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Value/Impact Assessment for Seismic Design Criteria

USI A-40

David W. Coats, Jr., and David A. Lappa

Prepared for
U.S. Nuclear Regulatory Commission

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ABSTRACT

In October 1981, the Nuclear Regulatory Commission approved a reorganization that resulted in the establishment of the Committee to Review Generic Requirements (CRGR). The charter for the CRGR requires that written justification accompany all proposed new regulatory requirements submitted to the CRGR for review.

At the request of the Nuclear Regulatory Commission's Generic Issues Branch, Lawrence Livermore National Laboratory has provided the required written justification to accompany proposed new regulatory requirements to SRP Sections 3.7.1, 3.7.2, and 3.7.3. These proposed new requirements are the result of technical studies performed, as part of the Unresolved Safety Issues (USI) A-40 program, by LLNL and others. NUREG/CR-1161, "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria", by LLNL, provided the technical resolution to USI A-40 and was the basis for the proposed new recommendations. The report contained herein presents a technical evaluation and value/impact assessment of the proposed new requirements.

CONTENTS

CONTENTS

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF TABLES

ACKNOWLEDGEMENTS

The authors would like to acknowledge the timely and valued technical contributions made to this study by Dr. Robert P. Kennedy and Dr. James J. Johnson, Structural Mechanics Associates; and Dr. Paul D. Smith, Lawrence Livermore National Laboratory. Dr. Kennedy provided input to the report concerning high-frequency mode combination methods, and methods of seismic analysis of above-ground tanks. In addition, his contributions to NUREG/CR-1161 were used extensively throughout this report. Dr. Johnson performed the calculations for the Power Spectral Density (PSD) comparisons; multiple time histories vs. SRP approach comparisons of force and displacement ratios for typical piping systems; and the comparison of in-structure response spectra calculated by SRP procedures vs. a best estimate, multiple time history approach. He also provided the bulk of the discussion dealing with soil-structure interaction. Dr. Smith provided general review and guidance throughout the project. We would also like to express our appreciation to NRC staff members S. K. Shaukat (GIB); D. Jeng, (SGEB); and N. Chokshi, (SGEB), for their comments and contributions.

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EXECUTIVE SUMMARY

The Committee to Review Generic Requirements (CRGR) was established in October, 1981 as part of a U. S. Nuclear Regulatory Commission (NRC) approved reorganization, and has the responsibility to review all proposed new regulatory requirements and recommend approval or disapproval to the Executive Director of Operations. Proposed new requirements submitted to the CRGR must be accompanied by written justification. This justification package must include (among other things) an assessment of the risk reduction expected from implementing the proposed requirment and an estimate of the costs to the NRC and the licensees.

The objective of the work contained herein is to provide technical support to the Generic Issues Branch (GIB) in preparing value/impact assessments of proposed new regulatory requirements to Standard Review Plan (SRP) sections 3.7.1, 3.7.2, and 3.7.3, dealing with seismic design criteria.

Many of the proposed changes represent alternative procedures or clarifications of existing requirements. Discussions with GIB staff members established that, for these cases, no explicit estimate of changes in risk or cost are required. Where appropriate, however, the technical issues involved and the potential benefits and impact of the proposed change on seismic response and risk are qualitatively discussed and quantitatively assessed (where possible) through the use of engineering judgement, experience data, and recent research, but not through the explicit use of seismic PRA calculations.

A total of 24 proposed changes were identified from marked-up copies of the SRP provided to Lawrence Livermore National Laboratory (LLNL). Of these new requirements, 14 were identified as having a potential impact on PRA results. These 14 proposed new requirements (some of which are related) formed the basis for the identification of eight task areas for which a value/impact assessment and/or a technical discussion would be provided. These eight areas are as follows:

1. Design Time History
2. Development of Floor Response Spectra and Effects of Parameter Variations on Floor Response Spectra
3. Percentage of Critical Damping Values
4. Soil-Structure Interaction
5. Seismic Analysis Methods
6. Seismic Analysis Methods and Combination of Modal Responses
7. Methods of Seismic Analysis of Above-Ground Tanks
8. Category I Buried Piping, Conduits and Tunnels

Of the above, only areas 1, 6, and 7 were identified as requiring an analytical value/impact assessment (i.e., PRA). Proposed new requirements related to the remaining areas are either editorial in nature, options, or clarifications of existing NRC requirements. However, technical discussions of all eight areas have been included in this report.

The results of the analytical evaluations made for areas 1, 6, and 7, using the Zion site as a base case, indicate that the proposed new requirements associated with these areas would have virtually no impact on seismic risk of future plants. However, these conclusions are Zion specific, and we believe the proposed new requirements have the potential for increasing seismic safety, reducing public risk, and reducing the variability in risk among future plant sites.

Furthermore, although no analytical value/impact assessment was made for other types of plants, such as BWR plants, Babcock and Wilcox plants, or Combustion Engineering plants, we believe the conclusions reached in this study would not be significantly different for these other types of plants.

Task area 1 would require that single, artificially generated time histories meet not only the current requirements but that they also fall no less than 15% below a Power Spectral Density (PSD) function proposed by the NRC staff. An evaluation of 14 artificial time histories, used by A/E firms and licensees to satisfy the current SRP criteria, indicates that these records generally exceed the NRC staff's PSD requirements at frequency ranges of interest for the design of Nuclear Power Plants (i.e., 20 Hz and less).

Thus, this requirement would, in general, have essentially no impact on future plant construction. However, the proposed PSD criteria provides an analytical method for identifying and evaluating potential nonconservatisms in energy content in artificially generated time histories.

Task area 6 would require that special consideration be given to the responses associated with high frequency modes when the response spectrum method of analysis is used. An evaluation of this requirement indicated that, as a worst case estimate, base shears and base overturning moments in a wall might only be 75% and 90% of their correct values, respectively, in certain isolated cases, if the provision of this proposed new requirement were not met. To assess the impact of this change on seismic risk, the PRA analysis performed on the Zion plant, as part of the SSMRP Phase II study, was used as a base case. The PRA analysis of this plant identified only one wall as having any significant impact on risk, i.e., the auxiliary building shear wall. This wall has a number of pipes and control/power cables penetrating it. As a conservative measure, it was assumed that failure of the wall would result in failure of those systems which are dependent on the penetrations. To assess the potential impact on seismic risk which the proposed new requirement might have, we increased the auxiliary building shear wall median strength by 33% and examined the resultant reduction in total risk at the plant. While the probability of failure of the wall itself decreased by an order of magnitude, there was essentially no change in the total seismic risk. This is due to the fact that containment base-mat uplift (resulting in piping failures between the reactor building and the AFT buildings) and the failure of the service water pump enclosure roof at the top of the crib house (resulting in the loss of AC power diesel generators, due to lack of cooling water) are by far the major contributors to seismic risk at the Zion plant. This does not preclude the possiblility that, at another plant, a shear wall failure might prove to be a much more dominant contributor to seismic risk. Such an evaluation, however, is not possible at this time. Never the less, the possibility exists that this new requirement could have a significant impact on seismic safety, leading to a reduction in public risk.

Task Area 7 deals with proposed new requirements for the design of above-ground, fluid-containing tanks. Specifically, the flexible response of tank walls needs to be considered in developing seismic loads. It is estimated that the increase in computed seismic forces, resulting from the application of this requirement, might be as high as a factor of 2.0 or 2.5. The Zion PRA study was again used as a base case to estimate the potential change in seismic risk resulting from the application of this new requirement. In the Zion study, the only tank which made any significant contribution to risk was the secondary condensate storage tank (SCST), which is part of the power conversion system and auxiliary feedwater system. The median strength of this tank was increased by 150% and the resultant reduction in total plant seismic risk was evaluated. The results indicated that although tank failure probability decreases with the increased strength, the total seismic risk is unaffected. As with the shear wall, this is due to the fact that this tank is an insignificant contributor to risk when compared to other structural failures in the Zion plant. Again, for another plant, this may not be the case. Application of this requirement would lead to increased seismic safety of above-ground vertical tanks, and could, in some cases, lead to substantial reuctions in public risk.

