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DRAFT

TASK 10/TAP A-40

PHASE I

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NOTE: This draft has not been reviewed by the core members or consultants.

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ABSTRACT

This report contains recommendations for changes in the NRC criteria currently used in the seismic design of nuclear power plants. Areas covered include:

Ground Motion; Soil-Structure Interaction; Structures and Equipment and Components. The recommendations represent a consensus from members of the Engineering Mechanics Section of the Nuclear Test Engineering Division at Lawrence Livermore Laboratory (LLL) and are based on the results of reports developed under the NRC's Task Action Plan (TAP) A-40 program and the recommendations of nationally recognized experts retained by LLL specifically for this task.

SUMMARY

The recommendations contained in this report are intended to bring NRC seismic design criteria up to the current state-of-the-art. These recommendations include:

- Changes in the specification and application of ground motion input for the design of structures and equipment.
- Significant changes to the philosophy and specifications for soil-structure interaction analysis.
- More specific guidelines for the seismic design and analysis of special structures (i.e., buried pipes, conduits and above ground vertical tanks).
- Specific criteria for the modal response combination of high frequency modes.
- The allowance of limited amounts of inelastic energy absorption capability for typical Category I structures.
- Revision of damping values for design based on the type and condition of the structure and the stress levels of interest.
- Direct generation of in-structure response spectra for equipment design.
- Accounting for uncertainties in the generation of in-structure response spectra through multiple analyses with variation of parameters and through

the use of nonexceedance probabilistic generation of in-structure response spectra.

- The option to use randomly selected multiple time histories (real or synthetic), when the time-history approach is used.

- Reduction in the number of OBE earthquake cycles required for design.

There is much additional research required to quantify the conservatisms in the seismic design sequence. The recommendations in this report reflect recent increased understanding in the art of seismic design as well as the relative degree of uncertainty that exists in the elements of the seismic design sequence. Furthermore, NRC criteria for the seismic design of nuclear power plants should be written in such a way as to indicate the nature of the performance that is required (to ensure that adequate margins of safety exist), but at the same time should not be so restrictive as to preclude the introduction of improved approaches. Thus, while some of the recommendations in this report are specific, this is done mostly to clarify. Other methods are equally acceptable as long as they provide a similar degree of conservatism.

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INTRODUCTION

This report summarizes the Phase I efforts on Task 10 of Task Action Plan A-40 (TAP A-40). The objectives of Task 10 are:

- Review Tasks 1 to 9 of TAP A-40.
- Recommend changes in the Standard Review Plan (SRP) and Regulatory Guides.

The NRC Division of Project Management initiated TAP A-40 to identify and quantify the conservatisms inherent to the seismic design sequence in current NRC criteria. The program is currently under the management of the Division of Reactor Safety Research, and is intended to provide a short-term improvement of these criteria until long-term solutions are obtained from the Seismic Safety Margins Research Program (SSMRP). Unfortunately, the short-term improvements recommended in this report are more complicated than we anticipate deriving from the SSMRP. This is because of our limited understanding and capability at this time in this complex area. We believe the knowledge gained from the SSMRP will allow the development of significant simplifications.

Task 10 is intended to bring the SRP and Regulatory Guides up to the current state-of-the-art of seismic design. The results of the TAP A-40 program and the recommendations of Task 10 will also be useful to the NRC staff in their review of existing plants under the Systematic Evaluation Program (SEP).

1737 077

The TAP A-40 program consisted of the following ten tasks:

Task 1 - Quantification of Seismic Conservatisms

The objective of this task was to identify and quantify the conservatisms in the following areas of the seismic design sequence:

- R.G. 1.60 Spectra
- R.G. 1.60 Time Histories
- Damping
- Soil-Structure Interaction
- Response to 3 Components of Motion
- Broadening of Spectral Peaks
- Structural & Mechanical Resistance
- Nonlinear Structural Response
- Subsystem Response
- OBE vs. SSE Response
- Overall Conservatism

Task 2 - Elastic-Plastic Seismic Analysis

This study was undertaken to evaluate the reserve capacity in a power plant braced steel frame from nonlinear effects and to determine the effect of supported equipment and piping on the overall response.

Task 3 - Site-Specific Response Spectra

The objective of this task was to develop a more realistic method for developing spectral shapes that are realistic and not overly conservative and that account for specific site characteristics.

Task 4 - Seismic Aftershocks

The objective of this task was to assess more thoroughly the possibility that aftershocks, although less severe than the main earthquake, may result in additional damage to the structures, systems or components that are allowed to respond inelastically during the SSE.

Preliminary investigation indicated that the available data base is very limited, and it was decided that the inelastic response due to an SSE for re-evaluation of existing designs will be limited to a small fraction of available ductility. As a result, this task was subsequently cancelled.

Task 5 - Nonlinear Structural Dynamic Analysis Procedures for Category I Structures

This task investigated the feasibility of using simplified nonlinear dynamic analysis techniques for the design of typical Category I structures by comparing the results of various simplified techniques with the results obtained from rigorous nonlinear dynamic analyses.

Task 6 - Soil-Structure Interaction

The objective of this task was to determine the limits and conditions of applicability as well as estimates of conservatism in the soil-structure interaction procedures and corresponding definition of seismic input currently used in the seismic analysis of nuclear power plants.

Task 7 - Earthquake Source Modeling

The objective of this task is to develop criteria for determining the adequacy of modeling techniques proposed by applicants to assess ground motion near faults.

Task 8 - Analysis of Strong Motion Near-Field Data

The objective of this task is to develop a methodology for determining ground motion spectra in the strong motion near-field region.
near-field of earthquake sources.

Task 9 - Development of Seismic Energy Attenuation Functionals

Functional relationships between seismic energy and source distance will be developed using wave propagation theory. The appropriate functionals will then be used to fit the available seismic records, to obtain the necessary coefficients so that seismic attenuation can be predicted.

Task 10 - Review and Implementation

The objective of this task is to provide a technical review of the results of the other tasks in the TAP A-40 program and to recommend changes to the existing NRC criteria based on this review. As noted earlier, TAP A-40 program consists of engineering tasks and seismological tasks. Since the engineering tasks are substantially completed, they are included in Phase I and the seismological tasks 7, 8, and 9 are in Phase II.

Phase I of Task 10, includes only Tasks 1, 2, 3, 5, and 6 for the review and implementation effort summarized in this report.

APPROACH

We used the team approach in order to accomplish the objectives of Task 10 in an efficient manner and to provide the best technical product possible within the limited time available. The team consisted of a core group of Lawrence Livermore Laboratory personnel, NRC staff members, and selected consultants. The LLL core group, drawn from the Engineering Mechanics Section of the Nuclear Test Engineering Division, included the following:

- D. L. Bernreuter
- S. E. Bumpus
- D. W. Coats.
- J. J. Johnson
- O. R. Maslenikov
- R. C. Murray
- T. A. Nelson
- P. D. Smith
- F. J. Tokarz

The NRC program manager for the TAP A-40 Program is Steve Hanauer (Unresolved Safety Issues) and the project manager is Goutam Bagchi (Structural Engineering Research Branch).

Consultants were selected based on recommendations of core members and NRC staff members and consisted of the following:

- o Dr. R. L. Cloud (R. Cloud Consultants).
- o Dr. W. J. Hall (U. of I.)
- o Dr. R. P. Kennedy (EDAC)
- o Dr. N. M. Newmark (U. of I.)
- o Dr. J. Roesset (U. of Texas)
- o Dr. J. C. Stepp (FUGRO)

The TAP A-40 tasks were placed into four categories. These categories and the consultant identified with each are:

- o Ground Motion - Stepp
- o Soil-Structure Interaction - Roesset
- o Structures - Kennedy
- o Equipment and Components - Cloud.

Drs. Newmark and Hall participated in the review of all four areas.

Copies of the pertinent sections of the Standard Review Plan and Regulatory Guides as well as the reports(1-18) developed under the TAP A-40 program were provided to the participants. These reports, as well as the experience of the consultants and core group provided the technical basis for the recommendations in this report.

The initial meeting for the Task 10 project was held at LLL on April 10, 1979 with LLL core members, consultants, and Goutam Bagchi and Sai Chan of the NRC. The purpose of this meeting was to:

- Relate objectives of the task to consultants.
- Describe approach used to accomplish objectives.
- Define scope of work.
- Provide participants with pertinent reports.

Early interaction with NRC staff members was considered essential, and a meeting was held in Bethesda, MD on June 19th and 20th, 1979, where the consultants made presentations to the LLL core members and NRC staff members on their recommended changes to the SRP and Regulatory Guides. The interaction between consultants, core members, and NRC staff at these meetings provided additional insight into staff concerns regarding the implementation of recommended changes to current NRC seismic design criteria. The discussions and comments made at this meeting were incorporated into the consultants final reports to Livermore.[19,20,21,22,52] These final reports have been reviewed by LLL core members and the recommendations presented in this report are drawn from the consultant's reports as well as the consensus of the LLL core members.

GENERAL PHILOSOPHY OF RECOMMENDATIONS

It was decided that it would be beneficial if a general philosophy and objective for the SRP could be established in order to allow the SRP to be more flexible and provide a degree of uniformity and consistency with respect to the recommendations made in this report. The following philosophy and objectives were generally agreed upon by LLL core members and consultants:

- SRP recommendations should be made with the purpose of indicating the nature of the performance that is required to ensure that adequate margins of safety exist, but at the same time are not so restrictive as to preclude the use of new and more rational approaches when these can be documented and checked readily against other approaches.
- Based on the assumption that typical present SSE peak accelerations result in an annual probability of exceedance of the order of 10^{-3}, the required conditional annual probability of exceeding design seismic response is about 10^{-1} conditional on the occurrence of an earthquake with a peak acceleration equal to the SSE. This is consistent with an

overall probability of failure, given the SSE occurs, in the order of 10^{-5} to 10^{-6} occurrences per year.

The remainder of this report consists of our recommendations for changes and/or additions to the Standard Review Plan and Regulatory Guides in the areas of: Ground Motion; Soil-Structure Interaction; Structures and Equipment and Components. Final reports of consultant's recommendations are included as Appendices.

