupled response.](image-url)
X5.4.2 Coupled Analysis. Conceptually, coupled analysis should cover coupling concerns, but one may consider degrees of decoupling with coarsened primary or secondary models. Depending on the costs and other circumstances, such models may be used to justify decoupling decisions or to demonstrate the need to carry out final coupled analysis. Even with detailed coupled analysis there is concern for the sensitivity of near resonant situations.

Just as peak broadening of a response spectra (generated by time history) is necessary to cover uncertainties in the model, there is a need to look for near resonant situations in a coupled model. An indication of near resonant situations would be closely spaced modes with large participation factors. The resonant two-degree-of-freedom model has nearly equal but opposite secondary mass modal displacements at small mass ratios. Thus, in the complex model, one should look for a secondary system with equal and opposite modal displacements once the primary system displacements are brought together. Then consideration should be given to varying the masses of the primary or secondary system toward resonance for conservatism in the secondary system and away from resonance for conservatism in the primary system. The data given by Tsai (20) suggest that such variations may be overcome by conservatism in the response spectrum method over time history analysis.

Based on conservatisms found in coupled systems, one approach to analyzing a system such as large piping with small branches involves decoupled analysis for the large pipe and coupled analysis for the small pipe system. The decoupled primary system is usually conservative in the sense that a potential vibration absorber is missing, and the coupled model for the small piping avoids possibly large conservatisms associated with cascaded analysis. Such conservatisms are compounded by small branch piping supported to ground as well as the attachment to the large pipe. In a cascaded analysis with the response spectrum method, the analyst would have to choose the larger attachment point spectrum or employ more recent methods of employing different response spectra at different supports. When a coupled model is used for small piping analysis, undue cost may be avoided by coarsening the large pipe portion of the model.

The opposite approach was taken by Pal et al. (4). Here concern was for accuracy in the analysis of the primary system, and single-degree-of-freedom oscillators were placed at attachment points to secondary piping. Model coarsening approaches, of course, can be generalized to modal synthesis. See Hurty et al. (21) for a review of the subject. Modal truncation becomes the criterion for coarsening; however, the method lacks intuitive appeal. Noting the scheme given by the NASTRAN Theoretical Manual (22), Sec. 14.1, the oscillators attached to the primary model might have linear constraints among themselves.
X5.4.3 Support Stiffness. Decoupled analysis is frequently carried out with incomplete or inaccessible specifications. The primary model analysis may be done by one organization and the secondary by another. Engineering judgement is necessary to permit a timely flow of analyses. It seems that an area causing frequent problems after secondary analysis is completed is that of support stiffness. Plant analysts generally expect equipment to be relatively stiff, and it is usually the intent to have the equipment natural frequency residing to the right of the high amplification frequency range of the response spectrum. However, support structure detail is sometimes one of the last things to be completed in the design process. The equipment analyst obviously starts modeling with the basic equipment entity and then extends it into the supporting structure. By that time, the complexity of the model and the lack of support detail may make rigid support assumptions tempting. Such assumptions can be unconservative, because they lead to lower frequencies and possibly significantly low response.

Clearly the problem of overestimating support structure stiffness is not strictly a decoupling problem. It is a question of proper quantification. Nevertheless, decisions regarding support stiffness are made at the same time as decoupling decisions. Support stiffness may be related to the problem of locating the boundary between primary and secondary systems.

Since support stiffness problems can have a significant impact when found in the design verification process they are worth discussing by example.

The first example is a support ledge for a reactor vessel as shown in Fig. X5.11. The initial analysis used the floor motion as input to the reactor vessel, i.e., the ledge was assumed rigid. The design review process brought up the question of ledge flexibility and reanalysis was required to settle the question. The frequency of the vessel with a rigid ledge was 7.2 Hz, while ledge flexibility resulted in 6.0 Hz. A broad peak occurs in the response spectrum around 2-3 Hz. As a result of the decreased natural frequency, the vessel response increased. The reanalysis showed a significantly smaller margin. It would be interesting to compare the ledge flexibility unconservatism with the possible conservatism given by the decoupled analysis of containment.

The second example involves electrical cabinets as shown in Fig. X5.11. Electrical cabinets are usually installed by bolting to channels which are in turn bolted to the floor. Deformation of the channel introduces flexibility into the system since the floor bolts and cabinet bolts do not line up. Cabinets are typically qualified by testing. The mounting to the test fixture should account for the channel flexibility in the actual installation.
Fig. X5.11. Support stiffness examples.
The third example is a pipeline shown in Fig. X5.12. Flexibility of a nozzle should be accounted for. Instead of anchoring the ends of the line, torsional and linear springs may be needed to represent the nozzle flexibility. Hanger and snubber flexibilities are also known to be necessary parts of piping models.

Fig. X5.12. Nozzle and hanger flexibilities.

X5.5. CRITERIA RECOMMENDATION

At this time, it is recommended that the current F 9-2T criteria for decoupling be retained as they are.

First, the 5% frequency error bounds shown in Fig. X5.6 are not far out of line from the current criteria if one chooses the lower mass ratio branch on each side of $\omega_2/\omega_1$. The intent of the ASCE criteria under development to distinguish between stiff and flexible secondary systems is understandable, but little is found in the literature about what one does with complex models. The concept of modal mass may be used to obtain a mass ratio, and natural frequencies may be used for a frequency ratio. In general, each of the primary and secondary structures have a large number of modes, and every pair in the cartesian product could be tested with the decoupling criteria. Virtually any frequency ratio is possible. It is difficult to imagine
anything like this being done on a practical basis, especially since
decoupling decisions are needed before detailed modeling takes place.
On a practical basis, one frequency might be obtained for the second-
ary system with a Rayleigh quotient. The ASCE criteria under devel-
opment are more lenient than the current F 9-2T criteria by a factor of
two at frequency ratios of 0.3 and 3.0 -- not a drastic change
compared to the older and current F 9-2T criteria shown in Fig. X5.2

The flat peak at \( m_2/m_1 = 0.01 \) of the current criteria does
not seem to conform with the frequency and random response error
bounds shown in Figs. X5.3, X5.4, and X5.6. Considering, however, the
approximate nature of frequency estimates at the preliminary analysis
stage, this flatness may be retained as peak broadening.

Based on the random responses observed at near resonance condi-
tions, one should expect the requiring coupled analysis between mass
ratios of 0.01 and 0.1 has the effect of avoiding large over-conserva-
tism in the secondary system of the model. This is the case in the
simple two-degree-of-freedom model. In a complex model, it is not
clear that the error will remain conservative in all parts of
structure. The use of a 25% conservative boundary by Hadjian (5) is
considered prudent.

The present F 9-2T criteria may be overly restrictive around mass
ratios of 0.1 with higher low frequency ratios well removed from 1.0
and in the area of mass ratios of 0.01 to 0.05 with frequency ratios
from 0.8 to 1.25. The restriction of requiring coupled analysis may
lead to significant cost penalties. For example, decoupled analysis
with proper cascaded procedures would normally be expected to give
conservative response values and would usually cost much less to do
the analysis. Accordingly, with some further work in this dynamic
decoupling technology, it appears likely that adequate justification
for a less restrictive criteria, such as being considered by ASCE, is
possible.

X5.6 RECOMMENDATIONS FOR FURTHER WORK

X5.6.1 Random Vibration. An obvious area for further work is in
exploiting random vibration concepts. These concepts seem to be a
logical choice for supporting decoupling criteria, because they work
the problem from dynamic loading to stress and acceleration without a
specific time history.

Another option in the white noise excitation of a two-degree-of-
freedom model would be to lump the secondary mass with the primary
mass in the decoupled (cascaded) response calculation. The intent is
to relieve the \( y_1 \) unconservative boundary at the higher frequency
ratio of Fig. X5.3. This would support the ASCE two-model criteria.
Preliminary results obtained at the time of this writing did not indicate much relief. White noise has a higher power content at low frequencies, and it is believed that the mass lumping has the effect of introducing unconservatism in the primary system.