Increased analysis, review, and construction costs associated with the proposed new requirements of task areas 1 and 6 are thought to be small (i.e., less than $5000 each). Changes in computer codes represent a small, one-time cost which would most likely be absorbed by overhead funds. No construction changes are anticipated as a result of these new requirements.

Increased analysis, review and construction costs associated with the proposed new requirements dealing with above-ground, vertical tank design (Task area 7) are small (i.e., less than $5000). This is due to the fact that current industry design and construction practices already incorporate the provisions included in this proposed new requirement. Thus, there would be no impact on tank design and construction for future plants, whether or not this revision to the SRP is adopted.

Although the proposed changes to the SRP relating to the use of multiple time histories are options, and thus did not receive an analytical value/impact assessment, we believe their use would lead to a balanced design in terms of risk, resulting in a more uniform level of risk among plant sites. Furthermore, the use of multiple time histories in the development of floor spectra should lead to reduced loads in piping systems and equipment.

Our study has indicated that virtually no impact on seismic risk of future plants can be expected by the adoption of the proposed new SRP requirements, based on our evaluation of the Zion plant. However, we also believe that there is virtually no cost impact associated with the adoption of these changes. Furthermore, we believe the proposed changes are based on sound engineering principles and experience data, and that they reflect current seismic design practices. We also believe, that these recommended changes have the potential to eliminate nonconservatisms, increase seismic safety, and reduce public risk at future plants. Thus, we strongly recommend that the proposed new recommendations be adopted.

Section 1: Introduction and Background

In October 1981, the Nuclear Regulatory Commission (NRC) approved a reorganization that resulted in the establishment of the Committee to Review Generic Requirements (CRGR). The CRGR has the responsibility to review all proposed new regulatory requirements and recommend approval or disapproval to the Executive Director for Operations. The charter for the CRGR requires that written justification accompany all proposed new regulatory requirements submitted to the CRGR for review. The justification package must include (among other information) an assessment of the risk reduction expected from implementing the proposed requirement and an estimate of the costs to the NRC and the licensees.

The scope of this work is to provide technical support to the Generic Issues Branch (GIB) in preparing value/impact assessments of proposed new regulatory requirements to Standard Review Plan (SRP) Sections 3.7.1, 3.7.2, and 3.7.3. Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 contains the complete text of proposed new regulatory requirements addressed in the value/impact assessments contained herein. The work performed is intended to support written justification prepared by the GIB to accompany regulatory requirements submitted to the CRGR. The major focus of the work deals with the estimates (both quantitative and qualitative) of changes in public risk resulting from the proposed regulatory requirements and major alternatives, and estimates of the industry and NRC resources required to implement the requirements.

A key assumption is that the proposed SRP changes are all "forward fit". That is, if any of the proposed changes are adopted by the NRC they will apply only to new plants. This assumption was established during early discussions among LLNL, GIB and SGEB staff members, and appears to be a reasonable one since analysis results indicate the proposed changes have little effect on risk, but might result in substantial costs if implemented on operating plants.

Many of the proposed changes to SRP Sections 3.7.1, 3.7.2 and 3.7.3 represent alternative procedures or clarifications of existing requirements. Discussions with GIB staff members established that, for these cases, no explicit estimate of changes in risk or cost estimates are required. Where appropriate however, the potential benefits and impact of the proposed change on seismic response and risk is qualitatively discussed and quantitatively assessed (where possible) through the use of engineering judgment experience data, and recent research, but not through the explicit use of seismic PRA calculations.

Estimates of changes in public risk and benefits associated with the adoption of proposed changes are based upon their effect on new plant construction only.

1.1 Basis of Proposed Changes

The proposed changes to SRP sections 3.7.1, 3.7.2 and 3.7.3 are based upon technical work performed by the Lawrence Livermore National Laboratory (LLNL) to assist the NRC in evaluating unresolved safety issues (USI) identified in Task Action Plan A-40. The results of this effort have been summarized in NUREG/CR-1161, [Coats, May 1980]. After detailed review, the NRC staff has accepted most of the work contained in NUREG/CR-1161 as the technical resolution of USI A-40.

The recommendations and results contained in NUREG/CR-1161 were based on an assessment of the current state-of-the-art of seismic engineering at the time the report was developed. This assessment included literature reviews, review of current research work, evaluation of studies performed as part of the TAP A-40 program, and the expert judgment of nationally recognized consultants.

At the time NUREG/CR-1161 was being formulated, no consideration was given as to the value/impact assessment of recommendations contained in that document. The intent was to provide recommendations for changes in SRP seismic design requirements that would reflect the current state-of-the-art of seismic design and provide more realistic estimates of the response of structures, components, and equipment subjected to seismic loadings. Recommendations were based on the philosophy that performance specifications for structures and equipment should be the ultimate goal, not procedural specifications.

Section 2: Proposed SRP Changes and Value/Impact Assessment Approach

A draft copy of proposed changes to SRP sections 3.7.1, 3.7.2, and 3.7.3, reflecting selected recommendations of NUREG/CR-1161 supplemented by others of the NRC staff, was sent to LLNL for review and comment. A total of 24 proposed changes were identified from the marked-up copy of the SRP sections with some of these changes related. Table 2.1 contains a brief summary of our initial review and comments of the 24 proposed changes. Our judgment led us to believe that 10 of these changes would have no impact on PRA results. We felt the remaining 14 (some of which are related) might have an impact on PRA results.

As a result of this initial review, the 14 changes which were flagged as having a potential impact in PRA results, were identified as changes for which LLNL would provide technical assistance to the Generic Issues Branch of the NRC, in preparing value/impact assessments.

Subsequent meetings and conversations with NRC staff members, from the Generic Issues Branch (GIB) and the Structural and Geotechnical Engineering Branch (SGEB), played key roles in the value/impact approach actually taken [Coats, Lappa, 1983].

2.1 Value/Impact Approach

As a result of our meetings and interactions with NRC staff members, the following approach was adopted for performing value/impact assessments of the 14 proposed changes to SRP sections 3.7.1, 3.7.2, and 3.7.3, which were identified as potentially impacting PRA results.

1. Proposed new requirements to the SRP (not alternatives or clarifications) were evaluated using previous analysis experience, results generated from the Seismic Safety Margins Research Program (SSMRP) [Bohn, et.al., 1983], and engineering judgment. These were used to assess structures and equipment affected and the potential magnitude of changes in seismic response resulting from the adoption of the new requirement on the design of future plants. Where appropriate, estimates of response changes were reflected in changes to fragility curves used in the PRA analysis of the Zion Plant [Bohn, et.al., 1983].

A re-analysis of the Zion plant was then made, using the modified fragility curves, to assess the potential change in public risk. Estimates of the industry and NRC resources required to implement the proposed change are based on engineering judgment.

2. Proposed changes to the SRP which represent alternative approaches, but are not requirements, do not require that an estimate of the potential change in public risk be made. For the sake of completeness however, the potential benefits and impact on seismic response and risk, of the proposed alternative procedures, is qualitatively discussed and quantitatively assessed (where possible) through the use of engineering judgment but not through the explicit use of seismic PRA calculations.

3. Proposed changes to the SRP which are editorial in nature or which are considered simple clarifications of existing requirements have been identified as such in subsequent sections of this report. No value/impact assessment of these items is required and none is made. The determination of which changes are editorial in nature or clarifications of existing requirements, was made in conjunction with NRC staff members during formal and informal conversations and meetings (see letter to S.K. Shaukat dated May 6, 1983 from Coats and Lappa).