RECOMMENDATIONS

I. GROUND MOTION

A. General

A review of the data base currently available in the area of ground motion for the design of nuclear power plants has been made as part of the TAP A-40 program. Note that not all of the tasks related to the ground motion input studies have been completed at this writing. However, it is clear that a case can be made for the use of site specific spectra in lieu of the current R.G. 1.60 spectra. We believe the results of tasks 7 and 8 and work on the SSMRP will confirm the feasibility and desirability of using site specific spectra as well as the determination of these spectra by techniques such as those proposed by Newmark and Hall (Ref. 35) in which peak ground accelerations, velocities and displacements are required to construct the response spectrum instead of just peak ground accelerations. We therefore recommend replacement of the existing R.G. 1.60 response spectra with the more site specific response spectra recommended by Profs. Newmark and Hall in NUREG/CR-0098.

B. NUREG/CR-0098 Response Spectra

Because of deficiencies that exist in the current Ground Design Response Spectra as specified in R.G. 1.60, we recommend the following:

- The current definition of ground design response spectra as contained in R.G. 1.60, should be replaced with the more site specific definition for ground motion response spectra as recommended by Newmark and Hall in NUREG/CR-0098. Amplification factors corresponding to the median plus one standard deviation (MSD) should be used.

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Also, recent studies (Refs. 2, 53 and 54) have indicated that the current definition of the vertical ground design response spectrum as given in R.G. 1.60 is quite conservative, and as a result of these studies we recommend that:

- The vertical ground design response spectra values should be taken as 2/3 of the values specified for the horizontal ground design response spectra, across the entire frequency range, using the MSD amplification factors as specified in NUREG/CR 0098.

Some of the reasons for recommending the NUREG spectra to replace the R.G. 1.60 spectra are enumerated below:

- The R.G. 1.60 spectra are deficient in the low frequency range because the data base used was not properly corrected.

- The NUREG spectrum approach allows for some site specificity.

- It is easier to get consistent synthetic earthquakes for various damping levels using the NUREG spectra than using R.G. 1.60.

- NUREG spectra give uniform risk with respect to different damping values, R.G. 1.60 spectra do not.

- The NUREG spectra utilized the same data base as the R.G. 1.60 spectra.

In order construct the NUREG spectra, it is necessary to determine the peak ground velocity and displacement as well as the peak ground acceleration. If site specific data regarding the peak ground velocity and displacement cannot be obtained, then the following procedure is recommended:

- When lacking site specific values for peak ground velocity and displacemen, a v/a ratio of 48 in/sec/g and a d/a ratio of 36 in/g should be used for competent soil conditions, and a v/a ratio of 36 in/sec/g and a d/a ratio of 20 in/g should be used for rock, where a,v and d are the maximum values of ground motion (accelertaion (in/sec^2), velocity (in/sec), and displacement (in), respectively.

Also, to ensure that the spectrum represents an adequate band (frequency) width to accommodate a possible range of earthquakes, it is recommended that ad/v^2 be taken equal to about 6.0.

We believe further studies on the statistics of response spectra generated from real earthquakes (such as the mean plus one standard deviation used in developing R.G. 1.60 and the NUREG spectra) are required, and that these studies should consider that spectral values from any one earthquake are correlated. That is, statistics at a given frequency are not independent of those at other frequencies. These studies should also consider that the motion specification provided by response spectra is not entirely satisfactory, for example, the probability of exceedance can vary with damping.

C. Standard Review Plan Section 2.5.2

The SRP Section 2.5.2 could benefit from a Statement of Objectives. The objective should be to provide criteria for reviewing site free-field vibratory ground motion proposed for seismic design input to nuclear power plant soil structure systems that are realistic and consistent with state-of-art-practice with conservatism to account for uncertainty in our knowledge and data. By state-of-the-art-practice, we refer to the application of technology that is common to the practice of the majority of scientists and engineers. This is important in the regulatory climate where conclusions must be strongly documented and often are subjected to lengthy and detailed review. Use of state-of-knowledge procedures and developing technology will likely always enter into some decisions, but should not be embodied in the SRP review criteria beyond the recognition that they may be required in some cases.

Also, it would be a useful perspective to identify "primary review" areas required to meet the requirements of Appendix A to 10CFR Part 100, and "subordinate review" items needed to complete the seismic design input evaluation. The primary review areas for evaluating the SSE are:

1. Tectonic provinces;
2. Correlations of earthquakes with tectonic structure;
3. Capable faulting;
4. Maximum earthquakes associated with tectonic provinces and capable faults.

The subordinate review areas are:

1. Regional geology;

2. Seismicity;
3. Site geology;
4. Site seismic wave amplification properties;
5. Fault characteristics, dimensions, and movement rates;
6. Ground motion attenuation; and
7. Site soil properties.

In addition, a primary and separate review area is the proper OBE consistent with the definition contained in Appendix A.

In the SRP Subsection 2.5.2 (II) the SSE is reference to "the maximum potential earthquake" though it is recognized that multiple maximum earthquakes are to be considered. This is somewhat confusing and has caused some to reference the maximum earthquake as the SSE rather than the ground motion for seismic design. The SSE should be defined as the <u>free-field vibratory ground motion at the site to be used for seismic design input to the soil-structure system</u>. Similarly, the OBE should be defined as the proper free-field vibratory ground motion at the site to be used as input to the soil-structure system for OBE design considerations.

One of the primary subjects of review in SRP 2.5.2 is the completeness of the historic and instrumental earthquake data presentation. To avoid unnecessary review and cost to applicants, the following reporting requirements are recommended:

- Eastern United States
  - Within 200 miles of the site: all known earthquakes with maximum MM intensities greater than or equal to IV or magnitudes greater than or equal to 3.0 should be included.
  - Within a distance of 50 miles of the site: all known earthquakes should be reported.
- Western United States

  Because of the rapid rate of tectonism in the Western United States resulting in frequent earthquake occurrence, it is not necessary to require all earthquakes to be reported.
  - Within 200 miles of the site, all known earthquakes which have maximum MM intensities greater than or equal to MM IV or magnitudes greater than or equal to 4.0 should be included.
  - Within 50 miles of the site, all known earthquakes should be included in the presentation.

    All magnitude designations should be identified (mb, mL, ms, etc.). When comparing events or when using the data in numerical evaluation, proper relationships among various magnitudes should be drawn and a common magnitude base established.
- Some source information such as rise time, rupture, velocity, total dislocation and fractional stress drop must be interpreted from indirect data. Generally these parameters are highly uncertain and are not presently incorporated into state-of-the-art-practice for determining seismic design input. It is recommended that this information not be required routinely as part of the presentation. For special cases where this information is to be used, it should be obtained through a special request.

Probability estimates of the SSE are requested in SRP Section 2.5.2 (II.5). This is in conflict with the requirements of Appendix A to 10 CFR Part 100. Moreover, no policy establishing acceptance criteria for the SSE in terms of probabilities has been put forward by the Commission. Currently, ongoing work at LLL in support of the NRC Systematic Evaluation Program (SEP) and as part of the Seismic Safety Margins Research Program (SSMRP) is providing important results which promise to offer a basis for establishing policy with respect to acceptable earthquake hazard in terms of probability of exceedence. This is particularly true for the SEP program because acceptance criteria will be required for that program. Until such policy is established, however, probabilistic estimates of the SSE should be permitted but not required in the SRP. Current methods for defining the SSE result in a level of hazard for the SSE in the range of 10^{-3}.

With regard to the Safe Shutdown Earthquake (SSE) the following recommendations are made:

- o The objective of the SSE review should be to evaluate whether or not the maximum vibratory ground motion for the site, defined at the free-field surface, is properly conservative in consideration of the sites' earthquake potential.
- o The SRP should provide that vibratory ground motion at the free-field surface may be described either by NUREG/CR 0098 response spectrum scaled to the appropriately conservative peak ground acceleration velocity and displacement or by an appropriately conservative broad band site specific spectrum.

- For sites where the controling earthquake(s) are associated with defined tectonic structure and the ground motion spectrum is defined by NUREG/CR 0098:
  - The mean plus one standard deviation (MSD) acceleration of the zero period accelerations for each of the structures obtained from appropriate attenuation relationships should generally be accepted as an acceptably conservative value for the peak ground acceleration.
  - Consideration should be given to site seismic wave amplification properties in determining the adequacy of the MSD value.

- Site specific spectra should be based on properly similar source properties, magnitude of controlling earthquake(s), source distance, and site properties.
  - Spectra should be derived from an adequate sample of site specific accelerograms appropriate to the site.
  - The MSD smoothed spectrum derived from an adequate sample of site specific accelerograms should generally be considered as being acceptably conservative for the free field surface motion at a site.
  - Site amplification properties should be evaluated and the final ground motion to be used at the free-field ground surface should conservatively account for site amplification.

- For sites where the controlling earthquake is the maximum historic intensity in the sites' tectonic province and the ground motion spectrum is defined by NUREG/CR 0098:
  - The MSD of acceleration taken from appropriate acceleration - MM intensity relationships should generally be acceptable as a properly

conservative value for the zero-period acceleration.

- o Consideration should be given to seismic wave amplification properties of the site in evaluating the adequacy of the MSD value.
- o For sites close to an earthquake source, the peak instrumental response should be modified to be consistent with the response of the larger heavy structures near the site. (i.e., effective peak ground accelerations should be used).

- o If both close-in and distant sources effect the site, then two separate spectra should be used for design. The larger of the responses from the application of these two spectra should be used for design.

Section 2.5.2 of the Standard Review Plan states: "The results should be used to establish the site free-field vibratory ground motion irrespective of how the plant structures will ultimately be situated or where they are founded."

It is recommended that additional clarification of this statement be included as follows:

- o If proper account is taken of the seismic wave amplification properties of a site in specifying the free-field motion, no specific consideration needs be given to the placement of structures.

Amplification of energy can be expected at all soil sites at the natural period of the soil column. For many sites in the Eastern United States relatively low density alluvial or glacial sediments overlie high density bedrock at shallow-depth. The seismic acoustical properties of the two media differ significantly, resulting in large amplification of ground motion in the frequency interval of concern to nuclear power plant design. For deeper soil sites, reduction of the surface motion by deconvolution may be appropriate after due consideration has been given to the amplification properties of the site. However, for sites characterized by shallow soils overlying bedrock and where structures are founded in bedrock it should be proper to take the simple approach and permit no reduction of the free-field surface. This approach will avoid unnecessary analysis and review.

D. Time History Analysis

Artificial or synthetic time histories continue to be an area of concern. For some time there have been questions about the frequency, amplitude, and energy content of these histories in spite of the fact that they lead to an enveloping of the design response spectra. Such synthetic records must be used with great care in the analysis of nonlinear systems (including soil-structure interaction) since the nonlinear behavior is strongly influenced by the cyclic history. Therefore, the following recommendations are made:

- o Both real and synthetic time histories are acceptable for the design and analysis of nulcear power plant systems, subsystems and components.