An alternative to white noise excitation is desirable. A seismic event does not have the low frequency power of white noise, and one expects a zero power spectral density above a certain frequency. Use of a banded or "typical" spectral density might relieve indications of primary system unconservatism discussed in the above paragraph. Moreover, work here is expected to show less amplification when both the primary and secondary systems have high frequencies. Some of the work of Suzuki (14) indicates that decreasing floor amplification factors occur as the natural period goes to zero. Results here might be used to modify decoupling criteria for high frequency systems. The simplicity of white noise excitation no longer is available, however. Another parameter, such as the natural frequency of the primary system is required. Maps such as Fig. X5.3 could be generated, but now each error boundary would represent an envelope of boundaries for several primary frequencies. The use of a typical spectral density also accommodates the notion of a stiff primary or secondary design residing to the right of a peak on a steep portion of a response spectrum. Here, frequency raising errors are expected to produce unconservative results, and the method should bring this out.

It should be mentioned that the transient results of Tsai (20) show much greater conservatisms for decoupled analysis at near resonance conditions than the random vibration bounds of Fig. X5.3. Random vibration results should be checked with some transient analyses.

X5.6.2 Realistic Models. There is very little work with realistic multi-degree-of-freedom models appearing in the literature. The header-branch treatment by Ibrahim et al. (18), with a parameter embedding technique provides a simplification that might be exploited to show how attachment might influence decoupling criteria. Intuitively, an attachment point nearer the header support would enhance the role of higher primary modes and a lower frequency ratio may be appropriate.

REFERENCES


APPENDIX - NONMANDATORY

X6. RECOMMENDED DAMPING VALUES FOR THE DESIGN OF EIGHT-INCH AND SMALLER INSULATED LMFBR PIPE SYSTEMS

X6.1 SCOPE

The purpose of this appendix is to present the data base that supports the recommendation associated with the increase of the damping values for small diameter LMFBR pipe systems. This data base has been obtained through the experimental research conducted by the Westinghouse Hanford Company (WHC) at the Hanford Engineering Development Laboratory (HEDL) and Westinghouse-Advanced Energy Systems Division (W-AESD) as part of the DOE Breeder Seismic Technology Program.

Material presented in this appendix has been obtained from (1).*

X6.2 SUMMARY

A survey has been completed of available damping test data for insulated Liquid Metal Fast Breeder Reactor (LMFBR) liquid metal pipe systems. The data indicate that damping in these systems is much larger than presently used for seismic design. The use of larger damping values will reduce costs of these pipe systems by reducing the number of seismic supports required. Reliability of the systems will also be improved because of the reduction in component parts and complexity.

Based on this survey, the following damping values are recommended for small diameter insulated LMFBR liquid metal pipe systems for both Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) seismic design:

<table>
<thead>
<tr>
<th>Recommended Value</th>
<th>Pipe Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Critical Damping</td>
<td></td>
</tr>
<tr>
<td>8.0%</td>
<td>1-in. or smaller</td>
</tr>
<tr>
<td>5.0%</td>
<td>8-in.</td>
</tr>
<tr>
<td>Linear Interpolation</td>
<td>&gt;1-in. &lt;8-in.</td>
</tr>
</tbody>
</table>

These values are recommended for insulated pipe systems meeting the insulation weight ratio limits shown in Fig. X6.1. Damping may be assumed independent of pipe response frequency for pipe sizes through 8-in. which comply with these weight ratio limits.

*The numbers in parentheses refer to the list of references at the end of this appendix.
Fig. X6.1. Applicability of damping values relative to ratio of insulation and pipe system weights and pipe size.

Fig. X6.2. Recommended pipe damping values for heavily insulated pipe.
For larger pipe sizes, or piping which does not meet the insulation weight ratio limits of Fig. X6.1, the frequency dependent damping values recommended by the Pressure Vessel Research Committee (PVRC) may be used as shown in Fig. X6.2. When the insulation weight ratio exceeds the minimum values shown in Fig. X6.2, higher damping values for seismic design can be determined experimentally if desired. The following recommendations made by the PVRC for light water reactor (LWR) pipe, are also acceptable for LMFBR pipe.

- Perform a test on a similar (nearly identical) system to determine the damping for each mode. The damping allowed for each mode of the system in question would be either:

  1. $\frac{2}{3}$ of the mean value of damping found for that mode in the test system, or
  2. $\frac{2}{3}$ of the damping for that mode taken from a smoothed curve of the test data when plotted graphically to show the relationship between damping and frequency.

- Perform a test on the actual system in question to determine the damping for each mode. When multiple values of damping for each mode are determined, the mean value of damping for each mode may be used.

### X6.3 BACKGROUND

Nuclear Regulatory Commission Guide 1.61 (2) presents the current NRC criteria for damping values to be used in dynamic analyses of nuclear power plant piping systems. The percent of critical damping for OBE and SSE load conditions, as permitted by Reg. Guide 1.61, are presented in Table X6.1. The damping values of Table X6.1 were established on a conservative basis such that they are applicable to a variety of piping systems. The Reg. Guide 1.61 allows damping values higher than Table X6.1 if specific test data substantiation are provided to support the higher values.

<table>
<thead>
<tr>
<th>Table X6.1. Current Damping Criteria, Piping</th>
<th>OBE</th>
<th>SSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large Diameter Piping</td>
<td>2%</td>
<td>3%</td>
</tr>
<tr>
<td>D &gt; 12 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Small Diameter Piping</td>
<td>1%</td>
<td>2%</td>
</tr>
<tr>
<td>D \leq 12 in.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reference: U. S. NRC Guide 1.61 (2)

OBE = Operating Basis Earthquake
SSE = Safe Shutdown Earthquake
Typical sodium pipe systems in LMFBR's operate at higher temperatures than similar systems in LWR's. For this reason, LMFBR's use a thicker insulation package to reduce heat loss. Additionally, sodium systems utilize trace heaters to maintain the pipe above the coolant freezing temperature. These trace heaters are often installed in an annulus between the pipe wall and insulation which is also used for leak detection purposes.

A typical LMFBR stand-off insulation package design is shown in Fig. X6.3. Such an insulation package was used for the Fast Flux Test Facility (FFTF), was planned for the Clinch River Breeder Reactor (CRBR) and is used on the INTERATOM SNR-300 LMFBR's.

The insulation package and trace heater weight is a significant fraction of the total LMFBR pipe system weight in sodium systems, particularly for small diameter piping. As shown in Fig. X6.4, this fraction exceeds 30% for pipe 8-in. or less in diameter.

Design of sodium pipe systems requires the use of more dead weight and seismic supports than typical for LWR piping because of this added insulation weight. These additional supports, in combination with relative motion between the pipe and insulation, dissipate energy during a seismic event and result in increased system damping.

These influential features, effecting system damping of LMFBR piping, have been assessed by laboratory and in-situ testing. Westinghouse Hanford Company (WHC) has completed laboratory tests on prototypic FFTF and LMFBR 1-in. diameter pipe loops. Westinghouse Hanford Company has also completed in-situ testing of typical FFTF 1-in. and 3-in. diameter piping systems. These tests indicated that system damping is much larger than presently assumed in design calculations. A similar laboratory test has been conducted on an 8-in. diameter scale model of a 24-in. LMFBR pipe system by the Westinghouse Advanced Energy Systems Division (W-AESD). Results also indicate that system damping is higher than NRC criteria. Recently published Japanese data show even lightly insulated piping to have significantly higher damping than uninsulated piping.

The available test data related to damping in insulated LMFBR pipe systems is reviewed herein. Recommendations are then made for the use of increased damping values for design of LMFBR sodium pipe systems.

A number of test programs, both U. S. and foreign, have been performed to measure the damping values associated with the special design features of LMFBR piping systems. These tests have included
Fig. X6.3. Typical LMFBR stand-off insulation and trace heat design.

Fig. X6.4. Insulation weight ratio for various pipe diameters stand-off insulation system.
laboratory tests of simple pipe spans, laboratory tests of prototypical pipe loops, and in-situ testing of typical LMFBR piping systems equipped with stand-off insulation. These test programs are discussed below.

X6.4.1 WHC Prototypic 1-in. Diameter Piping Test Loops, Test 1. Westinghouse Hanford Company has performed laboratory tests on a prototypic 1-in. diameter 316 stainless steel schedule 40 piping test loop. The test system consisted of approximately 40 ft of schedule 40 pipe, with several bends and risers. The pipe was anchored at each extremity and supported at 11 intermediate locations with 6 rigid struts and 14 mechanical snubbers. Fast Flux Test Facility insulated pipe clamps were used at the intermediate supports and the pipe was insulated in a manner identical to that used in FFTF. A pipe mounted sodium valve was included in the loop. The entire pipe loop was mounted to a rigid strongback through which dynamic loads were applied. Testing was performed at room temperature.