The analytical value/impact results contained in this study are specific to the Zion nuclear power plant. This is necessarily so since value/impact assessments require the comparison of changes in core melt probability and Man-REM release quantities, and cost estimates. To obtain these values, a comprehensive probabilistic risk assessment analysis is required. Only a few such comprehensive analyses have been performed for nuclear power plants and of these, even fewer have the flexibility to easily allow for sensitivity studies to determine changes in risk resulting from changes in component fragilities. The analysis of the Zion plant, performed as part of the Seismic Safety Margins Research Program at LLNL is one such analysis. LLNL's Zion analysis does allow for sensitivity studies, and as such, this plant was used as the base case to evaluate proposed changes to the Standard Review Plan.

The Zion plant is a pressurized water reactor (PWR) located on Lake Michigan just east of the town of Zion, Illinois, and about 40 miles north of Chicago. This plant was chosen for the SSMRP, PRA assessment on the basis of being reasonably typical (in terms of power, systems design, and site conditions) of pressurized water reactors in the 1960's era. No attempt to analytically evaluate the value/impact of proposed SRP changes has been made for other types of plants such as BWR plants, Babcock & Wilcox plants, or Combustion Engineering plants. However, it is the general opinion of the LLNL staff and our consultants that the conclusions reached in this study would not be significantly different for these other plants. This is based on our knowledge of the functional and structural design of these plants and on the limited ways that the proposed SRP changes would impact these designs.

Since the analytical (i.e., PRA) value/impact assessments contained in this report are based on SSMRP, Zion Nuclear Power Plant PRA results, a brief overview of the SSMRP is given below.

2.2 Overview of the Seismic Safety Margins Research Program

To assist the NRC in its seismic licensing and re-evaluation role, the research arm of the NRC sponsored the Seismic Safety Margins Research Program (SSMRP), at LLNL with the goal of developing tools and data bases to evaluate the risk of earthquake caused damage and subsequent radioactive release from a commercial nuclear power plant. This program began late in 1978, and the methodology was finalized in 1982 [Smith, et.al., 1981, 1982].

Table 2.1 Summary of Proposed Changes to SRP Sections 3.7.1, 3.7.2, and 3.7.3

NUMBER	SRP SECTION	SRP TOPIC	CHANGE IN REQUIREMENTS	IMPACT IN PRA**
	3.7.1			
1	3.7.1.I.1.a	Design response spectra	Editorial	None. Editorial change.
2	3.7.1.I.1.b	Design time history	Option to use multiple time histories is given	Review not required since change represents an option.
3	3.7.1.I.4	SGEB coordination of other branches'	Response spectra at foundation level reviewed	None. Editorial change since 3.7.1.II.1.b already requires this.
4	3.7.1.II.1.a	Design response spectra	Design response spectra should meet or exceed amplitudes of site specific spectra at all frequencies.	None. Requirement already exists.
5	3.7.1.II.1.a	Design response spectra	Editorial. Reference to 2.5.2	None. Editorial change.
6*	3.7.1.II.1.b	Design time history	Justification of single time history. Use of multiple time histories given as option.	Might impact PRA.
7	3.7.1.II.2	Percentage of critical damping values	Higher damping values may be used if justified.	No value impact assessment required since SRP change is an option.

* Indicates topics that might have an impact in PRA.

** Indicates LLNL's initial evaluation of potential impact in PRA. Does not necessarily reflect findings of this study.

NUMBER	SRP SECTION	SRP TOPIC	CHANGE IN REQUIREMENTS	IMPACT IN PRA**
8*	3.7.1.II.1	Percentage of critical damping values	Notification that compliance with stress provision in R.G. 1.61 will be reviewed.	Might impact PRA.
9	3.7.1.III.1	Design ground motion	Editorial	None. Editorial changes.
10	3.7.1.IV	Evaluation findings	Evaluation findings to be modified if option to use multiple time histories is used.	None. Changes relate to SRP option.
	3.7.2			
11*	3.7.2.I.4	Soil-structure interaction	Uncertainties must be "recognized".	Might impact PRA.
12	3.7.2.I.5	Development of floor response spectra	Various new methods allowed.	None. New methods are optional.
13*	3.7.2.II.1(4)	Dynamic analysis method	Acceptance criteria for adequacy of number of degrees of freedom modified.	Might impact PRA.
14*	3.7.2.II.1(5)	Dynamic analysis method	Demonstration required to show high frequency effects are included	Might impact PRA.

* Indicates topics that might have an impact in PRA.

** Indicates LLNL's initial evaluation of potehtial impact in PRA. Does not necessarily reflect findings of this study.

NUMBER	SRP SECTION	SRP TOPIC	CHANGE IN REQUIREMENTS	IMPACT IN PRA**
15*	3.7.2.II.4	Soil-structure interaction	Uncertainties to be "addressed" are listed.	Might impact PRA.
16*	3.7.2.II.5	Development of floor response spectra	Single time history use to be justified. Use of multiple histories reviewed on case-by-case basis. Direct generation methods reviewed.	Might impact PRA
17*	3.7.2.II.7	Combination of modal responses	Acceptance criteria for consideration of high frequency modes given in new Appendix.	Might impact PRA.
18*	3.7.2.II.9	Effects of parameter variations on floor response spectra.	Acceptance criteria for parameter variations referred back to SRP 3.7.2.II.5	Might impact PRA.
19	3.7.2.IV	Evaluation findings	Editorial change to include Category I above-ground tanks.	None. Editorial change.
	3.7.3			
20*	3.7.3.I.14	Methods for seismic analysis of above-ground tanks.	New topic. Fluid dynamics and tank flexibility included.	Might impact PRA.
21*	3.7.3.II.12(1)	Category I buried piping, conduits, and tunnels.	Specifically states the kinds of ground-shaking induced loadings to be considered.	Might impact PRA.

* Indicates topics that might have an impact in PRA.
** Indicates LLNL's initial evaluation of potential impact in PRA. Does not necessarily reflect findings of this study.

NUMBER	SRP SECTION	SRP TOPIC	CHANGE IN REQUIREMENTS	IMPACT IN PRA**
22*	3.7.3.II.12(3)	Category I buried piping, conduits, and tunnels.	Specifically states the kinds of seismic-induced loadings to be considered.	Might impact PRA.
23*	3.7.3.II.14	Methods for seismic analysis of above-ground tanks.	New topic. Fluid dynamics and tank flexibility must be included. Housner method not allowed in some cases.	Might impact PRA.
24*	3.7.3.III.14	Methods for seismic analysis of above-ground tanks.	New topic. Methods of seismic analysis are reviewed.	Might impact PRA.

* Indicates topics that might have an impact in PRA.
** Indicates LLNL's initial evaluation of potential impact in PRA. Does not necessarily reflect findings of this study.

In the past, it was generally believed that earthquake induced ground accelerations in excess of 1.0g were not possible. However, records obtained from the 1980 El Centro earthquake indicated ground accelerations up to 1.7g. Today, many experts believe that earthquake induced ground accelerations of even larger values may be recorded in the future.

Nuclear power plant facilities have not and are not being designed for accelerations this large. Thus, when evaluating the risk of seismically induced damage to a plant, it is necessary to allow for the possibility of earthquakes of all possible sizes, and then to recognize that smaller earthquakes occur more frequently than large ones.

A second important aspect of particular interest for nuclear power plants is that, during an earthquake, all parts of the plant are excited simultaneously. This means that the redundancy of safety systems could be compromised. For example, in order to force emergency core cooling water into the reactor core, following a pipe break or leak, certain valves must open. To ensure reliability, two valves are located in parallel so that, should one fail to open, the second valve would provide the necessary flow path. Since valve failure due to random causes (corrosion, electrical defect, etc.) is an unlikely event, the provision of two valves provides a high degree of reliability. However, during an earthquake, both valves would be shaken simultaneously. Thus, there is a high likelihood that both valves would be damaged. Hence, the planned-for redundancy would be compromised. This "common-cause" failure possibility represents the single most significant aspect of potential risk to nuclear power plants during earthquakes. One feature of the SSMRP methodology is that, for the first time, all such earthquake-induced, common-cause failures are being explicitly considered. In fact, a general purpose statistical analysis computer code (SEISIM) was written just to consider such common-cause failures, and this code constitutes a significant advance in the state-of-the-art in risk assessment methodology tools.