- o When synthetic time histories, are used, the following is acceptable:

  a) If only one synthetic time history is to be used, then it must envelop the MSD design response spectrum, and peak broadening of in-structure response spectra resulting from this should be done as currently required in R.G. 1.122.

b) If multiple (≥ 5) synthetic time histories are to be used, they shall each be mean centered about the MSD design response spectrum and the median must be at or above the MSD design spectrum frequency by frequency.

c) Synthetic time histories should not be used for non-linear analysis.

o When real time histories are used, the following is acceptable:

a) Multiple (≥ 7) real time histories properly scaled for frequency content, amplitude, energy content, etc. shall be used.

b) Real time histories should be selected based on similar site and geological conditions.

c) The MSD spectrum of the real time histories should be at or above the MSD design spectrum frequency by frequency.

d) Only real time histories should be used for nonlinear analysis.

II. SOIL-STRUCTURE INTERACTION

A. General

Considerable advances in computational techniques for soil-structure interaction have been made over the last few years. Unfortunately, only a small amount of field data is available, and experimental verification of analytical techniques has not been accomplished. The recommendations herein are therefore based on TAP A-40 reports [5,16,17,18] and the expertise and engineering judgment of the consultants and core members retained for this project. Several general recommendations are:

- o References in the SRP to "Finite Element" and "Lumped Parameter" Techniques of soil structure interaction analysis should be removed. Two categories of analytical techniques called the "Direct Solution" (analysis performed in one step) and "Substructure" (analysis performed in three steps) approaches should be identified instead. This terminology is more descriptive of the two broad categories of analytical methods.
- o Either the Direct Solution or Substructure approach may be used for soil-structure interaction analysis as long as they are properly applied and within the limitations discussed below. Performing independent analyses with each technique and enveloping the results should not be required.
- o All soil-structure interaction analyses must recognize the uncertainties prevalent throughout the phenomenon, i.e., transmission of the input motion at the site, the random nature of the soil configuration and material characteristics, the uncertainty in soil constitutive modeling, nonlinear soil properties, and the coupling between the structure(s) and soil.

- The term "equivalent linear" should not be used.
- Relatively simple methodologies need to be established by which soil-structure interaction analysis results may be checked for feasibility.
- It is not clear that complex, expensive calculations are justified in view of the large uncertainties that exist or that they are necessary to develop a soundly engineered design.

B. Nonlinear Soil Behavior

The nonlinear behavior of soil can be separated into primary and secondary components. Primary nonlinearity is due to the excitation itself and the nonlinear characteristics of the soil properties and affects the propagation of the wave. Secondary nonlinear behavior is induced by the structure and is due to the soil-structure interaction phenomenon (i.e., energy feedback, etc.).

At the present time, our current state of knowledge does not permit a rigorous nonlinear analysis for the soil-structure interacton phenomena. At best we can only estimate the soil properties necessary to account for nonlinear effects. The following recommendations are made with regard to the nonlinear nature of the soil-structure interaction phenomenon.

- The nonlinear behavior of soil, although clearly recognized to exist, should be approximated by linear techniques. Nonlinear analysis should not be required for design until the comparison of results from large scale tests or actual earthquakes and analytical results indicate deficiencies that cannot be accounted for in any other manner. Efforts and resources should be directed toward sensitivity studies and bounding solutions rather than a detailed nonlinear analysis.

The nonlinear soil behavior for design may be accounted for by the following:

- o Perform a consistent linear analysis which accounts for primary nonlinearities by determining the values of the linear constitutive parameters to account for the excitation level in the free field.
- o Perform an iterative linear analysis on the coupled soil-structure model. (Direct solution approach)

Either of these techniques is equally acceptable for structural response computations (even though only the direct solution approach purports to address secondary nonlinearity) since the effect of secondary nonlinearity appears to be of second order. Additionally, in view of the large uncertainties that exist, it is recommended that:

- o The linear, strain dependent, soil properties estimated from analyses of the seismic motion in the free field shall be limited. Values of shear modulus should not be less than 40% of their low strain values (at strains of 10^{-3} to 10^{-4}%). Values of internal soil damping of a hysteretic nature, should be limited to a maximum of 15% of critical.

- o Superposition of horizontal and vertical response as determined from separate analyses is acceptable considering the currently available simple material models.

- o A suite of real time histories (≥ 7) is recommended. A separate, randomly selected time history should be used for each of the ≥ 7 variations in soil property.

- The ≥ 7 sets of soil moduli to be used for design shall cover the range of:

  best estimate

  2.0 x (best estimate)

  (best estimate)/2.0

  When real time histories are used, for the soil-structure interaction analysis, the MSD of the responses resulting from the use of this range of soil properties and suite of time histories shall be used. When synthetic time histories with broad band target spectra are used, the mean of the responses shall be used.

- For slanted soil layering up to and including 25°, horizontal layering may be assumed for structural response purposes.

- For slanted soil layering greater than 25°, it is necessary to account for the coupling between the horizontal and vertical degrees-of-freedom in the stiffness and free-field seismic motion definitions.

C. Direct Solution Technique

The Direct Solution method is characterized by the following:

-- The analysis of the soil and structure/structures is performed in one step.

-- The finite element or finite difference discrete methods of analysis are used to spatially discretize the soil-structure system.

-- The definition of the motion along the boundaries of the model (bottom and sides) is either known, assumed, or computed as a pre-condition of the analysis.

The spatial representation of mathematical models employing the Direct Solution technique are typically two-dimensional, plane strain models or axisymmetric models. Dynamic analysis can be done in either the frequency domain (limited to linear analysis) or the time domain. The selection of the mesh size should be such as to adequately represent the static stress distribution under the foundation and to transmit the frequency content of interest. Two mathematical representations of the model side boundaries are available for use in the Direct Solution approach. These are: 1) simple or viscous boundaries; and 2) transmitting boundaries The location of the simple or viscous boundaries is dependent on strain and damping in the soil and is typically 3 base dimensions from the structure. The side boundary nodes can either be "fixed", in which case free-field displacements are specified, or "free", in which case forces are specified. When using the transmitting boundaries, it is theoretically possible to place the boundaries immediately adjacent to the structure, if secondary nonlinearities in the soil can be ignored. The solution of the soil-structure interaction problem, by use of the transmitting boundaries, is a rigorous solution done in the frequency domain, that corrects for disturbances from the structure.

The Direct Solution method is applicable for sites with a soil layer over competent rock for the rock surface within 150 feet of the surface. For a rock location greater than 150 feet, the following limitations must be observed:

o The model depth must be at least 2 base dimensions.

- The fundamental frequency of the soil stratum must be well below the structural frequencies of interest.

- Must consider an alternate to deconvolution for defining the model free-field motion. e.g., Trial and error.

Table 1 lists the advantages and disadvantages associated with the Direct Solution Technique.

Table 1

Direct Solution Technique

Advantages	Disadvantages
• Truly nonlinear analysis possible	• Economics generally limit models to two dimensions
• Can account for secondary nonlinear soil behavior	• Specification of seismic design environment for model boundary may be difficult
• Not limited to the assumption of vertically propagating shear waves	• Ability to analyze deep soil sites is questionable
	• Many currently used computer codes are limited by the assumption of vertically propagating shear waves

D. Substructure Solution Technique

The Substructure (3-Step) approach consists of the following steps:

1) Determine compatible motion at the foundation level. This includes both translational and rotational components.

2) Determine the foundation stiffness in terms of frequency dependent impedance functions.

3) Perform soil-structure interaction analysis.

Step 1) requires assumptions on the mechanism of wave motion at the site. The foundation motion may be determined by a number of techniques including analytic functions, boundary integral equations, finite element methods, and finite difference methods. In calculating the foundation motion by one of these methods, the normal assumption made is that the foundation itself is assumed rigid and bonded to the soil. However, this is not a limiting restriction as additional degrees of freedom may be specified for the foundation. Again, it must be emphasized that in general a translation specified on the surface of the soil produces a translation and rotation of the massless foundation.

The determination of the stiffness characteristics of the soil, as required in Step 2), may also be done by analytical functions, boundary integral equations, finite element methods, and finite difference methods. When calculating the soil stiffness, it is essential that the soil characteristics must account for variations with excitation level.

Typically, the SSI analysis of step 3) is done in the frequency domain. It is most important that the frequency dependence of soil impedances be properly accounted for.

Table 2 lists the advantages and disadvantages associated with the Substructure Technique for SSI analysis.

Table 2

Substructure Technique

Advantages	Disadvantages
• In each step, the most appropriate numerical technique may be used.	• Limited to linear analysis
• Sensitivity studies may be performed on each step easily and inexpensively.	• Only accounts for primary non-linear soil behavior in current applications. (extensions may be possible)
• Intermediate results may be obtained and evaluated	
• The effect of various angles of incidence may be studied.	
• 3-D analysis may not add significantly to cost	

E. Seismic Design Environment and Wave Passage Effects

In the specification of the seismic design environment, it is recommended that:

o The seismic design response spectrum to be used in the SSI analysis for both the Direct Solution or the Substructure techniques, should be specified at the highest level of competent material (i.e., low-strain shear wave velocity of 600-800 fps).

o Models for deconvolution must be consistent with the free field and soil-structure interaction computations. For example, SHAKE should not be used with LUSH or FLUSH.

While it is generally agreed that a reduction in acceleration is jusifiable based on embedment effects, the amount of reduction to be allowed and the location at which this reduction is specified is subject to considerable controversy. No consensus by Livermore core members or consultants was reached on this matter. The magnitude of allowable reduction ranged from 25% to 40% of the design ground response spectrum, frequency by frequency. The location for specifying this reduction ranged from, in the free field at the foundation level, to on the foundation mat for the direct approach and at the base of the massless, rigid foundation in the substructure approach. It is of interest to note that the Japanese have limited this embedment reduction effect to a maximum reduction of 25% of the ground design response spectrum. The location of this reduction is on the foundation basemat.

We believe additional consideration of this issue (with NRC staff members) is needed before a recommendation can be made.

However, it was generally agreed upon that if any reduction for embedment effects is to be allowed, the resulting rotational component of motion at the foundation level must be included in the analysis.