Snapback tests were conducted to determine actual system damping. Measured percent of critical damping is shown in Fig. X6.5 as a function of system response frequency [from Fig. 3 (3)]. The piping loop was supported at 11 intermediate locations and test results are shown for a support configuration of (1) snubbers and rigid struts, and (2) all rigid struts. Also shown in Fig. X6.5 is the special case of no intermediate supports on the piping test loop. The data do not indicate a significant reduction in damping with increasing frequencies.

![Damping Data](image)

Fig. X6.5. Damping data, WHC one-inch diameter piping test loop, reference F3.)
Results indicate that the case of piping supported by a combination of rigids and snubbers has substantially higher damping than the case of all rigid strut support. The average damping factor is 15.3 percent for the data shown in Fig. X6.5 (not including unsupported piping data). The range of damping is 9 to 27 percent.

X6.4.2 WHC Revised 1-in. Diameter Piping Test Loop, Test 2. Westinghouse Hanford Company decided to conduct a second series of tests on this loop to better understand the reasons for the high damping values observed (4 and 5). However, it was recognized that the 11 intermediate support configurations was typical of early designs which had not been optimized. The support configuration was therefore modified to represent more recent LMFBR design with a minimum number of intermediate supports. The number of intermediate supports was reduced from 11 to 4.

A series of tests were then conducted to evaluate damping for several different pipe and support configurations (5). A horizontal impulse load was used as the exciting force for these tests. The full amplitude impulse load was selected to provide an acceleration equivalent to the appropriate FFTF response spectrum peak acceleration for this pipe location.

The impulse tests were intended primarily to measure the piping system damping characteristics and secondarily to identify the predominant vibration mode frequencies. These tests were accomplished by applying short duration impulsive loads and then allowing the piping system to oscillate in free vibration after instantaneous removal of the load. The damping coefficients were then determined by evaluating the logarithmic decrement from the piping time domain response decay curves.

The system damping and the predominant response frequencies for each piping loop configuration tested at three impulsive excitation levels are summarized in Table X6.2. These data are plotted in Fig. X6.6. The average damping is 10.9 percent (not including bare pipe data). The range is 7 to 16 percent. Again, the data does not indicate a significant frequency influence.

X6.4.3 WHC In-Situ Tests of Small Diameter FFTF Piping. In-situ dynamic tests have been performed on 3 FFTF small-diameter pipelines to obtain damping data. The lines were 1-in. and 3-in. in diameter, were fabricated of 316 stainless steel material and were supported by rigid struts, seismic snubbers, and spring hangers. Insulated standard FFTF pipe clamps were also used. The piping was constructed with standard FFTF trace heat elements and standoff insulation installed. The two 1-in. lines tested were 24 and 30 ft. in length, while the 3-in. line was in excess of 100 feet in length as shown in Figs. X6.7, X6.8, and X6.9.
Table X6.2. Piping Loop Damping Coefficients and Predominant Response Frequencies at Three Impulsive Excitation Levels [Severud et al. (5)].

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Configuration</th>
<th>1/4 Amplitude Impulsive Load</th>
<th>1/2 Amplitude Impulsive Load</th>
<th>Full Amplitude Impulsive Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Coeff. % Freq. Hz</td>
<td>Coeff. % Freq. Hz</td>
<td>Coeff. % Freq. Hz</td>
</tr>
<tr>
<td>1.1.2</td>
<td>Support bearing gaps = 1.5 mm (.060 in.), Baseline .015 in gaps</td>
<td>10.6 9.4</td>
<td>11.0 9.4</td>
<td>12.5 9.4</td>
</tr>
<tr>
<td>1.2.2</td>
<td>Chain support at No-e 11-2, Simulated Strut Buckling</td>
<td>12.5 7.5</td>
<td>13.6 7.5</td>
<td>16.2 7.5</td>
</tr>
<tr>
<td>1.3.2</td>
<td>Clamp preload at 50% of Design Baseline</td>
<td>13.8 6.0</td>
<td>12.8 6.0</td>
<td>13.9 6.0</td>
</tr>
<tr>
<td>1.4.2</td>
<td>Clamp preload at 150%</td>
<td>9.3 6.0</td>
<td>9.4 6.0</td>
<td>9.6 9.4</td>
</tr>
<tr>
<td>1.5.2</td>
<td>Supported by 4 Struts and 3 Snubbers</td>
<td>9.3 9.4</td>
<td>10.3 9.4</td>
<td>12.0 11.9</td>
</tr>
<tr>
<td>1.6.2</td>
<td>222-4 (90-1b) Trapezoid Weight at Node 11</td>
<td>7.3 6.0</td>
<td>7.4 9.4</td>
<td>10.0 11.9</td>
</tr>
<tr>
<td>1.7.2</td>
<td>Supported by 7 Rigid Struts, Insulated Pipe</td>
<td>7.9 6.0</td>
<td>8.5 6.0</td>
<td>9.3 6.0</td>
</tr>
<tr>
<td>2.7.2</td>
<td>Supported by 7 Rigid Struts, Bare Pipe</td>
<td>4.2 12.7</td>
<td>4.3 12.7</td>
<td>4.5 23.3</td>
</tr>
</tbody>
</table>

Fig. X6.6. Damping data, WHC one-inch diameter piping test loop [Severud et al. (5)].
Fig. X6.7. System 61, Line 1" GEA 61176.

Fig. X6.8. System 61, Line 1" GEA 61177.
A total of 13 different pipe system support configurations were tested. This was accomplished by removing supports at selected locations to change the system natural frequencies. All testing was accomplished at room temperature using the snapback technique. Test details are reported in Lindquist (6).

Snapback load levels were selected to limit maximum pipe stress to about 1/2 of yield to protect the pipe. Testing was conducted at 33%, 67%, and 100% at the selected load levels. Data showed a definite increase in damping with increasing load level. However, only that data from the 100% load level test is summarized herein since a stress level of 1/2 yield is less than that which is normally allowed for an OBE seismic event. The data are conservative for an SSE event.

The damping data obtained from this test series [taken from Tables 8-1 through 8-13 of Lindquist (6)] are summarized in Fig. X6.10. Data points from all duplicate tests are plotted at the predominant response frequencies of the pipe systems. This provides a measure of the repeatability of test results. Results of both vertical and horizontal excitation are shown. Average damping was 10.1 percent for 1-in. pipe and 10.4 percent for 3-in. pipe with no significant change with frequency.

X6.4.4. W-AESD 8-in. Diameter Piping Test Loop. Dynamic tests have been performed by W-AESD on an 8-in. diameter piping test loop. The pipe loop was approximately 38 ft. long, was constructed of 304 stainless steel and was supported at 7 locations as shown in Fig. X6.11.
Fig. X6.10. Damping data, FFTF in-situ test, insulated small diameter pipe.

Fig. 6.11. Schematic of 8-in. W-AESD test loop.
The test piping loop was an approximate, 1/3 size representation of the CRBRP primary crossover piping leg which interconnected the primary sodium pump and intermediate heat exchanger. The test piping loop preserved the diameter-to-thickness ratio and configuration of the primary crossover leg to maintain appropriate flexibilities. Both insulation thickness and trace heater annulus size were also scaled down to provide a proper scale weight distribution of the primary crossover leg. This resulted in an insulation weight to pipe weight ratio typical of 24-in. LMFBR pipe which is much less than that for typical 8-in. LMFBR pipe, as indicated in Fig. X6.3. Damping data obtained from this test would therefore be lower than expected for typical 8-in. LMFBR pipe.

To assess changes in damping as a function of piping configuration, multiple tests were performed. Damping tests were conducted with the piping system insulated and uninsulated. During all tests, the pipe was filled with water to simulate the weight of sodium in LMFBR piping systems. The type of seismic restraint was also varied during the test program. The piping system was restrained by either 5 rigid struts or 5 mechanical snubbers.