The SSMRP was begun in 1978 when it became evident that an accurate seismic risk analysis must simultaneously consider all the interrelated factors that determine the probabilty of radioactive release and exposure to the public. In the traditional design procedure, by contrast, each factor is usually analyzed separately. These closely coupled factors are:

- The likelihood and magnitude of an earthquake.
- The transfer of earthquake energy from a fault source to the power plant, a phenomenon that varies greatly with the magnitude of an earthquake.
- Interaction between soil and structures during an earthquake, a phenomenon that depends on the soil properties at the site and the location of the fault source relative to the plant.
- Coupled responses between the power plant's buildings and the massive reactor vessels, piping systems, and emergency safety systems within.
- Numerous accident scenarios, which vary according to the types of failures assumed and the success or failure of the engineered safety features intended to mitigate the consequences of an accident.

A nuclear power plant is designed to ensure the survival of all buildings and emergency safety systems in a worst-case ("safe shutdown") earthquake. The assumptions underlying this design process are deterministic. In practice, however, these assumptions are subject to uncertainty. It is not possible, for example, to accurately predict the worst earthquake that will occur at a given site. Dynamic characteristics of soil, structures, and subsystems vary significantly. To model and analyze the coupled phenomena that contribute to the total risk of radioactive release, it is therefore necessary to consider all significant sources of uncertainty, as well as all, significant interactions. Total risk is then obtained by considering the entire spectrum of possible earthquakes and integrating their calculated consequences. This point underscores another vitally important feature of the SSMRP: the nuclear power plant is examined in its entirety, as a system.

There are five steps in the SSMRP methodology for calculating the seismic risk at a nuclear power plant:

1. Determine the local earthquake hazard at the site.
2. Determine seismic response of structures and components.
3. Determine structure and component failure modes.
4. Construct plant logic models (event trees and fault trees).
5. Compute the probability of radioactive release using the information from Steps 1 through 4.

An in-depth discussion of each of these steps can be found in the 10 volume SSMRP report [Smith, et.al., 1981, 1982].

Results of SSMRP Zion Risk Analysis

In the analysis of Zion, a number of structural failure modes were identified. (These were localized failure of certain walls or roof slabs rather than collapse of the structures.) These localized structural failures were examined, and it was found that two of them had the potential of causing significant common-cause failures of the plant safety systems. In the results presented below, these structural failures were assumed to have their most serious hypothesized consequences:

(i) The failure of the roof of the service water pump enclosure room (at the top of the crib house) is assumed to fail all six service water pumps beneath it. This results in loss of the emergency AC power diesel generators, due to lack of cooling water.

(ii) The failure of the wall between the turbine building and the auxiliary building is assumed to cause loss of all electrical wiring and control air conduits, so both power and control to the reactor building are lost.

(iii) Soil failure under the toe of the containment is an identified failure mode, and this was assumed to result in sufficiently large rocking motions so as to fail the safety injection system (SIS), the charging system (CHG), and residual heat removal (RHR) piping between the Auxiliary-Fuel-Turbine (AFT) complex and the reactor building.

Probability of Radioactive Release

The median frequency of radioactive release was computed to be 3.6×10^{-6} per year. This value reflects inherent randomness in all the input variables and the hazard curve, as well as modeling uncertainties in all the input variables due to lack of exact knowledge of their mean values. The 10-90% confidence band on the release frequency was found to be about 3 orders of magnitude. This was due primarily to uncertainties associated with the seismic hazard model. The median values and confidence bounds were obtained by making repeated calculations of the release frequencies, while varying the median values of all input variables according to an experimental design.

The release frequencies at earthquake levels 2, 3, and 4 were the dominant contributors both for probability of release and dose. The probabilities of both release and dose have only minor contributions from earthquake levels 1, 5 and 6. This indicates that the bulk of the risk is at the intermediate earthquake levels (2-4 SSE), and that the range of peak ground accelerations considered was adequate. Risk, in this context, refers to the probability of core-melt. For a complete description of the SSMRP Zion risk analysis results, the reader is referred to the SSMRP Phase II summary report [Bohn, et.al., 1983].

In summary, in computing the probability of radioactive release, the dominant contributors are soil failure and uplift of the containment basemat and failure of the service water system due to failure of the pump enclosure roof. To a smaller extent, there are contributions from failure of interconnecting pipes due to differential motion between the reactor building and the auxiliary building. If uplift occurs, but the interconnecting pipes are not damaged, and if the pump enclosure roof fails, but the service water system still functions, then the risk decreases by a factor of five. Thus, the assumptions as to the effects of the structural failures play an important role in determining the risk at the plant.

The SSMRP methodology has been, and will be, used (this study for instance) to provide insights to NRC decision makers on generic safety issues. It also has the potential for providing the nuclear industry with a tool to justify the economics (or lack thereof) of retrofitted design changes for operating reactors, and new requirements for future plant construction. Thus, the seismic risk assessment methodology developed in the SSMRP represents a significant advance in the state-of-the-art of probabilistic risk assessment, which can have far-reaching implications in assessing, standardizing and improving the safety of nuclear power plants in the United States.

Section 3: Summary of Value/Impact Assessments of Proposed SRP Changes

In this section, a brief description of each proposed SRP change is given, along with a detailed evaluation of the value/impact associated with the change. Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 contains the complete text of the proposed SRP changes outlined below. Related changes have been grouped together and treated in a single value/impact assessment.

3.1 Design Time History

This section deals with proposed changes to SRP section 3.7.1.II.1.6 - Design Time History. The proposed changes deal with the use and acceptance of single artificial time histories and the optional use of multiple time histories, both real and artificial.

3.1.1 Summary of NRC Proposed SRP Changes*

#1. The use of a single artificial time history is to be justified through demonstration of sufficient energy in the frequency range of interest. This demonstration will be accomplished through the generation of power spectrum density (PSD) functions, which the NRC staff will review using an acceptance criteria established by the NRC staff [Shinozuka, 1983].

#2. The use of multiple time histories, artificial or recorded, is reviewed on a case-by-case basis. Items of interest are:

- number of time histories
- frequency content
- amplitude
- energy content
- duration
- criteria for selection of time histories

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

3.1.2 Technical Discussion of Issues

NUREG/CR-1161 [Coats, 1980] expressed concern regarding the use of artificially generated or synthetic time histories in the seismic analysis process. The SRP enveloping criteria for artificially generated time histories does not lead to their unique specification. Smith, et.al. [1977], studied a series of synthetic time histories generated to envelop R.G. 1.60 design ground response spectra and satisfy the SRP criteria. This series of 16 horizontal and 12 vertical motions were obtained from eleven firms in the nuclear industry. Variability in frequency content was observed and described by coefficients of variation of 0.2 - 0.3 for spectral accelerations at 5% damping. Analyses performed with identical models and procedures will lead to a variability in response due to the time histories. Hence, the current criteria does not lead to a unique level of design, which in turn leads to varying levels of conservatism (and risk) among plants.

A second concern discussed in NUREG/CR-1161 was the possibility the resulting artificial time histories were deficient in energy content at significant frequencies of the system to be analyzed. As stated in NUREG/CR-1161, this is potentially most significant for the analysis of nonlinear systems whose response is more dependent on the specific characteristics of the excitation. NUREG/CR-1161 recommends only recorded time histories be used when performing nonlinear analysis.

To alleviate these concerns, two modifications to the SRP are proposed.