With regard to wave passage effects, the following recommendations are made:

o Alteration of the translational input due to wave passage effects, must be accompanied by resulting torsion and rocking response.

- The apparent wave velocity used should be consistent between the SSI analysis and the analysis and design of buried components.

Waves incidenting the surface at an angle produce rocking and torsional effects at the surface as well as a reduction of the translational motion. Because of the complexities involved in incorporating the torsional effects in the structural response, it is recommended that:

- Torsional effects induced in the structure due to wave passage effects should be accounted for by specifying a minimum eccentricity for the structure.

F. Special Problems

Many areas in the soil-structure interaction phenomenon are not well understood and much additional study is required to increase our understanding. The following brief discussions touch upon some of these problems.

- Further investigation of the effect of structure to structure interaction, especially in three-dimensions are needed before design conditions should be specified. Parameter studies are required for typical simplified models. The results will be sensitive to the nonlinear soil behavior between the structures so it is not clear that linear methods can be used to develop appropriate design requirements.

- The assumption of representing a three dimensional configuration with two dimensional plane strain models requires further evaluation, particularly for deep soil profiles.

- Flexible side boundaries (of the foundation or containment walls) can be incorporated in the substructure approach by computing motions and stiffness coefficients at a sufficient number of contact points between the foundation and the soil. For typical containment buildings the effect of flexible side boundaries is probably negligible as far as the overall dynamic response is concerned. Foundation flexibility may be important when all buildings are constructed on a large basemat. Sensitivity studies are necessary.

- The use of simplified models and sensitivity studies to obtain bounding analyses is quite important.

- For embedded foundations the net rotational component of foundation motion, due to the spatial variation of ground motion, is necessary. Otherwise, the reduction in the translational component of motion would be unconservative. If no rotational component of motion is specified, then the surface motion should be applied directly at the foundation level without any reduction.

- Further study is required to determine if the use of the linear secant modulus for soil properties precludes the transmission of high frequency motion. Studies to date (16,17) are contradictory.

The main application of the above discussion and recommendations on soil structure interaction is in the area of structural response. The other important area of interest is foundation evaluation.

The different areas of application of soil-structure interaction analysis, structural response or foundation evaluation, can result in different requirements on the soil-structure interaction method. For example, while secondary nonlinearity probably has a relatively minor effect on structural response, it probably has a more significant effect on the stress history in the soil near the foundation of the structure. Conversely, in cases where basemat flexibility is of minor importance in structural response, it may be relatively more significant in its effect on foundation stress histories near the structure. Again, considering the spatial mesh refinement, the coarse mesh often adequate for kinematic purposes (for example, acceleration histories) may be inadequate for soil stress calculation purposes. Finally, the procedure used in a so-called equivalent linear method could and probably should be different depending on whether the method is "equivalent" in the sense of acceleration histories in the structure or stress histories in the soil foundation, or some other sense.

There is a logical implication direction; if we knew the soil constitutive properties adequately enough to estimate soil stresses accurately, then we would surely be able to estimate structural response adequately (considering the extensive existing research in the structural area compared to the lack of large scale soil tests). The converse is also true; as the above discussion suggests, our capability to estimate structure response due to soil structure interaction is presently poor so our present capability to estimate soil stresses must be worse.

We should also consider the more general implications of the procedures used in structural analysis and design for earthquakes. Quite often, the structural model used to estimate dynamic response is not used directly to obtain values for structural design. Secondary analyses (more detailed and often static) are performed which use the results of the dynamic analyses as input. The analogy for the purpose of evaluating the soil foundation would be to use the soil structure interaction analyses to obtain an estimate of the overall dynamic behavior and then using these results as input perform more detailed studies on the foundation material near the structure's foundation.

In summary, if accurate dynamic stresses in the soil foundation are required to evaluate foundation stability (for example, as in liquefaction analyses), it is a difficult and complex problem indeed. Analyses purporting to produce such stresses should be used with extreme caution, should never be performed with synthetic broad-band time histories, and the results always corroborated on a case-by-case basis with large-scale field experience rather than small specimen laboratory tests. There is an extraordinary amount of research required in this area before reliable analytical methods will be obtained. It is useful to recall that such analyses are attempting to estimate failure levels, something that's still quite elusive in structures.

III. STRUCTURES

A. General

There are many areas of conservatism that exist in the current NRC criteria for the seismic design of Nuclear Power Plant structures. This section attempts to identify some of these areas and make recommendations to reduce these often excessive levels of conservatism. A variety of topics are covered, including: Special Structures (Buried pipes, conduits, etc. and aboveground vertical tanks); Modal Response Combinations; Inelastic Seismic Design and Analysis of Structures; Damping Values for Seismic Design of Nuclear Power Plants; and Seismic Analysis Methods.

Because of the redundancy incorporated in SRP 3.7.2 and 3.7.3, it is recommended that:

- o Standard Review Plan sections 3.7.2 and 3.7.3 should be combined and rewritten into one SRP section 3.7.2 covering seismic system and subsystem analysis, and SRP section 3.7.3 should be devoted to special structures.

B. Special Structures:

The current Standard Review Plan (SRP) does not provide sufficient guidance concerning minimum requirements for an adequate seismic analysis and design of certain categories of special structures. These special structures include buried pipes, conduits, etc., underground horizontal tanks and aboveground vertical tanks. These types of structures have special seismic design requirements which are currently being interpreted in different ways by different designers. This lack of consistency in the seismic design of these special structures can result in cases of unconservative design.

Buried Pipes, Conduits, etc.

Although item 12 of each part of SRP Section 3.7.3 and the references contained therein provide good guidance regarding acceptable methods for the design of buried pipes, conduits, etc., this guidance is incomplete and leaves room for significantly differing interpretations. A considerable amount of work has been performed in this area in the last few years to expand upon the guidance and references given in Section 3.7.3. It should be pointed out that while item 12 of Section 3.7.3 talks about inertial effects with regard to buried pipes, conduits, etc., the real problem is that these buried structures are primarily subjected to relative displacement induced strains rather than inertial effects. These strains are induced primarily by seismic wave passage and by differential displacements between anchor points to buildings and the ground surrounding the buried structure.

The following recommendations deal with long, buried structures continuously supported by the surrounding soil and the connection of such structures into buildings or other effective anchor points. References 23-27 and 55-59 should be consulted for further details regarding these recommendations.

- Each of the following seismic induced loadings must be considered for long, buried structures:

1. Abrupt differential displacement in a zone of earthquake fault breakage.

2. Ground failures such as liquefaction, landsliding, lateral spreading and settlement.

3. Transient, recoverable deformation, shaking of the ground or anchor points relative to the ground.

Zones of abrupt differential displacement due to fault movement should be avoided for long, buried safety class structures. Severe loading on such structures due to ground failures should also be avoided by: a) rerouting to avoid areas of problem soils, b) removal and replacement of such soils, c) soil stabilization (e.g., by densifying, grouting, or draining), or d) supporting long, buried structures in soils not susceptible to failure (e.g., by deeper burial or pile foundations extending into stable soils). If avoidance is not possible, then special designs to conservatively accommodate the maximum predicted loadings from postulated abrupt differential displacement or ground failure must be utilized. These designs are beyond the scope of this standard and must be approved on a case-by-case basis.

- Two types of ground shaking induced loadings must be considered for design. These are:

1 Relative deformations imposed by seismic waves traveling through the surrounding soil or by differential deformations between the soil and anchor points.

2. Lateral earth pressures acting on the cross sections of the structural element.

References 23-27 and 55-59 give acceptable methodologies for determining design parameters associated with seismically induced transient relative deformations. The formulas given in these references are conservative and permissible for use in design. However, more sophisticated, analyses may be substituted in lieu of these formulas.

When computing the relative joint displacements and joint rotations, it is important that reasonable values of the apparent axial wave propagation speed, C_E, and the apparent curvature propagation speed, C_K, be used. The apparent wave propagation speeds, C_E and C_K, depend upon the wave type which results in the maximum ground velocity and acceleration. Wave types that must be considered are: Compressional waves; shear waves; and Rayleigh waves. It is recommended that:

o The apparent wave propagation speeds, C_E and C_K, to be used are as follows:

	Wave Type*		
Apparent Wave Propagation Speed	Compression	Shear	Rayleigh
C_E	C_c	$2*C_s$	C_R
C_K	$1.6*C_c$	C_s	C_R

*Numerical coefficients in this table account for the worst direction of wave propagation. See References 23 and 25 for a complete explanation and derivation.

where C_c, C_s and C_R are the effective compressional, shear, and Rayleigh wave velocities, respectively, associated with the wave travel path from the location of energy release to the location of the long, linear structure. For structures located close to an earthquake (less than about 2 to 5 focal depths), body waves (compression, and shear) will predominate while at far ranges (beyond 5 focal depths), Rayleigh waves are likely to predominate. Use of effective wave velocities associated with the soil at or near the ground surface is acceptable but generally overly conservative. The apparent wave propagation speeds, C_E and C_K, ahould generally be determined from a geotechnical investigation. In lieu of this investigation, it is permissible to use the Rayleigh wave speed corresponding to material at approximately one half a wave length below the ground surface for C_E and C_K.

- In addition to computing the forces and strains in the buried long, linear structure due to wave propagation effects, it is also necessary to determine the forces and strains due to the maximum relative dynamic movement between anchor points (such as a building attachment point) and the adjacent soil which occurs as a result of the dynamic response of the anchor point. Motion of adjacent anchor points should be considered to be out-of-phase so as to result in maximum calculated forces and strain in the buried structure.

- Forces and strains associated with dynamic anchor point movement should be combined with the corresponding forces and strains from wave propagation effects using the square-root-sum-of-the squares (SRSS) method.

- The forces and strains computed for buried structures due to wave propagation effects and dynamic anchor point movements can be treated as secondary (displacement controlled) forces and strains. Thus, for steel structures, the applicable secondary stress and strain limits may be used in lieu of primary stress and strain limits. For concrete structures, longitudinal strains should be limited to 0.3 percent (for compression) in lieu of the use of more conservative stress limits. When specially reinforced to insure ductile behavior, larger strain limits may be justified. Strain limits for crushing and cracking of concrete should be taken as 0.004 and 0.0002, respectively.

- Long, buried structures must also be designed to accommodate primary loadings (such as lateral earth pressure, dead and live loads) applied concurrently with the ground shaking induced secondary strains and forces.