Snapback and sine-sweep tests were performed to obtain damping values for the piping system. The two different types of tests were chosen to reinforce the results determined from each test type. Performing different tests also enabled better evaluation of damping as a function of frequency and excitation amplitude. Damping results are presented in Hulbert et al. (7).

Damping data resulting from tests performed on the 8-in. diameter piping system are summarized in Table X6.3. The average values were calculated from all usable snapback and sine-sweep data, averaging any damping variation due to test type, excitation frequency or load level. As indicated, adding insulation to the pipe for the rigid strut restrained configuration changed the average damping from 4.4 percent to 5.3 percent for a 20 percent increase. Changing from struts to snubbers increased the average system damping an additional 23 percent to 6.5 percent of critical. Although these increases are not as large as these effects on smaller pipe, the damping values are still substantially above those quoted by the present Reg. Guide, Table X6.1.

Table X6.3. W-AESD 8-in. Pipe Damping Test Results

<table>
<thead>
<tr>
<th>Restraint Type</th>
<th>Damping, Percent of Critical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uninsulated Pipe</td>
</tr>
<tr>
<td>Rigid Strut</td>
<td>4.4</td>
</tr>
<tr>
<td>Mechanical Snubber</td>
<td>4.5</td>
</tr>
</tbody>
</table>

1 Includes dead weight supports.

2 Mean value calculated from all usable data.
Snapback and sine-sweep tests were performed at various excitation amplitudes to assess the effect on damping. The damping results did not significantly vary with excitation level for this larger pipe. However, piping system damping was found to be inversely proportional to frequency as shown in Fig. X6.12 for the insulated pipe condition.

X6.4.5 Japanese Damping Test Data. The Japanese nuclear industry has performed extensive testing on LWR piping systems to measure damping values. The purpose of these tests was to identify and quantify the contributors to pipe system damping. These tests are detailed in (8) through (11).

This Japanese LWR test data is included herein because it further confirms that insulation and seismic snubbers provide significant contributions to overall pipe system damping.
Figure X6.13 shows the effect of approximately 2-in. of calcium silicate insulation on a 4-in. pipe. The data for insulation with a gap correspond to the LMFBR insulation design with a trace heat/leak detection annulus. Even though the insulation thickness is less than 1/2 that used on LMFBR pipe, the average damping is about 7 percent. The data also indicate a trend for increased damping with increasing excitation amplitude. It should also be noted that insulation without a gap results in lower damping values.

Figure X6.14 shows similar results for 3-in. of calcium silicate on 8-in. pipe. Here, the average damping is about 2 percent with half the typical LMFBR insulation thickness. The Japanese data confirm that system damping is large for small diameter insulated pipe and decreases as pipe and contents weight increases relative to insulation weight.

Figure X6.15 illustrates the damping effect of Japanese mechanical snubbers. The data confirm that substantial damping is provided by these devices which are similar to the mechanical snubbers used on US-LMFBR's.

Fig. X6.13. Japanese LWR data - damping effect of thermal insulation on 4-in. pipe [Fig. 14 (11)].
Fig. X6.14. Japanese LWR data — damping effect of thermal insulation on 4-in. pipe [Fig. 15 of Shibata et al. (11)].

Fig. X6.15. Japanese LWR data — damping effect of thermal insulation on 8-in. pipe [Fig. 19 of Shibata et al. (11)].
X6.5. APPLICABLE TEST DATA (STRAP-ON INSULATION)

The test data presented in Sec. X6.4 of this report is based on pipe systems equipped with stand-off insulation. The Japanese data indicates that a gap or stand-off between the pipe wall and insulation package contributes increased damping. Future LMFBR pipe systems may not all use stand-off insulation. For this reason, WHC undertook an additional test program to quantify the effect on damping of removing the stand-off from the insulation design.

X6.5.1 Strap-on Insulation Test Series. The FFTF No. 1 Closed Loop Module (CLM) is equipped with pipe systems using direct strap-on insulation as shown in Fig. X6.16. These sodium pipe systems are made of Schedule 40 316SS and employ insulated pipe clamps identical with other FFTF pipe systems.

Fig. X6.16. Strap-on insulation, CLM No. 1 piping.
A series of snapback tests were conducted on a 1-in. pipeline, Fig. X6.17, and a 3-in. pipeline, Fig. X6.18, Lindquist (12). Testing was conducted at 33%, 67%, and 100% of the selected load level. Data indicated increased damping with increasing load level. However, the 100% load level was limited to produce a minimum pipe stress of 1/2 yield. This is approximately equal to half of the stress levels normally allowed for an OBE event. The data are therefore considered conservative for an SSE event.

Fig. X6.17. CLM No. 1 pipeline 35 (1").
The CLM is not in service. All testing was conducted at room temperature. The unit is in the "as-received" condition with rigid struts installed in place of snubbers for shipping purposes. The pipe systems were initially tested in this configuration. A single seismic snubber was then installed at the locations indicated in Figs. X6.17 and X6.18 and the pipe retested.

Fig. X6.18. CLM No.1, pipeline 08 (3").
Test data is summarized in Fig. X6.19. Average damping was 6.8% with no significant variation with frequency. Because of the low amplitude input snapback load and the use of only a single snubber, this data is considered as a lower bound for pipe equipped with strap-on insulation. Nevertheless, the data confirm that use of strap-on insulation will result in higher damping than allowed by Reg. Guide 1.61 but less of an increase than obtained with stand-off insulation.

Fig. X6.19. Test results, damping vs frequency CLM no.1 with strap on insulation.

X6.6 CONCLUSIONS - LMFBR DAMPING DATA BASE

The available data base for insulated LMFBR pipe systems has been presented in Sections X6.4 and X6.5 of this report. Average damping and standard deviation for pipe equipped with stand-off insulation and with various support configurations is presented in Table X6.4. Similar data for pipe equipped with strap-on insulation is presented in Table X6.5. Insulation weight ratios for the various test systems are shown in Fig. X6.20.
Table X6.4. Summary of Standoff Insulation Results.

<table>
<thead>
<tr>
<th>Pipe Configuration</th>
<th>Number of Data Points</th>
<th>Mean % Critical</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-In. Supported with Rigid Struts Only</td>
<td>8</td>
<td>10.2</td>
<td>2.5</td>
</tr>
<tr>
<td>1-In. Supported with Both Snubbers and Rigid Struts</td>
<td>110</td>
<td>11.3</td>
<td>4.4</td>
</tr>
<tr>
<td>3-In. Supported with Both Snubber and Rigid Struts</td>
<td>16</td>
<td>10.4</td>
<td>5.4</td>
</tr>
<tr>
<td>1-3 In. All Data</td>
<td>134</td>
<td>11.1</td>
<td>4.5</td>
</tr>
<tr>
<td>8-In. with Rigid Struts</td>
<td>235</td>
<td>8.5</td>
<td>1.8</td>
</tr>
<tr>
<td>8-In. with Snubbers</td>
<td>325</td>
<td>6.5</td>
<td>3.2</td>
</tr>
<tr>
<td>8-In. All Data</td>
<td>560</td>
<td>6.0</td>
<td>7.7</td>
</tr>
</tbody>
</table>

Notes: ① Vertical supports typically rigid struts with mechanical snubbers for lateral seismic support.
② 1/3 scale model of 24-in. pipe. Insulation typical of 24-in. pipe.

Table X6.5. Summary of Strap-on Insulation Test Results.

<table>
<thead>
<tr>
<th>Pipe Configuration</th>
<th>Number of Data Points</th>
<th>Mean % Critical</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-In. Supported with Rigid Struts Only</td>
<td>8</td>
<td>5.8</td>
<td>0.7</td>
</tr>
<tr>
<td>1-In. Supported with Rigid Struts Plus One Snubber</td>
<td>16</td>
<td>8.4</td>
<td>1.3</td>
</tr>
<tr>
<td>3-In. Supported with Rigid Struts Only</td>
<td>24</td>
<td>6.0</td>
<td>0.6</td>
</tr>
<tr>
<td>3-In. Supported with Rigid Struts Plus One Snubber</td>
<td>28</td>
<td>7.1</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Weighted Average All Data = 6.8%
The following conclusions are made relative to damping in LMFBR pipe systems from a review of this data.

1. All tests confirm that the presence of thermal insulation increases damping significantly, particularly on small diameter pipe.