Before discussing these changes, let us place the issue in some perspective:

- The design ground response spectra of R.G. 1.60 define the seismic input phase of the seismic design criteria. Excitations "greater than" R. G. 1.60 exceed this requirement and, ostensibly, introduce conservatism. The existing criterion that artificially generated time histories have response spectra which envelop R.G. 1.60 design ground response spectra adds conservatism of an unquantified amount.
- The bases for R.G. 1.60 design response spectra were response spectra of recorded motions. The design spectra were targeted to approximately the 84% nonexceedance probability of the recorded spectra. Artificially generated time histories whose response spectra envelop R.G. 1.60 achieve a nonexceedance probability greater than 84% at frequencies where the spectra is greater.

- The impact of the conservatism or nonconservatism of the artificially generated time histories is through those response components for which time history analysis procedures are used. This typically includes in-structure response spectra, which are the basis for the design of piping systems and the qualification of components. In addition, it may affect force quantities of structures determined from the second stage of a two stage analysis procedure.

Recognizing the potential for added conservatism discussed above and the extent to which artificially generated time histories will impact design quantities, let us consider the potential deficiencies in the current criteria and the proposed modifications. The two potential deficiencies noted earlier are variability in the artificial time histories and lack of sufficient energy in specific frequency ranges which may be of consequence to the analysis. Two modifications to the SRP are proposed to address these issues:

1.) Single artificial time history

Use of a single artificial time history is permitted if its response spectra satisfies current SRP enveloping criteria and its power spectral density (PSD) satisfies the acceptance criteria of the NRC staff. The term "single artificial time history" here refers to a single set of three components of motion (two horizontal and the vertical) to be used in the seismic analysis, i.e. a single earthquake simulation. The additional PSD criterion is intended to reduce variability in artificially generated time histories and to ensure adequate energy content over all frequency ranges of interest.

To investigate the potential impact of this proposed change, PSD's were generated for fourteen of the sixteen artificial horizontal acceleration time histories used in Smith's study and were compared with the acceptance criteria of the NRC staff. Two of the 16 records were discarded, since their fit to R.G. 1.60 did not meet existing criteria. Figures 3.1 - 3.14 display the results and also a comparison of the response spectra for each of the 14 time histories with the target R.G. 1.60 spectra. Examining the figures, one can make the following observations:

- In general, the PSD's of the artificial time histories exceed the criteria over the amplified frequency range of interest for typical nuclear power plant structures (1 to 10 Hz).

- In general, the PSD's of the artificial time histories exceed the criteria in the 10 to 20 Hz range also; although, some exceptions are apparent
- For frequencies above 20 Hz, the PSD's of the artificial time histories generally oscillate about the target, with many points below. This frequency range is the least important to response of typical nuclear power plant structures since high frequency ground motions are filtered by travel distance and SSI effects.
- The magnitude of the exceedances from the 1 to 10 Hz frequency range vary up to factors of 100 and greater.
- It would appear that the concern expressed in NUREG/CR-1161, regarding a deficiency in energy content at significant frequencies of the system, is not supported by the data-set of time histories evaluated. However, the generation of PSD's provides an analytical method for identifying and evaluating potential nonconservatisms in energy content of artificially generated time histories.

Figures 3.1-3.14 are smoothed versions of the PSD functions. The smoothing is performed by means of the moving average method involving two successive frequency points (ω_i and ω_{i+1}) with the average values plotted at ω_i. Futhermore, these PSD's have been calculated using the strong-motion duration of the artificial history rather than the entire duration of the record.

2.) Multiple time histories

The use of multiple time histories, artificial or recorded, is to be permitted as an option and reviewed on a case-by-case basis. The motivations to perform multiple analyses are:

- When multiple analyses are performed in a best estimate format, one can explicitly account for the recognized variability in definition of the seismic input and in the system characteristics (properties of the soil, structures, piping systems, equipment, etc.) in a probabilistic fashion. A design goal is then established (e.g., an 84% nonexceedance probability), and design quantities (in-structure force quantities, response spectra, etc.) are obtained. The degree of conservatism due to the response calculational process is, hence, quantified. This leads to a more balanced design, in particular, for subsystems whose input environment is defined by in-structure response spectra; their values being smoother and broader than spectra obtained from conventional single time history analyses. The use of a best estimate format refers to the use of realistic modeling parameters; ground motion input; and analysis techniques; in which factors of conservatism, incorporated in standard design approaches, have been eliminated. A comparison of this approach with a typical design approach is given in Table 3.1.

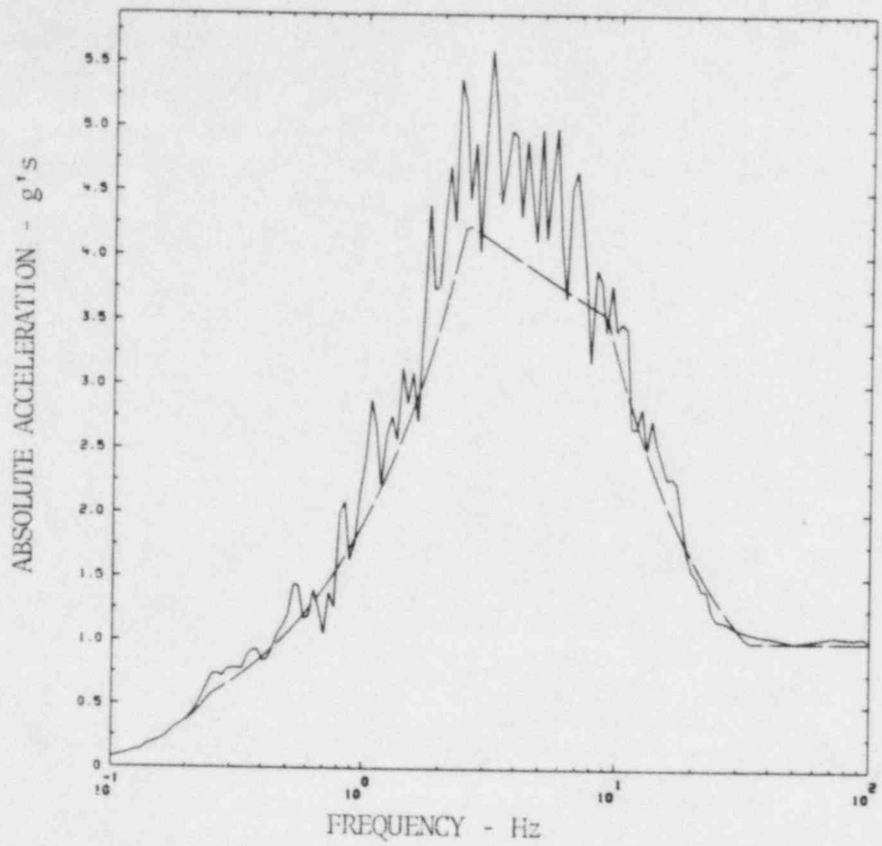

Fig. 3.1a - R.G. 1.60 Horizontal Time History 1
2% Response Spectrum vs R.G. 1.60