C. Aboveground Vertical Tanks

The majority of aboveground fluid containing vertical tanks do not warrant sophisticated finite element fluid-structure interaction analyses for seismic loading. However, the commonly used alternative of analyzing such tanks by the "Housner-method" (Ref. 28) may, in some cases, be significantly unconservative. The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, this assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. More recent evaluation techniques (30, 31) have shown that for typical tank designs, the modal frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically significantly greater than the zero period acceleration, and thus the assumption of a rigid tank could lead to significantly unconservative design loadings.

The following recommendations made are based upon the information contained in References 28-31 and represent minimum requirements for the safe design of aboveground vertical tanks. These references also contain acceptable calculational techniques for the implementation of these recommendations. However, they are not intended to preclude the use of more sophisticated analytical procedures which account for each of the minimum requirements contained herein.

- A minimum acceptable analysis must incorporate at least two horizontal modes of combined fluid-tank vibration and at least one vertical mode of fluid vibration. The horizontal response analysis must include at least one <u>impulsive</u> mode in which the response of the tank shell and roof are coupled together with the portion of the fluid contents which moves in unison with the shell. Furthermore, at least the fundamental sloshing (<u>convective</u>) mode of the fluid must be included in the horizontal analysis.

- It is necessary to estimate the fundamental frequency of vibration of the tank including the impulsive contained fluid weight. It is unacceptable to assume a rigid tank unless such an assumption can be justified. The horizontal impulsive mode spectral acceleration, S_{a_1}, is then determined using this impulsive mode frequency and tank shell damping. In lieu of determining the impulsive mode fundamental frequency, it is permissible to use the maximum horizontal spectral acceleration associated with the tank support at the tank shell damping level.

- Damping values to be used to determine the spectral acceleration in the <u>impulsive</u> mode shall be based upon the values for tank shell material as specified in paragraph "F" of this report.

- In determining the spectral acceleration in the horizontal <u>convective</u> mode, S_{a_2}, the fluid damping ratio shall be taken as 0.5 percent of critical damping unless a higher value can be substantiated by experimental results.

- The maximum overturning moment, M_B, at the base of the tank should be obtained by the square-root-sum-of-squares (SRSS) combination of the impulsive and convective horizontal overturning moments. The uplift tension resulting from this base moment must be resisted either by tying the tank to the foundation with anchor bolts, etc., or by mobilizing sufficient fluid weight on a thickened base skirt plate.

- The seismic induced hydrodynamic pressures on the tank shell at any level can be determined by the square-root-sum-of-squares (SRSS) combination of the impulsive (P_1), convective (P_2), and vertical (P_V) hydrodynamic pressures. The hydrodynamic pressure at any level must be added to the hydrostatic pressure at that level to determine the hoop tension in the tank shell. This hoop tension must be treated as a primary stress.

- Either the tank top head must be located at greater than the slosh height, d, above the top of the fluid or else must be designed for pressures resulting from fluid sloshing against this head.

- At the point of attachment, the tank shell must be designed to withstand the seismic forces imposed by the attached piping. An appropriate analysis must be performed to verify this design.

- The tank foundation must be designed to accommodate the seismic forces imposed by the base of the tank. These forces include the hydrodynamic fluid pressures imposed on the base of the tank as well as the tank shell longitudinal compressive and tensile forces resulting from the base moment, M_B.

D. Modal Response Combinations

As currently written, Standard Review Plan Section 3.7.2 and Regulatory Guide 1.92 do not properly address the problems of the response combination of high frequency modes or the response combination of closely spaced modes. The SRSS combination of high frequency modes, as currently allowed, may be significantly unconservative in some cases while the response combination of closely spaced modes using the "Double-Sum" method for SRSS combination may be too conservative.

Section 3.7.2 of the SRP requires that sufficient modes be included in a dynamic response analysis to insure that an inclusion of additional modes does not result in more than a 10% <u>increase</u> in responses. The implementation of this requirement may require the inclusion of modes with natural frequencies at which the spectral acceleration roughly returns to the peak zero period acceleration. An SRSS combination of such modes is highly inaccurate and may be significantly unconservative.

1737 119

The SRSS combination of modal responses is based on the premise that peak modal responses are randomly time phased. This has been shown to be an adequate premise throughout the majority of the frequency range for earthquake type responses. However, at frequencies approximately equal to the frequency at which the spectral acceleration, S_a, roughly returns to the peak zero period acceleration, ZPA, and

greater, this is not a valid premise. At these high frequencies, the seismic input motion does not contain significant energy content and the structure simply responds to the inertial forces from the peak ZPA in a pseudo-static fashion. The phasing of the maximum response from modes at these high frequencies (roughly 33 Hz and greater for the Regulatory Guide 1.60 response spectra) will be essentially deterministic and, in accordance with this, pseudo-static response to the peak ZPA.

The frequency above which the SRSS procedure for the combination of modal response tends to break down is not well defined. Possibly research should be conducted on this point. However, it is believed that this frequency roughly corresponds to the frequency at which the spectral acceleration approximately returns to the ZPA.

There are several solutions to the problem of how to combine responses associated with high frequency modes when the lower frequency modes do not adequately define the mass content of the structure.

The following procedure appears to be the simplest and most accurate one for incorporating responses associated with high frequency modes.

o 1. Determine the modal responses only for those modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz in the case of the Regulatory Guide 1.60 response spectra).

Combine such modes in accordance with current rules for the SRSS combination of modes.

o 2. For each degree-of-freedom included in the dynamic analysis, determine the fraction of degree-of-freedom (DOF) mass included in the summation of all of the modes included in Step 1. This fraction F_i for each degree-of-freedom i is given by:

$$F_i = \sum_{m=1}^{M} PF_m * \phi_{m,i}$$

where

m is each mode number

M is the number of modes included in Step 1.

PF_m is the participation factor for mode m

$\phi_{m,i}$ is the eigenvector value for mode m and DOF i

Next, determine the fraction of DOF mass not included in the summation of these modes:

$$K_i = F_i - \bar{\delta}$$

where

$\bar{\delta}$ is the Kronecker delta which is one if DOF i is in the direction of the earthquake input motion and zero if DOF i is a rotation or not in the direction of the earthquake input motion.

If, for any DOF i the absolute value of this fraction K_i exceeds 0.1, one should include the response from higher modes with those included in Step 1.

o 3. Higher modes can be assumed to respond in phase with the peak ZPA and thus with each other so that these modes are combined algebraically which is equivalent to pseudo-static response to the inertial forces from these higher modes excited at the ZPA. The pseudo-static inertial forces associated with the summation of all higher modes for each DOF i are given by:

$$P_i = ZPA * M_i * K_i$$

where

P_i is the force or moment to be applied at degree-of-freedom (DOF), i

M_i is the mass or mass moment of inertia associated with DOF i

The structure is then statically analyzed for this set of pseudo-static inertial forces applied to all of the degrees-of-freedom to determine the maximum responses associated with the high frequency modes not included in Step 1.

o 4. The total combined response to high frequency modes (Step 3) are SRSS combined with the total combined response from lower frequency modes (Step 1) to determine the overall structural peak response.

This procedure is easy because it requires the computation of individual modal responses only for the lower frequency modes (below 33 Hz for the Regulatory Guide 1.60 response spectrum). Thus, the more difficult higher frequency modes do not have to be determined. The procedure is accurate because it assures inclusion of all modes of the structural model and proper representation of DOF masses. It is not susceptible to inaccuracies due to an improperly low cutoff in the number of modes included.

An acceptable alternative to this procedure is as follows:

o Modal responses are computed for a sufficient number of modes to insure that the inclusion of additional modes does not result in more than a 10% <u>increase</u> in the total response. Modes with natural frequencies less than that at which the spectral acceleration approximately returns to the ZPA (33 Hz in the case of the Regulatory Guide 1.60 response spectrum) are combined in accordance with current rules for the SRSS combination of modes. Higher mode responses are combined algebraically (i.e., retain sign) with each other. The absolute value of the combined higher modes are then added directly to the total response from the combined lower modes.

The method in Regulatory Guide 1.92 for the response combination of closely spaced modes represents a deviation from the way the so-called "Double-sum" method was first proposed.[32] In Regulatory Guide 1.92 absolute signs are used for individual modal responses in lieu of the algebraic signs as required by the derivation contained in Reference 32. Studies (33, 34) have shown that the "Double-sum" method using the algebraic signs provides more accurate results for peak combined response than does the pure SRSS method. However, this "Double-sum" modification of the pure SRSS method only results in minor improvement in the vast majority of cases. Additionally, the studies presented in Reference 34 shows that the use of the absolute signs with the "Double-sum" method introduces considerable conservative bias to the peak combined response with closely spaced modes. In fact, with the introduction of absolute signs, the results are considerably less accurate than those obtained from the pure SRSS method. Based on these observations, the following recommendations are made:

- o No special procedures, other than the normal SRSS method, are required for the modal combination of closely spaced modes.

- o If closely spaced modes must receive special treatment, then one should use relative algebraic signs for individual modal responses and not absolute signs in the "Double-sum" method.

E. Inelastic Seismic Design and Analysis of Structures:

Numerous observations of the actual performance of structures subjected to seismic motions have demonstrated the capacity of structures to absorb and dissipate a considerable amount of energy when strained in inelastic response. The energy absorption obtained from a linear elastic analysis carried up to the design or yield level is only a fraction of the total energy absorption capability of a structure. Unless corrected for inelastic response capability, a linear elastic response analysis is incapable of accounting for the inelastic energy absorption capacity of a structure.

A number of studies have demonstrated the reduction in required strength permitted by accounting for a limited amount of inelastic energy absorption capability and have made such recommendations (see for instance, References 14, 15, 35, 36, 37). Equivalencing computed response and the results of damage surveys conducted after major earthquakes have required accounting for the inelastic energy absorption capability of structures. Otherwise, computed responses predict far greater damage than actually observed.

As a result of the numerous studies and observations confirming the inelastic capacity of structures, it is recommended that:

- Regulatory Guides and Standard Review Plans should specifically allow a limited amount of inelastic energy absorption for the SSE level earthquake. Both simplified inelastic response spectrum techniques and nonlinear time history analysis techniques are acceptable for design and analysis.

Reference 15 shows that both the Blume Reserve Energy Technique, and the Newmark Inelastic Response Spectrum Technique adequately predict the inelastic response of typical structures as compared to inelastic time-history analyses, so long as the total inelastic response is low.