2. The damping appears to increase as the insulation weight ratio increases.

3. The presence of seismic snubbers increases damping significantly, particularly on small diameter pipe.

4. Variation in damping with response frequency is insignificant in small size pipe but becomes significant with larger pipe sizes.
5. Damping increases with excitation level for the small diameter pipe systems.

6. Most testing has occurred at piping stress levels somewhat below material yield strengths. Data are therefore appropriate for OBE conditions and conservative for SSE conditions, since local plasticity greatly increases damping (13).

7. Test data indicates that damping contributed by insulation and supports tends to decrease as pipe size increases.

8. All test data demonstrate that insulated LMFBR pipe systems have higher damping than presently recommended by NRC Reg. Guide 1.61 (2).

9. A sufficient data base has been generated to justify recommending higher damping values for the seismic design of insulated LMFBR pipe systems.

X6.7. PVRC DATA/BASE FOR LWR PIPE DAMPING

The PVRC has recommended the use of increased damping values for the seismic design of LWR pipe systems. These recommendations have been reviewed and approved by the NRC Pipe Review Committee. The PVRC damping recommendations are based on mean damping values. Detailed justification for this position was included in the PVRC Interim Summary Report which is included in Appendix A of Anderson et. al. (1).

Data on damping obtained from testing of uninsulated LMFBR pipe systems was included in the PVRC data base. However, data from heavily insulated LMFBR tests was recognized by PVRC to have higher damping than their recommendations but was judged atypical of LWR pipe and was not included in the PVRC data base.

Small diameter LMFBR pipe systems differ from LWR systems only in that they operate at higher temperatures which require the use of more insulation and consequently more seismic supports. The seismic analyses techniques are identical.

The PVRC mean damping value justification analyses are therefore judged to be equally applicable to the LMFBR insulated small pipe systems. This is further confirmed by Barta et al. (4) and Hulbert et al. (7) which compare LMFBR pipe test results with response spectrum analyses, using mean value damping, and confirms the analyses results are conservative.

The PVRC recommendations for damping in LWR pipe systems are summarized below for information.
X6.7.1 PVRC Technical Position on Damping Values for LWR Piping.

The results of the data evaluations clearly provide support for increasing the present damping values for seismic design of nuclear power plant piping specified in Reg. Guide 1.61. Based on these data evaluations, the position of the Task Group members is as follows.

• Changes in the present damping values, with significant increases in the lower frequency range, are justified and recommended to provide a more balanced design between thermal and seismic requirements.

• Correlations of damping with frequency are indicated by (a) the results of the regression analysis completed to date and (b) engineering assessment of all data. Possibilities of dependency on acceleration or stress levels, support types, insulation, and modal number were also observed. Pipe diameter does not appear to be an important parameter.

• Numerically, the damping value recommended by the Task Group is 5% to 10 Hz, linearly decreasing to 2% at 20 Hz and held constant at 2% to 33 Hz. Figure X6.21 illustrates the position in relation to present Reg. Guide 1.61 values. The recommendation is applicable to both OBE and SSE and is independent of pipe diameter.

• The numerical recommendation is based on mean data. This mean-data-based damping recommendation has been critically evaluated by independent assessments to assure that acceptable overall margins are maintained.

• The damping recommendations have been studied using the response spectrum method of analysis. However, the basic data are independent of analysis method.

• Additional damping for specific parts of the total system, like snubbers and supports should not be utilized unless one of the testing options described below has been implemented.

In addition to the generic data-based conclusions noted above, higher damping values for seismic and damping values for other excitation forces can be determined experimentally. The following methods are acceptable:

• Perform a test on a similar (nearly identical) system to determine the damping for each mode. The damping allowed for each mode of the system in question would be either:
Fig. X6.21. Graphical representation of the PVRC technical position on damping values for LWR pipe.

(1) $2/3$ of the mean value of damping found for that mode in the test system, or

(2) $2/3$ of the damping for that mode taken from a smoothed curve of the test data when plotted graphically to show the relationship between damping and frequency.

Perform a test on the actual system in question to determine the damping for each mode. When multiple values of damping for each mode are determined, the mean value of damping for each mode may be used.
A considerable body of laboratory and in-situ test data are available which demonstrates that damping in heavily insulated LMFBR sodium piping is much larger than presently recommended in the NRC Reg. Guide 1.61, (2) and Table X6.6. This is particularly true for small diameter pipe systems.

In developing recommended damping values for insulated LMFBR pipe systems, several factors are considered.

1. The recommendations should cover both stand-off and strap-on insulation designs.
2. The recommendations should cover pipe seismically supported by rigid struts, snubbers, and by combinations thereof.
3. The recommendations should be easily understandable and implemented by production pipe designers.

In studying the data base, it is noted that the variation in damping with frequency, observed by the PVRC Task Group, is apparently offset by insulation effects for small diameter pipe. A slight frequency effect did become apparent in the W-AESD 8-in. pipe data. It is also noted that damping in the 8-in. pipe approaches the values recommended by the PVRC. For these reasons, the increased damping recommendations are applicable for pipe size 8-in. and smaller only.

Most of the available damping data was obtained at low pipe stress levels. However, it was demonstrated that damping increased with excitation level for small diameter pipe. The data is reasonable for OBE excitation and conservative for SSE excitation levels. The damping recommendations should be applicable for both of these seismic excitation levels.

The mean test results, summarized in Tables X6.4 and X6.5 show large standard deviations. This is to be expected since the data base includes measurements parallel and perpendicular to the applied load direction and data obtained both near the load point and remote from it. It also includes data for several support configurations. Such data is not amenable to statistical evaluation. However, since damping varies with displacement, mode shape, and frequency; and since most pipe system seismic analyses use the response spectra technique where both vertical and horizontal mode participation and excitation are combined, it would appear reasonable to use mean damping values.

Such an approach has been used and justified in PVRC recommendations for LWR pipe damping [Sec. X6.7 and (4 and 7)] and is deemed technically appropriate for LMFBR also.
The mean value damping data from Tables X6.4 and X6.5 are plotted against pipe size in Fig. X6.22. Three trends become apparent from this plot. First, insulated pipe damping decreases with increasing pipe size and approaches the PVRC recommended damping values for LWR pipe at 8-in. Second, strap-on insulation produces less damping than stand-off insulation. Third, pipe supported with snubbers has more damping than pipe supported by rigid struts.

![Graph of damping vs pipe size](image)

**Fig. X6.22.** LMFBR mean damping vs pipe size with insulation.

In selecting a recommended damping value for the 1-in. pipe size, it is noted that the mean value for pipe with strap-on insulation and supported with rigid struts is quite low. Adding a single snubber produces much larger damping. Since heavily insulated pipe operates at elevated temperatures, it is considered reasonable to assume that at least one snubber will be required on such pipe systems to accommodate thermal motion. It should also be noted that the strap-on insulation tests were conducted at pipe stress levels equal one-half those allowed for an OBE event. For these reasons, a damping value of 8% is recommended for 1-in. and smaller heavily insulated pipe systems.

At the 8-in. size, the PVRC recommendation of 5% damping is less than the mean value observed for stand-off insulated pipe with both snubbers and rigid struts. The 5% value is also a lower bound for uninsulated pipe, should be conservative for pipe equipped with strap-on insulation, and is recommended.
A straight line interpolation between 8% damping for 1-in. and 5% damping at 8-in. is recommended for intermediate pipe sizes. As shown in Fig. X6.22, this approach produces a good fit with the 3-in. test data.

It is necessary to define "heavy insulation" in order to implement these damping recommendations. This can be done by referring to the insulation weight ratio plot of Fig. X6.20. By constructing a curve through the lowest insulation weight ratios for the test pipe systems, a limit curve can be derived above which the heavily insulated damping recommendations are applicable. This curve is shown in Fig. X6.1.

For those pipe systems which do not meet the limiting insulation weight ratios of Fig. X6.1, or are larger than 8-in. in diameter, it is recommended that the PVRC damping values for LWR pipe be utilized. These recommendations are summarized in Fig. X6.2.

The relationship of these recommendations to the heavily insulated LMFBR data base is shown in Fig. X6.23.