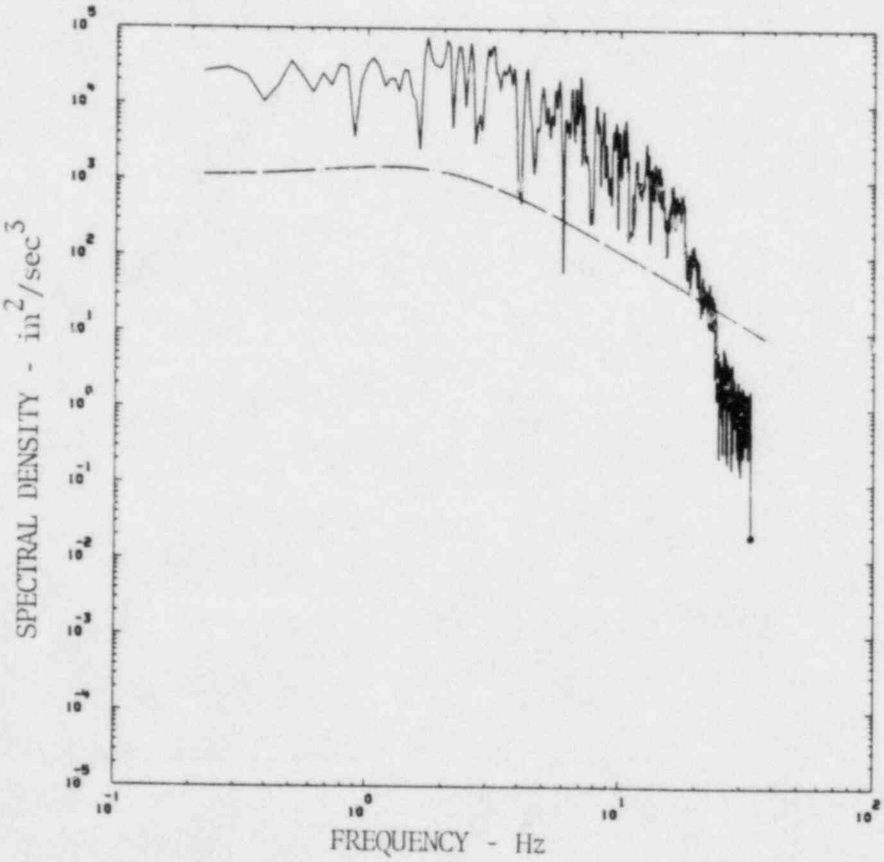

Fig. 3.1b - R.G. 1.60 Horizontal Time History 1
PSD vs NRC Proposed Criteria

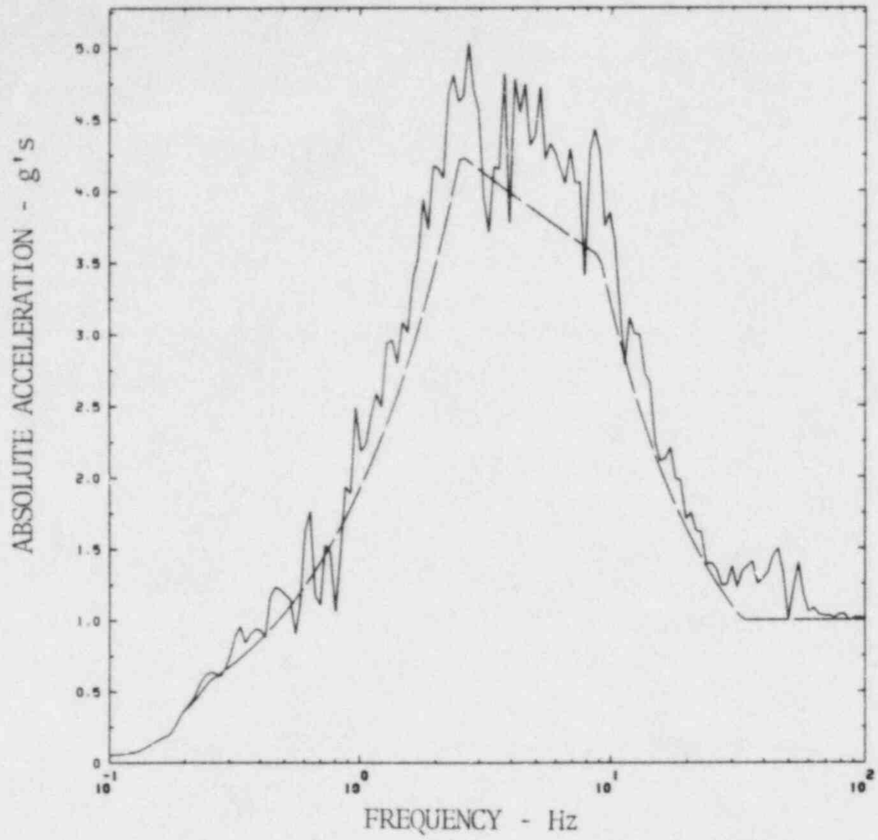

Fig. 3.2a - R.G. 1.60 Horizontal Time History 2
2% Response Spectrum vs R.G. 1.60