Based on Reference 35, it is recommended that:

o Structures and systems be classified into 4 seismic design classifications depending upon their operability requirements. Table 3 (reproduced from reference 35) presents recommendations for permissible systems ductility factors for each seismic design classification. The ductility factors recommended in this table adequately account for:

a. The definition of ductility factor presented in Figure 1.

b. The approximate nature of simplified inelastic dynamic analysis techniques.

c. The difference between maximum member ductility factor, maximum story drift ductility factor, and systems ductility factor, and

d. The relative importance of each class of structure or system.

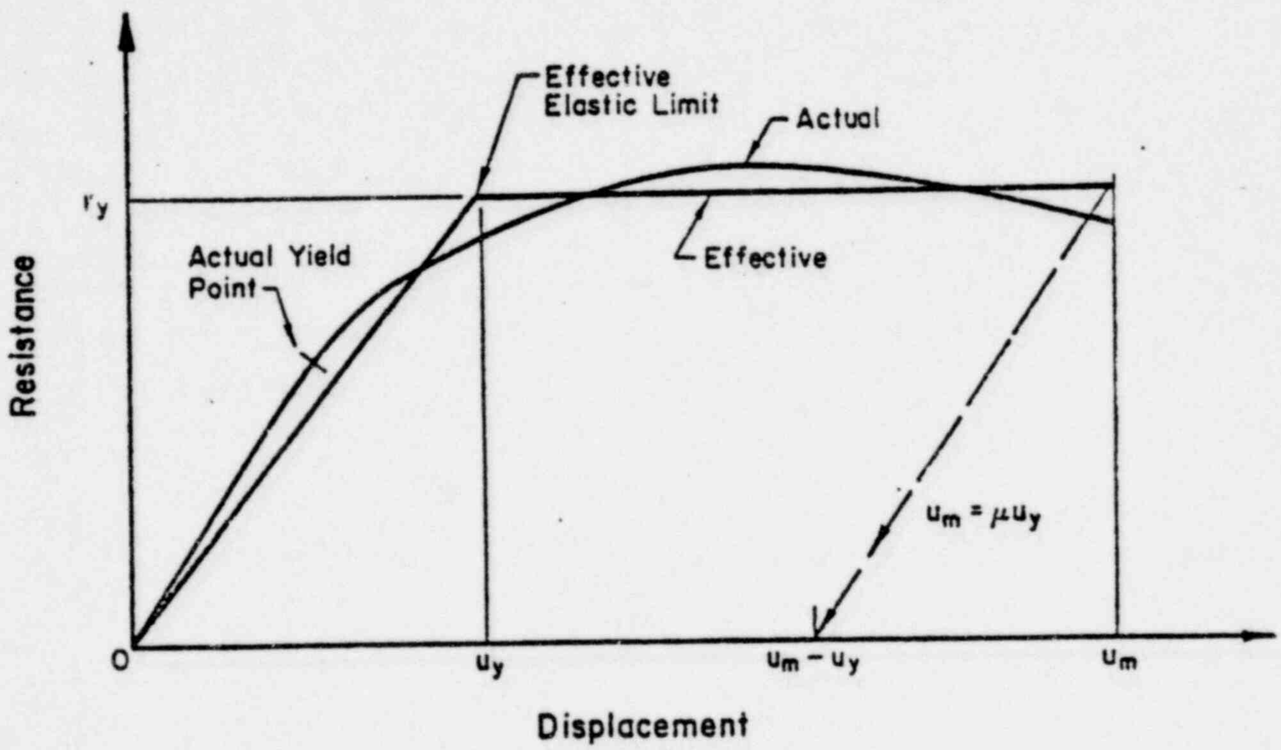


Figure 1. Ductility Factor: $\mu = \frac{u_m}{u_y}$

TABLE 3
Seismic Design Classification

CLASS	DESCRIPTION
I-S	Equipment, instruments, or components performing vital functions that must remain operative during and after earthquakes; Structures that must remain elastic or nearly elastic; Facilities performing a vital safety-related function that must remain functional without repair. Ductility factor = 1 to 1.3.
I	Items that must remain operative after an earthquake but need not operate during the event; Structures that can deform slightly in the inelastic range; Facilities that are vital but whose service can be interrupted until minor repairs are made. Ductility factor = 1.3 to 2.
II	Facilities, structures, equipment, instruments, or components that can deform inelastically to a moderate extent without unacceptable loss of function; Structures housing items of Class I or I-S that must not be permitted to cause damage to such items by excessive deformation of the structure. Ductility factor = 2 to 3.
III	All other items which are usually governed by ordinary seismic design codes; Structures requiring seismic resistance in order to be repairable after an earthquake. Ductility factor = 3 to 8, depending on material, type of construction, design of details, and control of quality.

It is further recommended that:

o The Standard Review Plan permit the use of limited nonlinear dynamic analysis techniques using the lower bound system ductility factors presented in Table 3 for the design of seismic classes I-S, I, and II and the upper bound values for re-evaluation of existing structures. Class III structures can be designed using ordinary seismic design codes.

The lower bound system ductility factors presented in Table 3 for seismic classes I-S, and I, are sufficientlv low so as to not require special ductility requirements to insure this level of ductility. A system ductility factor of 1.3 can easily be achieved by application of the provisions of normal design codes. The system ductility limit of 2 assigned for seismic Cla s II may require additional minimum ductile design requirements beyond those in normal design codes.

As stated earlier, the Blume Reserve Energy Technique[38-41] and the Newmark Inelastic Response Spectrum Technique[35-37] have been shown to adequately predict the inelastic response of structures for low overall levels of inelastic response, and as such are acceptible simplified techniques for use in the inelastic design and/or analysis of structures. An alternative method (Reference 14) proposed by Nelson, uses the results of an elastic analysis to predict the ductility demand of structural components. This method differs from the other methods in that local member ductilities are the quantities of interest, and, therefore, a correlation between the overall allowable system ductilities, as given in Table 3, and local member ductilities needs to be made.

F. Damping Values for Seismic Design of Nuclear Power Plants:

The energy dissipation within a structure due to material and structural damping is dependent on a number of factors such as types of joints or connections, structural material, stress level, and magnitude of deformations. In a dynamic elastic analysis, this energy dissipation is usually accounted for by specifying an amount of viscous damping that would result in energy dissipation in the analytical model equivalent to that expected to occur in the real structure as a result of material and structural damping.

A recent paper by Newmark and Hall[35] summarizes levels of damping from a variety of sources as functions of the type and condition of the structure as well as the stress level of interest. Based on this information, it is recommended that:

- The damping values as given in Table 1 of Regulatory Guide 1.61 should be replaced by the value as given in the following table:

TABLE 4. RECOMMENDED DAMPING VALUES

Stress Level	Type and Condition of Structure	Percentage Critical Damping
Working stress no more than about 1/2 yield point	a. Vital piping	1 to 2
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2 to 3
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	a. Vital piping	2 to 3
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
	c. Prestressed concrete with no prestress left	7 to 10
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

1737 130

The lower levels of the pair of values given for each item are considered to be nearly lower bounds, and are therefore highly conservative and suitable for design; the upper levels are considered to be average or slightly above average values, and are acceptable for re-evaluation of existing structures.

The stress levels indicated in the table are total stresses, not just seismic. The damping values used should be based on the highest stress level in the structure or component of interest. Interpolation between stress levels and the structure type and condition is acceptable.

G. Seismic Analysis Methods:

The section of Standard Review Plan 3.7.2 on Procedures Used for Analytical Modeling needs to provide additional guidance with regard to the appropriate modeling techniques that are acceptable for typical Nuclear Power Plant structures. The following recommendations are taken from Reference 50 and should be used to augment this section of SRP 3.7.2.

- Typically, normal nuclear power plant buildings can be modelled in one of the following ways, according to their structural characteristics:

  (a) Shell-type buildings with internal structures (e.g. typical containment buildings): stick model or finite element models.

  (b) Box-like buildings (e.g. typical auxiliary buildings): can usually be modelled as rigid structures on elastic foundation; if this is not the case, finite element or stick models may be considered but some difficulties may be encountered because of the complexity and dimensions of the structure.

(c) Frame-like buildings: stick models.

(d) Slender chimney-like structures: any reasonable model.

IV. EQUIPMENT AND COMPONENTS

A. General

This section presents recommendations that upgrade the seismic design criteria for subsystems, equipment and components, that eliminate unnecessary conservatisms and that attempt to bring the Standard Review Plan and Regulatory Guides up to the current state-of-the-art in this area. Some of these recommendations are aimed at clarification of the SRP and Regulatory Guides while others are specifically intended to reduce areas of excessive conservatism.

The performance of actual Power Plants during earthquakes tend to verify the assertion that excessive conservatisms are introduced during the seismic design methodology chain for structures, subsystems, equipment and components. A recent review by Cloud[51] of the performance of Power Plant piping in actual earthquakes shows that even though ground accelerations were in most cases greater than the design value, there were no failures of piping. In cases reported by Cloud, it is understood that pipe distress has occurred with slope instability problems.

Areas that are covered in this section include: Seismic Analysis Methods; Direct Generation of Floor Spectra; Effects of Uncertainties on Floor Spectra; Generation of Floor Spectra for Structures with Limited Inelastic Response; Eccentricity Considerations for Floor Design Response Spectra; and Number of Earthquake Cycles During Plant Life.

B. Seismic Analysis Methods

The section of Standard Review Plan 3.7.3 dealing with procedures used for analytical modeling needs to provide additional guidance with regard to the modeling technques that are appropriate for specific equipment and components of Nuclear Power Plants. The following recommendations are taken from Reference 50 and should be used to augment this section of SRP 3.7.3.

- o The modeling of structures, components and equipment can be divided into the following five categories:
  - (a) <u>Rigid Body Model</u> - For these items the item itself is assumed rigid (i.e., fundamental frequency typically ≥ 33 Hz). The model is typically represented as a rigid body with attachment at support points represented by springs or stiffness or flexibility matrices. Response of the item then would be by rocking or translational modes of vibration at support points. Typical valve, pumps, motors, fans, some heat exchangers fall into this category.
  - (b) <u>Single Mass Model</u> - For these items the total mass is assumed to be lumped at a single point with the composite stiffness restraining the mass represented as a single element. More than one degree of freedom may be permitted. In general this modeling is considered as an alternate to method (a) and is applicable to the same types of items.
  - (c) <u>Beam Model or One Dimensional Finite Element</u> - This type of modeling is typically applied to beams, columns, frames, piping, ducts, cable trays, conduit, symmetric tanks, cabinets, storage racks, pressure vessels and heat exchangers and may be formulated as continuous or one dimensional finite elements in two or three dimensional space. Representation of masses may be made by lumped parameter which develop a diagonalized elemental mass matrix or by means of

consistent mass matrices which have the same off diagonal form as the elemental stiffness or flexibility matrices.