Fig. X6.23. Recommended damping values in relation to all U. S. LMFBR test data.
For pipe systems whose insulation weight ratio exceeds the minimum of Fig. X6.1, higher damping values for seismic design and damping values for other excitation forces can be determined experimentally if desired.

The recommended methods specified by PVRC for LWR pipe and given in Sec. X6.7 are also acceptable for the determination of damping on LMFBR piping systems.

REFERENCES


APPENDIX - NONMANDATORY

X7. SEISMIC ANALYSIS OF FAST REACTOR CORE STRUCTURES

X7.1 SCOPE

This appendix provides recommended guidelines for the analysis of the seismic response of the large arrays of fuel, blanket, control, and reflector assemblies typical of fast reactor cores. The essential feature of such cores is that the assemblies are not individually supported; rather their lateral support, except at the inlet nozzle, is provided by spacer pads between adjacent assemblies and, optionally, peripheral restraint rings attached to the core barrel. For this reason it is necessary to analyze the dynamic response of the whole array to determine forces and displacements of individual assemblies. The only comparable situation in thermal reactors is that of the high temperature gas reactor (HTGR) where the core consists of a large array of graphite blocks.

The guidelines are based on experience of a core seismic analysis development program conducted at Argonne National Laboratory (ANL) during the period 1976 through 1983. It begins with a description of the nature of the problem and an introduction to the literature of U.S. and foreign efforts in core seismic analysis and testing. This is followed by detailed recommendations in the areas of analysis methods, modeling of physical phenomena, input time histories, and response measures based on the experience gained in the ANL program. Finally, specific guidelines for analysis and testing are given. A comprehensive bibliography is provided.

X7.2 SUMMARY

Two methods of analysis are recommended for use in evaluating the seismic response of fast reactor cores. The first is a simplified impulse-momentum approach suitable for core designs with peripheral restraint rings and intended for use in scoping studies, verification of detailed analysis and interpretation of results.

The second is a nonlinear time-history numerical-simulation approach. Several levels of detail are suggested for different portions of the core seismic analysis with a model of a diagonal row of the core recommended as the primary detailed analysis tool. Recommendations for modeling structural stiffness, mass, fluid coupling, gaps, and impacts are included along with discussions of support acceleration time histories, duration of simulations, integration time steps and response measures. The core seismic analysis code SCRAP, developed at ANL and modified by GE-ANTO, is particularly suited for these analyses but other dynamic analysis codes could also be adapted.
An updated version of the computer program SCRAP (9 and 10)* configured for the analytical tool mode rather than for a research tool, is currently available from the National Energy Software Center (NESC) at ANL, Argonne, Illinois.

Two types of tests are recommended to support the analysis: sine assembly tests intended to identify assembly vibration characteristics in fluid and impact tests intended to identify coefficients of restitution and duration of impacts.

X7.3. PROBLEM DESCRIPTION

The seismic response of fast reactor cores is a vibration-impact problem involving a large number of components (1000 assemblies). The response is strongly nonlinear due to the closing of gaps (impacts) between assemblies. Both the vibration and impact aspects are strongly influenced by the presence of coolant (liquid sodium) in the narrow gaps between assemblies. The core array experiences a seismic excitation which has been amplified and filtered by the building, reactor vessel, and core support structure to produce a narrow band excitation of as much as 1-2 g peak acceleration in the 2-10 Hz range.

Two simplifying assumptions are generally made: 1) the vertical component of seismic excitation, which must be considered to avoid "liftoff" of the assemblies from the lower support, does not couple with the horizontal motion of the core array, and 2) the coolant between assemblies is constrained from significant vertical flow by the support plate and one or more levels of spacer pads so that its flow is planar.

Designers are principally concerned with damage to the assembly hexagonal ducts due to impacts, alignment between the core and control drivelines, and potential reactivity effects due to relative motion of assemblies. Of less concern is bending stresses in the assemblies. Knowledge of the typical assembly response is also needed for analysis of the fuel pin bundle within the assembly.

The important phenomena that must be considered include:

1) natural frequencies and damping of the individual assemblies;
2) effects of gaps at the nozzle support on assembly frequencies;
3) impact characteristics between assemblies including the effect of coolant "squeeze-film";
4) forces transmitted between assemblies by the coolant inertia and damping;

*The numbers in parentheses refer to the list of references at the end of this appendix.
5) gaps between assemblies at load pads and their distribution due to assembly bow;

6) the effects of manufacturing tolerances on the total effective gap between assemblies; and

7) the direction of seismic excitation relative to the triangular pitch of the array.

A major hurdle which must be overcome in obtaining simulations of the response of fast reactor cores relates to the vibration-impact nature of the problem. The vibrations have periods of 100 to 500 ms while the duration of the impacts are of the order of 1 ms. Thus there are two characteristic time scales for the problem: a fast time related to impacts and a slow time related to vibrations. Such problems are termed "stiff" and are notoriously consumptive of computer time for simulations. Since a very large number of impacts occur within the array during only a few oscillations many special techniques for solving such problems are not applicable. The result is that there is a great incentive to reduce the number-of-degrees-of-freedom in a mathematical model so that simulation is feasible.

X7.4. FAST REACTOR CORE SEISMIC LITERATURE

X7.4.1 Fast Flux Test Facility (FFTF). The first significant attempts to address this problem were for the FFTF. A dynamic simulation of the reactor vessel including lumped core components and gaps was used. Impact forces from this model were used in scram analysis and design calculations for the core load pads. Morrone (1) and Julyk and Hecht (2) document this work.

X7.4.2 Clinch River Breeder Reactor (CRBR). A dynamic simulation was also used for scram analysis of the CRBR but an equivalent static approach was used to estimate seismic loads on load pads. In this approach the load distribution for the array of load pads was calculated for a 1 g static load and this was multiplied by the peak core acceleration to give seismic loads for design calculations. References (3) through (5) document aspects of this work.

X7.4.3 Argonne National Laboratory Core Seismic Analysis Development Program. Argonne National Laboratory conducted a base program to develop a core seismic analysis method during the period 1976-1983. This program resulted in a special purpose computer code, SCRAP, for core seismic analysis based on assumed mode modeling techniques. The SCRAP code will conduct a time history analysis of a core array with up to 100 degrees-of-freedom. It includes modeling of fluid-structure coupling between assemblies and fluid effects during impact and allows motion of the assembly and seismic impact in the horizontal plane. Verification and qualification of the code using
Japanese full scale tests have been conducted. In addition, various sensitivity studies and design studies have been conducted using SCRAP. References (6) through (24) document this work.

X7.4.4 General Electric Core Seismic Development Program. General Electric Company (GE), Advanced Nuclear Technology Operations (ANTO) has had long standing interest in core seismic analysis problems and has a current effort in this area. They have made modifications to the SCRAP program to improve calculation times and facilitate designer use. They have also analyzed Japanese test data to determine its applicability to U.S. designs. Fakhari and Fox (25), and (40) give partial documentation to this effort but the program is continuing.

X7.4.5 Gas Cooled Fast Reactor (GCFR). The GCFR has many of the same problems as the LMFBR with the exception of the fluid effects. References (26) and (27) document aspects of core seismic analysis and testing for this design.

X7.4.6 Japanese Test Programs. The only significant test programs for LMFBR core seismic response have been conducted by the Japanese Power Reactor and Nuclear Fuel Development Corporation for the MONJU reactor. These tests include full scale tests of 29 assemblies in a row and 37 assemblies in an array both in air and water. References (28) through (32) document the Japanese tests and related analysis.

X7.4.7 European Literature. Open literature reports of European efforts on core seismic analysis and testing are sketchy. It is known that the Belgians have been investigating fluid coupling effects for fast reactor cores and the French and Italians have collaborated in developing a core seismic analysis computer code. References (33) and (34) report some of this work.

X7.4.8 High Temperature Gas Reactor (HTGR). While the HTGR core is substantially different from a fast reactor core, many of the problems of impact response of a large array are similar. There is a large amount of literature dealing with tests and analysis of this problem and References (35) and (36) document recent reports of this work.

X7.4.9 Other Related Literature. Agbabian Associates conducted an assessment of core seismic analysis procedures in 1977 (37) and the Breeder Reactor Seismic Technology plan has documented core seismic analysis needs (38 and 39). Progress of the ANL and GE programs have been documented in the Seismic Design Technology Programs Semiannual Progress Report series (40).
X7.5. MODELING RECOMMENDATIONS

X7.5.1 Analysis Methods.