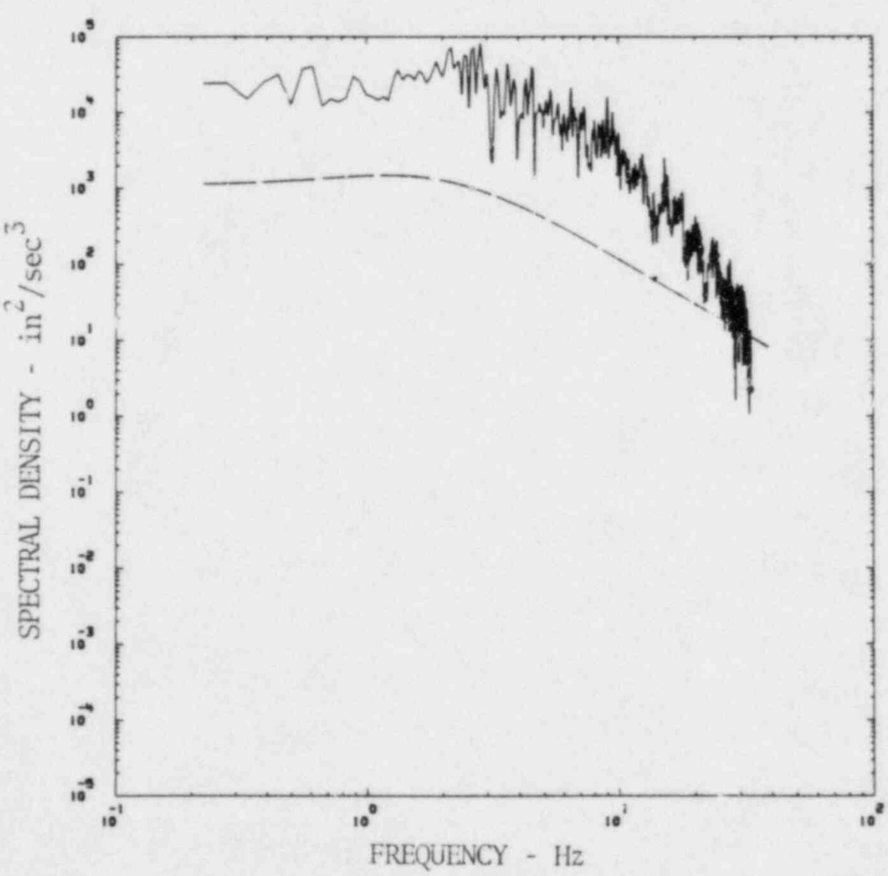

Fig. 3.2b - R.G. 1.60 Horizontal Time History 2
PSD .s NRC Proposed Criteria

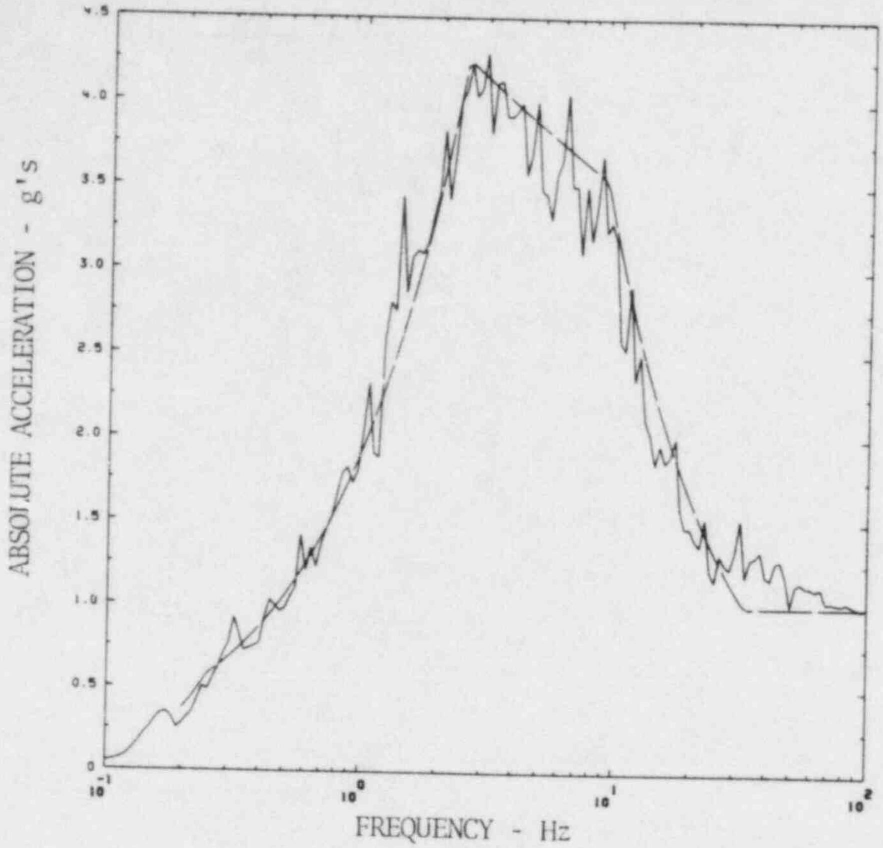

Fig. 3.3a - R.G. 1.60 Horizontal Time History 3
2% Response Spectrum vs R.G. 1.60

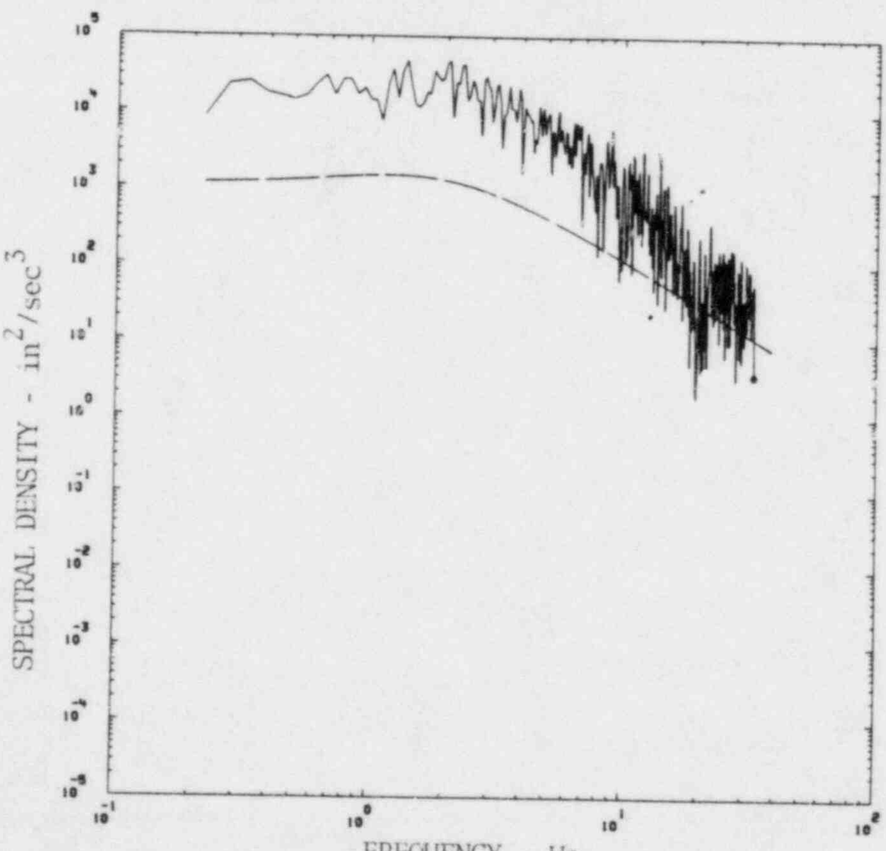

Fig. 3.3b - R.G. 1.60 Horizontal Time History 3
PSD vs NRC Proposed Criteria

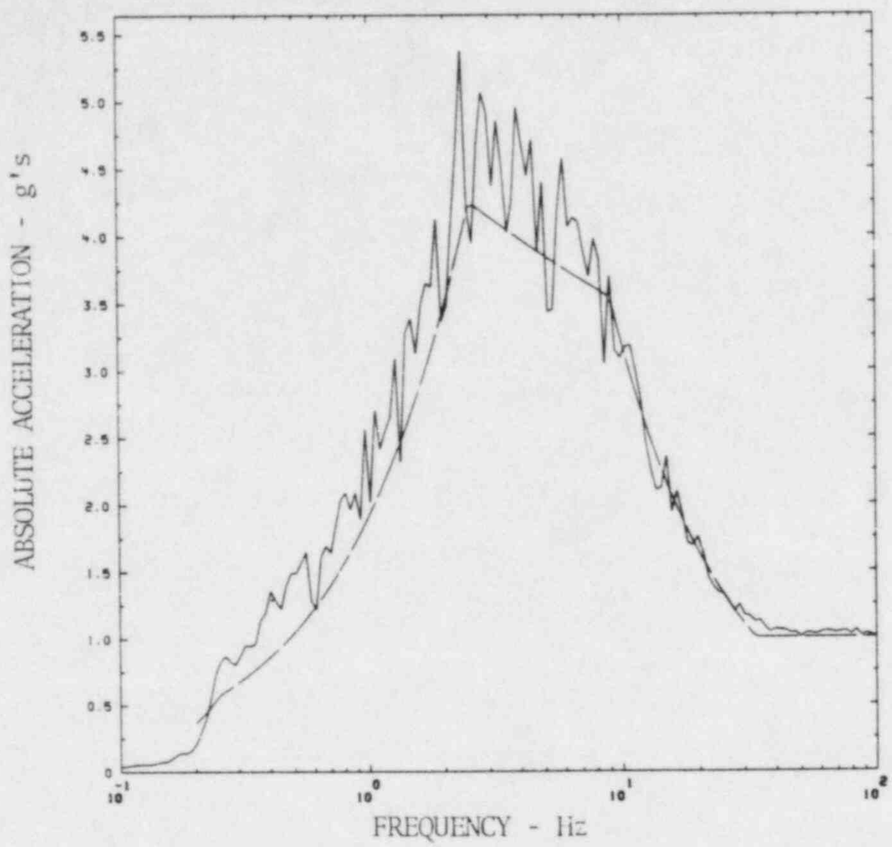

Fig. 3.4a - R.G. 1.60 Horizontal Time History 4
2% Response Spectrum vs R.G. 1.60