(d) Plate or Shell or Two Dimensional Finite Elements - This type of modeling is typically performed on items whose primary mode of failure is by biaxial bending, plane stress or plane strain. Included in this category are, typically, foundation media, cabinents, slabs and tanks, pressure vessels and heat exchangers whose shells support significant eccentric loads which would tend to excite shell or lobar modes of vibrations.

(e) Three Dimensional Finite Element - This type of modeling has not been used extensively to date but would be applied to thick wall vessels.

C. Direct Generation of In-Structure Response Spectra

Currently, Section 3.7.1 of the Standard Review Plan states that: "For the analysis of interior equipment, where the equipment analysis is decoupled from the building, a compatible time history is needed for computation of the time-history response of each floor. The design floor spectra for equipment are obtained from this time history information." Furthermore, it is standard practice to require that response spectra obtained from this artificial time history of motion should generally envelope the design response spectra for all damping values to be used. In addition, Section 3.7.2 of the Standard Review Plan encourages the use of a time history approach to generate floor spectra by stating: "In general, development of the floor response spectra is acceptable if a time history approach is used. If a modal response spectra method of analysis is used to develop the floor response spectra, the justification for its conservatism and equivalency to that of a time history method must be demonstrated by representative examples".

The use of time histories for which the response spectra envelope the design response spectra for all damping values tends to artificially introduce added and unnecessary conservatism into the analysis. The amount of conservatism depends upon the ability of the analyst to "tinker" with the time history in order to cause a minimum amount of deviation between the resultant response spectra and the design response spectra. After much "tinkering", the time history no longer closely resembles an earthquake generated time history but does provide a relatively smooth response spectra which reasonably closely envelopes the design response spectra. Reference 3 indicates that the average industry-generated artifical time history tends to introduce about 10 percent conservatism except at high frequencies for which the conservatism is about 20% at 33 Hz.

It has also been observed that different artificial time histories, both of which result in response spectra which adequately envelope the Regulatory Guide 1.60 response spectra, can lead to floor spectra which may differ by a factor of 2 or more (for instance, see Reference 6). Use of the artificial time history method results in a small arbitrary amount of conservatism on the average and considerable dispersion in the resultant floor spectra, as a function of the time-history used.

A number of recent algorithms have been developed to compute the floor response spectra directly from the ground response spectra without time-history analysis (References 42-47). All of these methods are based upon sound theoretical backgrounds and are suitable for adaptation on computers. Because these algorithms are efficient, parametric studies are economically feasible. These methods use the SRSS method for combination of components, and produce smooth, realistic spectra. These methods in conjunction with parametric studies would reduce the uncertainties associated with floor spectra generation through the use of artificial time-histories. Based on these observations, the following recommendation is made:

- o The Standard Review Plan should give equal wieght to the use of both time-history analysis methods and direct solution methods for the generation of in-structure response spectra.

D. Effect of Uncertainties on In-Structure Response Spectra

Regulatory Guide 1.122 requires the broadening of floor spectra to account for uncertainties in the structural response characteristics. This broadening of floor spectra to account for uncertainty is certainly valid and should be retained when a single time history analysis is done to generate in-structure response spectra. However, the same uncertainties which lead to broadening of the floor spectra also lead to a reduction in the peak spectral amplitudes with a given probability of exceedance. This process of considering uncertainty where it is harmful (i.e., broadening of frequencies for peak response) and ignoring uncertainty where beneficial (i.e., not lowering the probable peak response at any given frequency) further leads to arbitrary conservatism in the resultant design floor spectra.

Studies have been performed (References 7 and 48) which compare equal probability of exceedance floor spectra with deterministic floor spectra. The equal probability of exceedance floor spectra show much broader peaks with much lower maximum amplitudes for each peak than do the deterministic spectra. For 2% damping, the deterministic peaks may be more than a factor of 2 greater than the equal probability of exceedance spectra. Thus, considerable conservatism is introduced within the broadened peak region of the deterministic spectra. On the other hand, slight unconservatism as compared to the equal probability of exceedance spectra may occur at frequencies outside of the region of broadened peaks.

If the direct generation of in-structure response spectra by modal response spectrum techniques, as described in the previous section, is allowed, it would then be a practical matter to generate equal probability of exceedance in-structure response spectra. These floor spectra would account for the uncertainty in the ground response spectrum, and the response characteristics (frequencies, damping, etc.). Such spectra will be flatter than current spectra with the valleys raised and peaks lowered, and as such it is believed that they would represent a more rational seismic design basis for subsystem design than do deterministic in-structure response spectra. Therefore, it is recommended that:

- o The Standard Review Plan should allow the use of probabilistic generated in-structure response spectra corresponding to an 0.84 nonexceedance probability (NEP) in lieu of deterministic in-structure response spectra. (The 0.84 NEP is conditional on the SSE occurrence.)

If time history analysis methods are to be used to generate in-structure response spectra, several options are available. These are:

a) A single synthetic time history which envelops the MSD ground design response spectrum can be used to generate in-structure response spectra. Peak broadening to account for uncertainties is done as currently specified in R.G. 1.122.

b) Multiple (≥7) real time histories properly scaled for frequency content, amplitude, energy content, etc. can be used. The MSD spectrum of the real time histories should be at or above the MSD ground design response spectrum frequency by frequency. Uncertainties are accounted for with this technique through variation of parameters in the multiple analyses.

c) Multiple (≥5) synthetic time histories each being mean centered about the MSD ground design response spectrum and the median of their spectra being at or above the MSD of the ground design response spectrum can be used to generate in-structure response spectra. As above, uncertainties are accounted for through variation of parameters in the multiple analyses.

Figures A and B outline two different ways 0.84 NEP floor spectra could be obtained using multiple time histories. It will be useful to review the mechanics of the procedures suggested in these figures.

The procedure outlined in Fig. A applies to the case where real time histories are used.

In Block 1, ≥7 such histories are selected. The requirements on these histories are not discussed in detail here, but at a minimum their peak acceleration should correspond to the value used for the site, and their frequency content should also reflect site conditions.

1737 139

In Block 2, one dimensional soil analyses should be used to select soil properties for use in the SSI analyses. As discussed previously, factors of 2.0 and 1/2.0 define the range of soil properties, and the ≥7 sets of properties lie within this range.

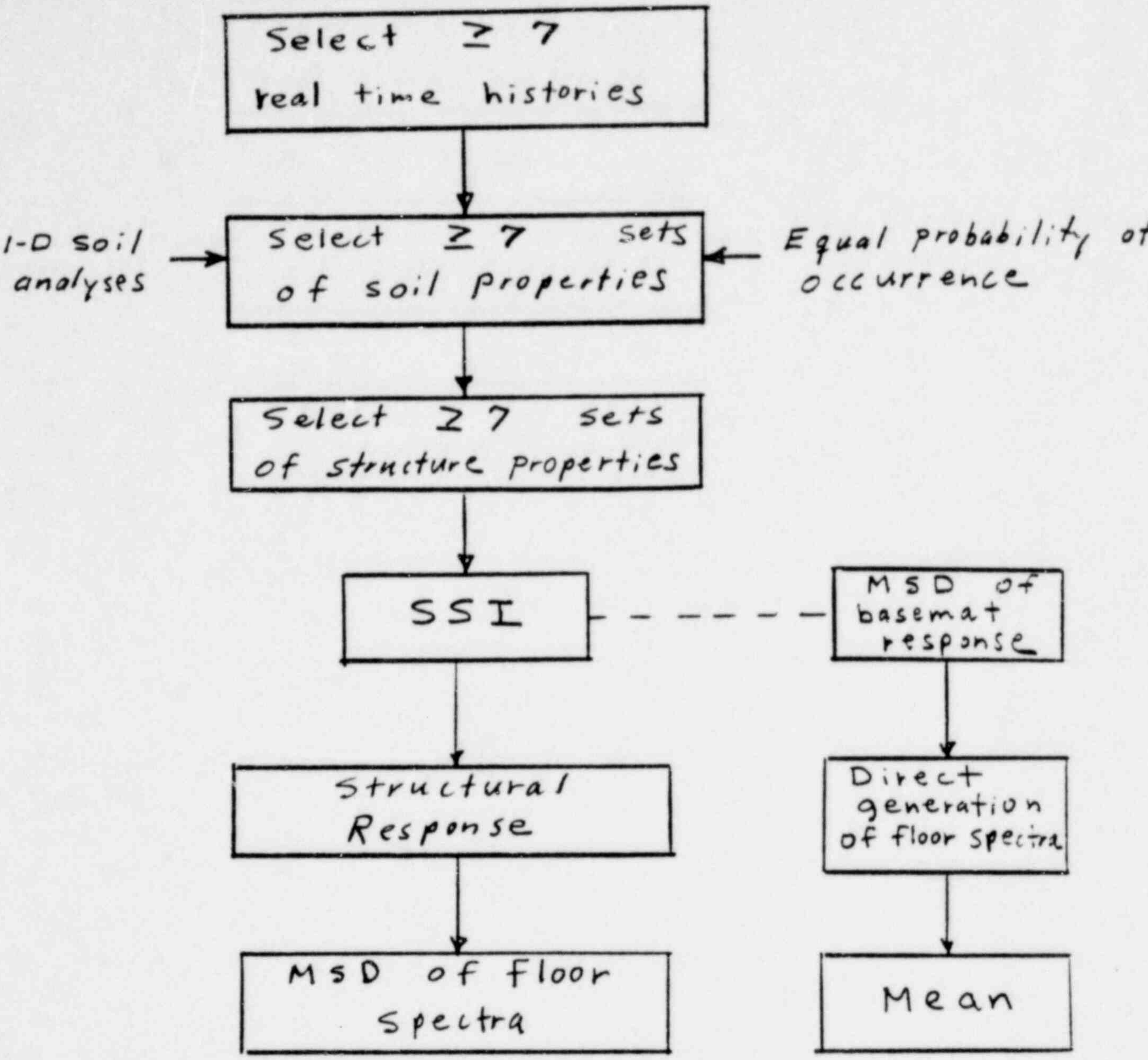
In Block 3, ≥ 7 sets of structural properties (for example, frequency and damping) should be selected. No ranges can be given at this time, although the range for damping is probably much larger than for frequency. Work being executed on the SSMRP at this time will be available before these recommendations can be implemented. This will be used to define the appropriate factors.