X7.5.1.1 Equivalent Static Method. One simplified method of seismic analysis for nuclear reactor components is to assume a static force on the component. This is normally assumed to be

\[ F_s = 1.5 \cdot W \cdot A_s \]  \hspace{1cm} (X7.1)

where \( F_s \) is the equivalent static force;
\( W \) is the weight of the system including liquid contents;
\( A_s \) is the maximum peak acceleration of the response spectra which apply at the points of support of the structural system.

Since the peak of the response spectra at the core support is normally quite high (10's of g's) this method can give rather large forces. Moreover, it is not clear that such a method is conservative for the dynamic loads on a core which is dominated by impacts. This method is not recommended for core seismic analysis.

X7.5.1.2 Response Spectrum Methods. Another commonly used method of seismic analysis for reactor components is the response spectrum approach. The response spectrum method is inherently a linear method and is only suitable for components whose response is primarily a linear vibration. The response of a constrained fast reactor core (a core with peripheral restraint rings) is dominated by the impacting of the core array against the restraint rings and is strongly nonlinear. For such cores the response spectrum approach is not recommended. However, for cores without peripheral restraint, sometimes called "free flowering" cores, this method may be of limited value because the dominant response is a nearly linear vibration of the array when the motion is sufficiently strong to dominate nonlinear effects at the nozzle support.

X7.5.1.3 Nonlinear Time History Methods. The most general method of analysis commonly used for seismic analysis of reactor components is a time history method in which differential equations describing the component are numerically integrated using a simulated time history of the support acceleration. Several methods are available to develop discrete differential equation models of the core: lumped mass and stiffness models, finite element models, and assumed mode (Rayleigh-Ritz) methods are examples. Such models can include the gaps between assemblies, equations modeling the impact phenomena, as well as models of the fluid forces between assemblies. However, care must be exercised to limit the number of degrees-of-freedom retained in the model so that the resulting "stiff" differential equations can be integrated with available computers. For lumped mass and stiffness modeling this is done directly by limiting the number of masses and degrees-of-freedom per mass. For finite element models a substructuring approach
substructuring approach can be used. In the assumed mode approach the
number of modes used gives direct control of the degrees-of-freedom.
This is the recommended analysis method for detailed simulation of fast
reactor core seismic response. The specialized computer code SCRAP
(9 and 10) is particularly recommended because it contains suitable
models of the fluid coupling between components and the fluid effects
during impact. SCRAP is also valuable because it is designed to
facilitate changing the number or degrees-of-freedom in the model so
that either detailed analysis with many degrees-of-freedom or scoping
studies with few degrees-of-freedom can be easily modeled.

X7.5.1.4 Simplified Impulse-Momentum Analysis. A simplified
design rule based on impulse-momentum analysis can be used for estima-
tion of impact loads for core designs with peripheral restraint. Two
assumptions, which have been generally confirmed by tests and dynamic
analysis of various core designs, are required:

1) The entire core moves together in a coherent fashion through
the available gap at the period of the support.

2) The core moves with nearly constant velocity between impacts
with the restraint rings.

The impulse momentum law then requires:

\[ F = \frac{2 \times M \times G}{T} \]  \hspace{1cm} (X7.2)

where \( F \) is the total impulse acting on the core during an impact with
the restraint ring.

\( M \) is the effective mass of the core including the entrained
coolant but not including a portion of the assembly
associated with the nozzle restraint.

\( G \) is the effective gap. It must account for the design gap,
manufacturing tolerances and the fact that the restraint
ring is moving due to seismic excitation.

\( T \) is the period of time the core requires to transmit the
effective gap. \( T \) must be less than half the period of the
support excitation but experience has shown a value of 1/4
to 1/8th the support motion period is more appropriate to
account for the time during which the core "rides along
with" the restraint ring.

Impulse-momentum analysis is recommended as a basis for understanding
impact loading of restrained core designs and as a useful tool in esti-
mating the effects of design options.
X7.5.2 Physical Phenomena.

X7.5.2.1 Structural Stiffness. Normal structural modeling techniques can be used to model the stiffness of the various components of the core. Special care should be used in modeling the nozzle stiffness and fixity, particularly for unrestrained core designs, because this strongly affects the natural frequencies of the individual assemblies. Several analyses have shown the internal stiffness of the pin to be negligible compared to the duct stiffness but shielding stiffness may be important depending upon the assembly design. Most analyses have assumed a simple beam bending theory for the assembly stiffness although a full Timoshenko beam model was once used in a SCRAP analysis to no avail. Core barrels have normally been assumed rigid although this assumption should be verified by comparing the barrel deflection under anticipated impulse loading to the available gap in the core. If the deflection is a significant fraction of the available gap a rigid assumption for the barrel is not valid.

X7.5.2.2 Structural Mass. Usual dynamic modeling assumptions regarding mass distribution should be used. The mass of the coolant contained within the assembly can be lumped to the assembly but the mass of the coolant between assemblies should be treated with special fluid inertia coupling models. The location of the fission gas plenum with its low mass density can have a significant effect on seismic response and should be modeled. If fluid inertia coupling is to be included in the model there is no advantage to a diagonal structural mass matrix so consistent mass modeling is appropriate.

X7.5.2.3 Structural Damping. Core assemblies have been found in Japanese full scale tests to have moderate structural damping; values of 2-3% are appropriate. However, except for unrestrained core designs, the structural damping in the assemblies does not have a major effect on the core seismic response.

X7.5.2.4 Structural Gaps. Because of the need to accommodate thermal expansion and to refuel the core, core designs incorporate gaps. For restrained core designs the most important aspect of the model is the total available gap within the restraint rings. This determines both the maximum displacements at the load pad levels, and indirectly the maximum bending stresses. It also determines, through the impulse momentum relationship, the magnitude of the impact loads on assemblies. While the design gap is readily modeled, the true effective gap is difficult to model. Any deviations in the shape of a hexagonal load pad due to manufacturing tolerance leads to a dislocation in the array which is biased toward increasing the space required for the load pads. Since the available gap is the difference between the space available within the restraint ring and the accumulated space required for the array, it is very sensitive to manufacturing tolerance.
The distribution of the individual gaps between assemblies which go to make up the total gap is not particularly important to the model but bowing of the assemblies, which tends to keep some gaps open and absorbs energy when the core is compacted under impulse, has been shown to significantly reduce peak impact forces (17). Modeling of this bowing effect requires detailed knowledge of the thermal and creep bowing of individual assemblies. With this information, bilinear springs in the gap impact model can account for assembly bow.

Gaps in the nozzle support can have a strong effect on the natural frequencies of individual assemblies causing the apparent frequency to increase with increasing amplitude of the motion (a stiffening effect). While this is not particularly important to the gross dynamics of restrained cores, it can significantly affect bending modes and thus bending stresses in the assemblies. It will also strongly affect the response of unrestrained or partially restrained cores.

Another gap effect which has not been treated in the literature is the potential for contact between ducts in the core region. Swelling and creep causes the duct wall to dilate during the life of the assembly reducing the gap between assemblies and adding a potential for impact during a seismic event. A complete seismic analysis of a reactor core should model this potential impact although previously reported analyses have not addressed the problem.

X7.5.2.5 Fluid Coupling Effects. The force transmitted by a fluid between two parallel walls with relative normal acceleration is inversely proportional to the gap between the walls. Since the core array contains thousands of such parallel faces with a narrow gap the fluid coupling phenomenon can have a dominate effect on the core dynamics. The recommended approach to considering the fluid coupling is to assume:

1) The coolant between assemblies is constrained to flow in the horizontal plane by the support plate and load pads.

2) The motion of the duct walls is small in comparison to the gap between ducts.

Under these assumptions the fluid forces can be shown to consist of a force proportional to the accelerations of the duct and its neighbors, termed the fluid inertia coupling coefficients, and a force proportional to the velocities, termed the fluid damping coupling coefficients. Such coefficients have been calculated for the hexagonal geometry of a reactor core (15).
It is recommended that fluid coupling between assemblies be included in seismic modeling of reactor cores because its effect is to prevent relative motion of assemblies, a phenomenon consistently observed in tests. The fluid damping coupling coefficients have been found to have negligible effect and need not be modeled. Moreover, inertia coupling to only the six nearest neighbors should be modeled since the timestep used in the numerical integration of the equations of motion is normally less than the time required for a force to be transmitted to more distant neighbors. Violation of this last recommendation can lead to instability in the numerical simulation.