In Blocks 4 and 5, the SSI and structural response calculations are executed. Note that ≥ 7 calculations are suggested, not 7 x 7 x 7, (Blocks 1, 2, and 3). In each calculation time history results are contemplated. Admittedly, this is more calculation than is typically required today, but the economic impact is much less severe than might at first appear. This is because one of the most significant costs is associated with mathematical model development rather than analysis. This cost is not multiplicative for each model analyzed since what is proposed is to modify the parameters in the basic model for each of the ≥ 7 analyses. Further, for various reasons, multiple analyses are often performed in present practice, even though not required. The overall benefits of the suggested procedure (for example, smoother, less sharply peaked floor spectra without additional conservatism introduced by peak broadening, spectra easier to replicate in tests; recognition and direct inclusion of uncertainty, more nearly equal probability of exceedance across the frequency range of interest; etc.) are believed to significantly outweigh any disadvantages.

In Block 6, the MSD of the floor spectra from the ≥ 7 individual analyses is calculated. The MSD is used (rather than, for example, the mean) to introduce the appropriate degree of conservatism across the frequency range (conservatism already being included in the peak acceleration in Block 1). These MSD floor spectra could be used with the current multisupport capability which exists in the industry. Note that this method does not require broadening of spectra as this effect is included directly. It would be acceptable to carry the methodology suggested in Fig. A to include ≥7 time history results in mechanical subsystems, for example, piping, and then compute the MSD at the stress level, but this is not being suggested as required.

Blocks 7, 8, 9 are an alternate approach using one of the recent methods currently available for the direct generation of floor spectra without obtaining time history analysis results. This could be extended to Blocks 1 through 6, include the effect of uncertainty in the models, and eliminate the need for ≥7 time hiistory analyses entirely.

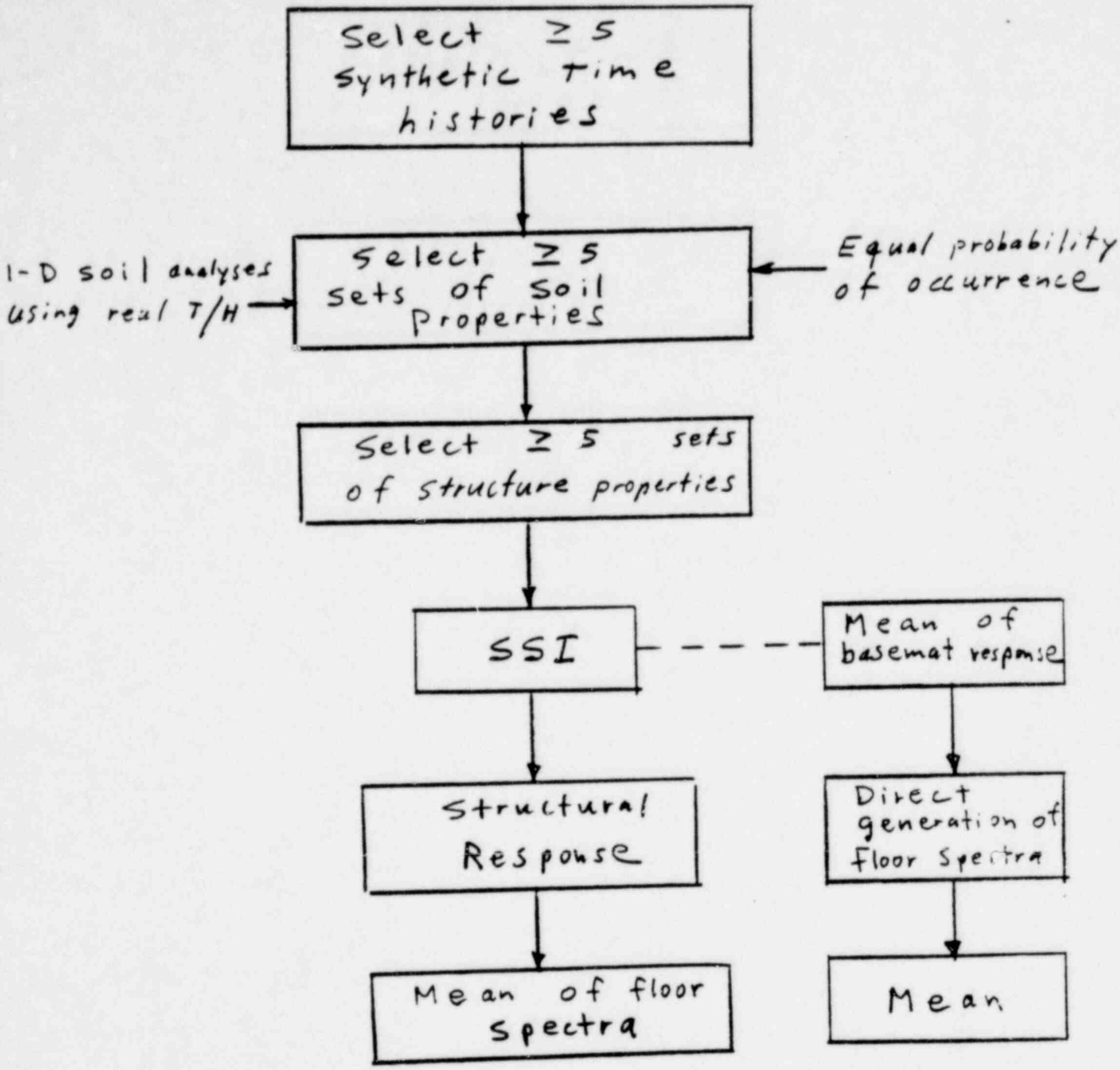
The approach outlined in Fig. B is essentially the same as in Fig. A, except the MSD requirement is introduced in the broad band nature of the synthetic time histories, (Block 1), and thus mean results are appropriate at succeeding steps. Additionally, fewer time history analyses are required using synthetic histories because of their statistical nature.



MSD - Mean plus one standard deviation

1737 142

Figure A - Real Time History Approach



1737 143

Figure B - Artificial or Synthetic Time History Approach

E. Generation of In-Structure Response Spectra for Structures with Limited Inelastic Response

As previously indicated, the seismic input to structure supported subsystems is generally defined in terms of floor response spectra. Therefore, it is necessary to generate elastic floor response spectra at various locations on the structure for use as input to the subsystem seismic analysis. For the case in which a limited amount of inelastic response of the structure has been allowed, these elastic floor spectra should be modified to account for the inelastic response of the structure.

Comparisons of elastic vs. inelastic floor spectra[9] for low levels of overall inelastic structure response as well as observations by Kennedy[21] have indicated the following characteristics between elastic and inelastic calculated floor spectra:

1. There is a reduction in peak spectral acceleration roughly corresponding to $1/\mu$ where μ is the system ductility factor.
2. There is generally a reduction in the frequency of the peak spectral acceleration roughly corresponding to $\sqrt{1/\mu}$.
3. There may be an increase in spectral acceleration in the high frequency regime. This potential increase is uncertain and is difficult to predict, but is small for small system ductility factors.
4. The broadened elastic calculated elastic spectra tend to envelope the inelastic calculated elastic spectra when the systems ductility factor is less than 1.3.

1737 144

Based on these observations, it is recommended that, for structures in which a limited amount of inelastic energy absorption is allowed, that the elastic calculated floor response spectra be modified to account for the inelastic response of the structure as follows:

- The elastic calculated floor response spectra should be used as subsystem input for subsystems mounted on Class I-S, and I structures where the system ductility factor is limited to 1.3 or less.
- For Class II structures in which the system ductility factors exceeds 1.3, it is necessary to obtain both elastic and inelastic calculated elastic floor spectra, and the design elastic floor spectra should envelope both. For the computation of inelastic calculated elastic floor spectra with system ductility factors less than 2, it is permissible to use a simplified model of the structure which accurately reproduces the elastic response and roughly approximates the inelastic response.
- Load combinations, load factors, and allowable strengths are to be unchanged from those used when inelastic energy absorption capability is not included.

The allowance of nonlinear response of piping and equipment is an area that needs careful research, especially in terms of ways of inspecting piping and equipment in a nondestructive manner to verify that the resistance capability has not degraded after some years of service, and in fact, can still be mobilized. At such time as there is an improvement in the understanding in this area, one would expect that it might be possible to permit some degree of nonlinear behavior in piping and equipment. For the present the following recommendation is made:

- Inelastic response of piping and equipment should not be permitted pending additional research on this topic.

F. Eccentricity Considerations for Floor Design Response Spectra

The sections of Regulatory Guide 1.122 and Standard Review Plan 3.7.2 dealing with the development of floor design response spectra need to indicate the necessity for modifying the floor design response spectra for the case in which there is accidental and actual eccentricity between the center of rigidity and center of mass at a given floor elevation. It is recommended that the following statement be added to R.G. 1.122 and SRP 3.7.2:

- "In symmetric structures, as well as unsymmetric structures, the floor design response spectrum should be modified to account for actual eccentricities between the center of mass and center of rigidity as well as an accidental eccentricity equal to 5% of the largest plan dimension of the structure. This additional response is a function of the distance of the system, subsystem or component from the center of rigidity of the structure. The accidental eccentricity should be algebraically combined with the actual eccentricity in such a way as to produce the maximum overall response when combined with the translational floor response for a particular system, subsystem or component."

G. Number of Earthquake Cycles During Plant Life

Section 3.7.3 of the Standard Review Plan requires that at least one safe shutdown earthquake (SSE) and five operating basis earthquakes (OBE) should be assumed to occur during the plant life. When coupled with the high load factors required, the requirement of five OBE's is excessively conservative. A preliminary comparison by Kennedy [21] of the ratio of the OBE levels assigned for operating reactors in the United States, to the estimated acceleration in rock with a 90% nonexceedance probability during a 50 year life (taken from Reference 49) shows that on the average, the OBE acceleration exceeds that estimated to correspond to the 90% nonexceedance probability in a 50 year life. This would indicate that, on the average, the OBE acceleration has more than a 90% nonexceedance probability during a 50 year life. Therefore, it is recommended that:

- The Standard Review Plan should only require that <u>two</u> operating basis earthquakes (OBE's) be assumed to occur during the plant life.

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