X7.5.2.6 Impact Phenomena. The simplest form of an impact model is a compression-only linear spring. It is conservative, i.e., its coefficient of restitution is one, and bodies colliding on such a spring will receive the correct total impulse to rebound with their relative velocity conserved in magnitude and reversed in sign. The duration of contact is inversely proportional to the square root of the spring stiffness and the magnitude of the peak force experienced is proportional to the square root of the spring stiffness. Adding a linear damper to the model will absorb energy reducing the coefficient of restitution by approximately (10):

\[ \mu = e^{-2\zeta} \] (X7.3)

where \( \mu \) is the coefficient of restitution;
\( \zeta \) is the critical damping fraction.

The difficulty with using this model for impacts at assembly load pads lies in choosing the spring constant. The stiffness of a hexagonal load pad varies from

\[ K_2 = \frac{12}{D/t} \frac{3}{2} \left( \frac{D}{t} \right)^2 + 9 \] (X7.4)

in two face loading to

\[ K_6 = \frac{2}{3} \frac{3}{D/t} \frac{E}{h} \] (X7.5)

in six face loading, where

- \( K \) is the diametral stiffness, the applied load divided by the change in across flats diameter;
- \( E \) is Young's modulus;
- \( h \) is the height of the load pad;
- \( D \) is the across flats diameter of the load pad;
- \( t \) is the thickness of the load pad.
The ratio $\frac{K_6}{K_2} = \left(\frac{D}{3t}\right)^2 + 1/2$, is a large number so that the load pad stiffness can vary drastically for different loading states.

An alternative to using such structural stiffness formula for choosing an impact stiffness is to base the stiffness on the duration of impact. If the duration of the impulsive load, $T$, is known from experiments and an effective mass, $M$, for the impact can be estimated, the spring stiffness can be chosen by

$$K = \frac{M \pi^2}{T^2} \quad (X7.6)$$

This relation will preserve the duration of impacts but it is important that the experimental basis for the duration be based on impulses of similar magnitude to those expected in the core response. It is important to verify that the spring stiffness used in the impact model does not give unreasonably large deformations under impulsive loads of the magnitude expected in the core simulations.

Another factor to consider in modeling impacts in liquid is the effect of the fluid squeezing out of the contact area during impact. A model of this "squeeze film" effect was developed for use in the SCRAP code (40). While it was found that this fluid effect substantially reduced the peak impact force and extended the duration of the impact when only a few assemblies were involved it did not have much effect on the peak impact loads for models of full reactor cores. The reason for this is that the momentum involved in the full reactor core is sufficiently large that the fluid forces could not generate sufficient impulse to stop the core. The fluid impact model was sufficiently complex that it did significantly increase the computer time required for the simulation.

It is recommended that linear spring-dashpot impact models be used with impact damping corresponding to a coefficient of restitution of 0.5. In the absence of experimental data the spring constant corresponding to 2 face loading should be used. Fluid effects should only be considered for final confirmatory analysis. If assembly bowing information is available, a bilinear impact spring should be used to account for the bowing energy stored in closing a gap.

X7.5.3 Numerical Simulation.

X7.5.3.1 Support Motion. The input to a nonlinear time-history analysis is a time history of the core support acceleration. Normally this is available as a time histories generated in the building seismic analysis or a response spectrum. In the latter case several standard methods are available for generating time history compatible with the response spectrum. In most past analyses the motion is assumed to be rectilinear in one of the three principle directions of the hexagonal array. Little experience is available considering planar input or rectilinear input along another axis.
X7.5.3.2 Duration of the Simulation. For constrained core designs a simulation of about 1-2 minutes involving 5-10 cycles of support motion is sufficient to define the core response provided the strongest portion of the support acceleration is used. One to two minutes simulation is also sufficient in most cases for unconstrained or partially constrained core designs but may not be sufficient if a resonance is developing. Displacement response time histories should be examined for these designs to verify that the motion is not continuing to grow in a resonance fashion.

X7.5.3.3 Integration Time Step. The time step used in the numerical integration will depend on the algorithm selected but certain problem-dependent features should be considered. Normally the time step required for stability and accuracy of the integration is governed by the maximum stiffness in the impact models. Thus, time step and ultimately computer time are coupled with the impact models and should be considered together.

Accuracy of the gross core dynamics (vibrations) does not require detailed accuracy of the impact force time history; only the total impulse is important to the vibrations and this is inherently self-correcting. The time step need not be chosen so small as to give accurate impact force histories if only the gross dynamics is of interest.

X7.5.4 Response Measures.

X7.5.4.1 Displacement Time Histories. The most stable response history is the displacement at various location in the core model. In most simulations it will not be necessary to record displacements at each time step of the integration because of this stability. Care should be exercised to verify that the simulated displacements are not excessive at the impact models.

X7.5.4.2 Stress Time Histories. Bending stresses are particularly sensitive to the number of degrees-of-freedom or modes used in the individual assembly model. Normally a full core model will not allow sufficient detail to give an accurate distribution of bending stresses on individual assemblies. The analysis plan should provide for a supplementary detailed stress analysis of selected individual assemblies using the displacement time histories from the full core simulation as input.

X7.5.4.3 Acceleration Time Histories. Acceleration time histories will have considerably more high frequency content than the displacement or bending stress histories so they should be recorded at a shorter time interval in the simulation. The accuracy of the acceleration histories at locations near impacts have the same sensitivity to time step as the impact forces. They should be interpreted accordingly.
X7.5.4.4 Impact Force Time Histories. The impact forces are very sensitive to the impact modeling assumptions, particularly stiffness, and to the integration time step. They should be recorded at each integration step and interpreted with care.

X7.5.4.5 Integrated Measures. The total impulse during impact is a more stable measure than the impact force history and is a valuable parameter in estimating the potential for damage. Provision should be made for a time integration of the force time history to provide impulse information.

A second integrated measure that is useful in interpreting core seismic simulations is a reactivity time history. This is obtained by calculating a weighted sum of the relative displacements of assemblies in the core region. The weighing factor is a measure of the reactivity worth of the individual assembly displacement.

X7.6. GUIDELINES

X7.6.1 Analysis. It is recommended that the following elements be incorporated into any fast reactor core seismic analysis plan.

A simplified impulse-momentum model of the core design be developed and used for preliminary evaluations of design options, verifying detailed time history analyses and interpreting the results of such analyses.

A nonlinear time history model of a diagonal row of the core be developed including fluid inertia coupling and linear or bilinear spring-dashpot impact models. This model should allow individual deformations of each assembly and should contain 2 or 3 degrees-of-freedom per assembly. This model should be the primary tool used for detailed study of the core seismic response.

A simplified version of the previous model with 25-30 degrees-of-freedom should be constructed by constraining groups of several assemblies to deform together. This model should require considerably less computer running time and is suitable for evaluation of various design options and investigations of the sensitivity of the design to modeling assumptions.

A full core model which allows displacements of the assemblies in two horizontal directions should be constructed with the same fluid inertia and impact models used in the row model. This model will also require "clustering" of groups of assemblies to achieve a reasonable computer simulation time. It should be used as a check of the row model and to evaluate support motions in different directions.
A version of the row model with fluid impact modeling should be used if the peak impact forces are constraining the design.

X7.6.2 Testing. The recommendations for testing are based on considerably less experience than the analysis recommendations. Unless full scale tests of at least a full diagonal row are contemplated, the testing program should be used in support of analysis. The following tests are recommended.

Single assembly tests should be conducted at full scale for each of the core assembly types. These tests should give careful consideration to the nozzle support detail and should be directed toward identifying natural frequencies, mode shapes and structural damping in fluid.

Impact tests between two assemblies should be conducted to determine coefficients of restitution and duration of impacts. Special care should be given to ensure that the appropriate momentum and velocity conditions are provided. This may require attaching heavy masses to the test assemblies. These tests may also be useful in determining the damage threshold of the assemblies under impact.

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