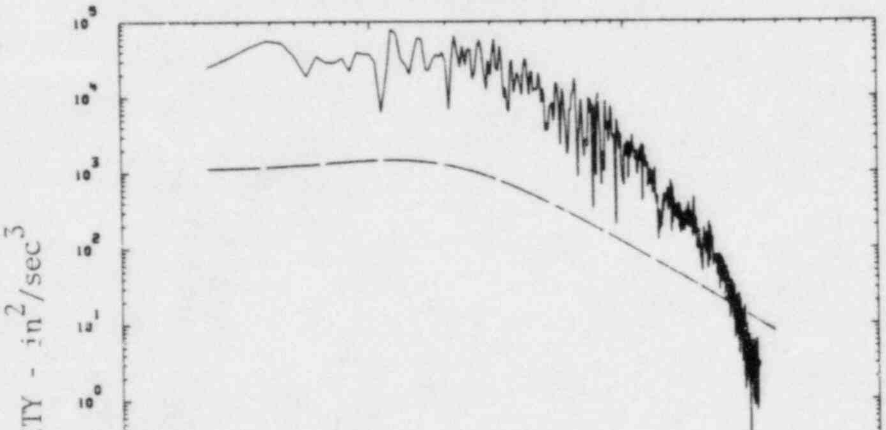

Fig. 3.4b - R.G. 1.60 Horizontal Time History 4
PSD vs NRC Proposed Criteria

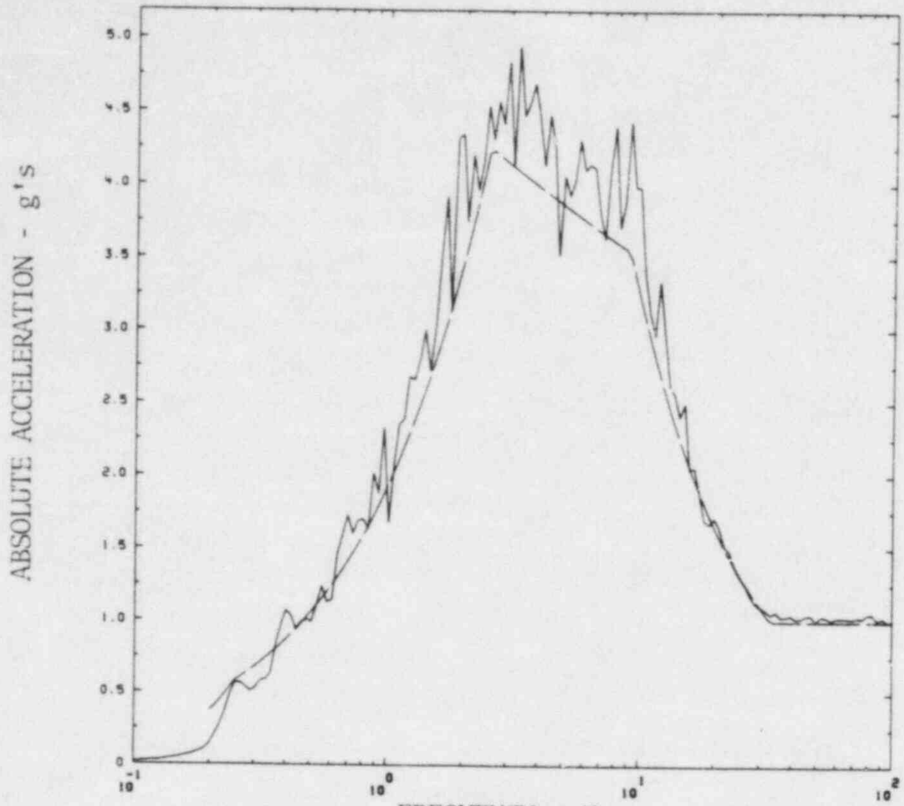

Fig. 3.5a - R.G. 1.60 Horizontal Time History 5
2% Response Spectrum vs R.G. 1.60

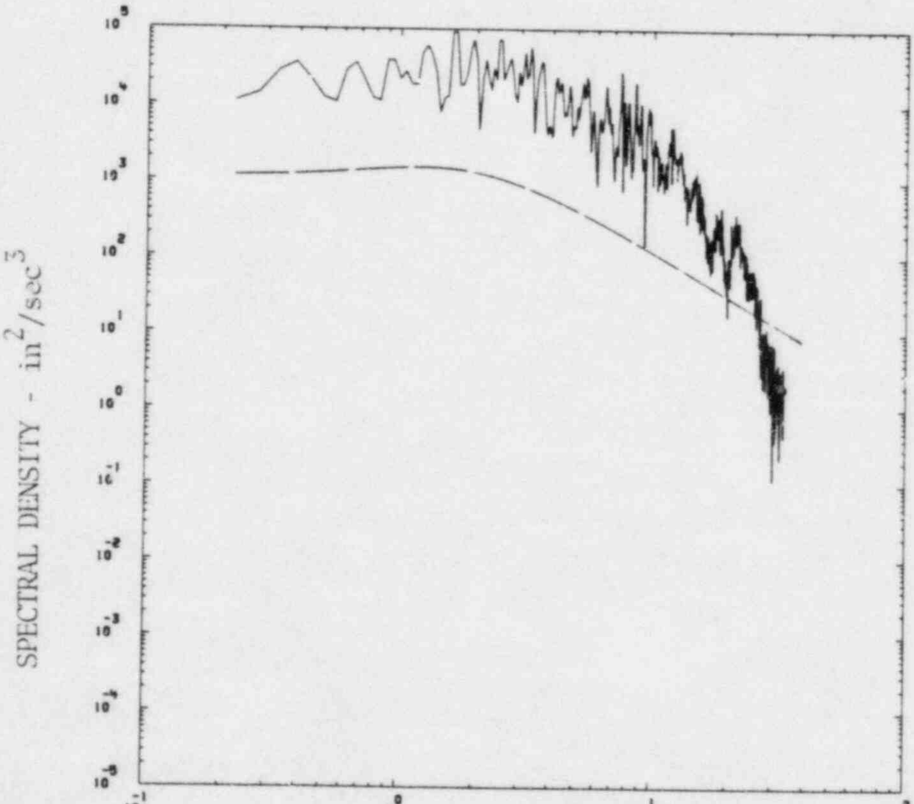

Fig. 3.5b - R.G. 1.60 Horizontal Time History 5
PSD vs NRC Proposed Criteria

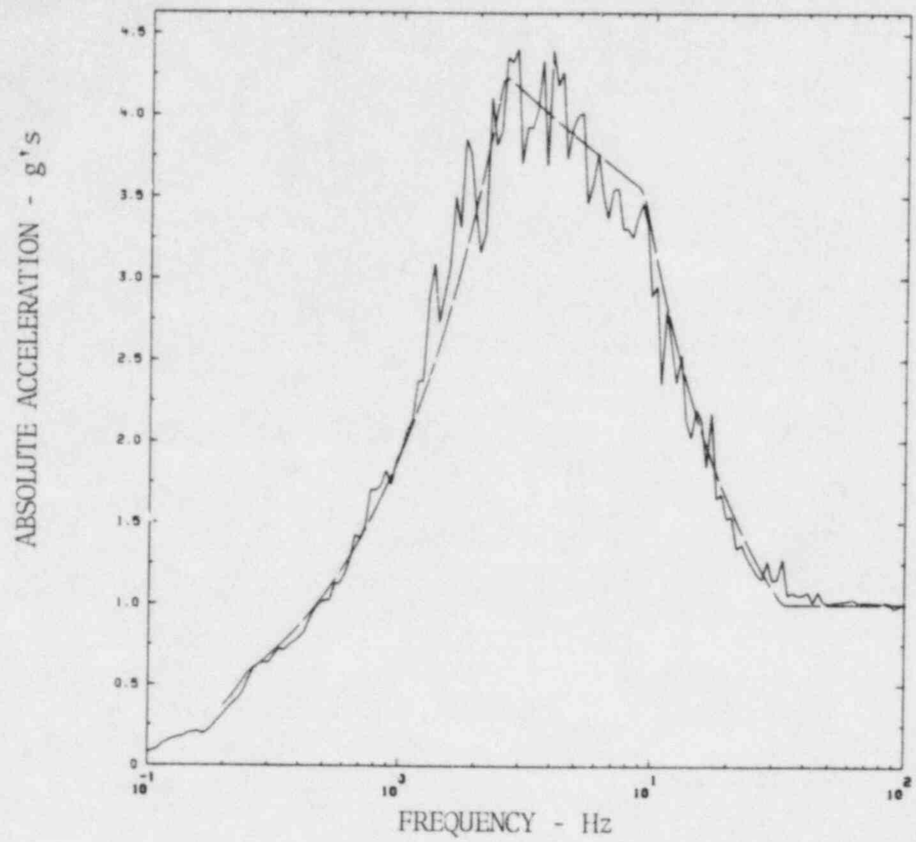

Fig. 3.6a - R.G. 1.60 Horizontal Time History 6
2% Response Spectrum vs R.G. 1.60

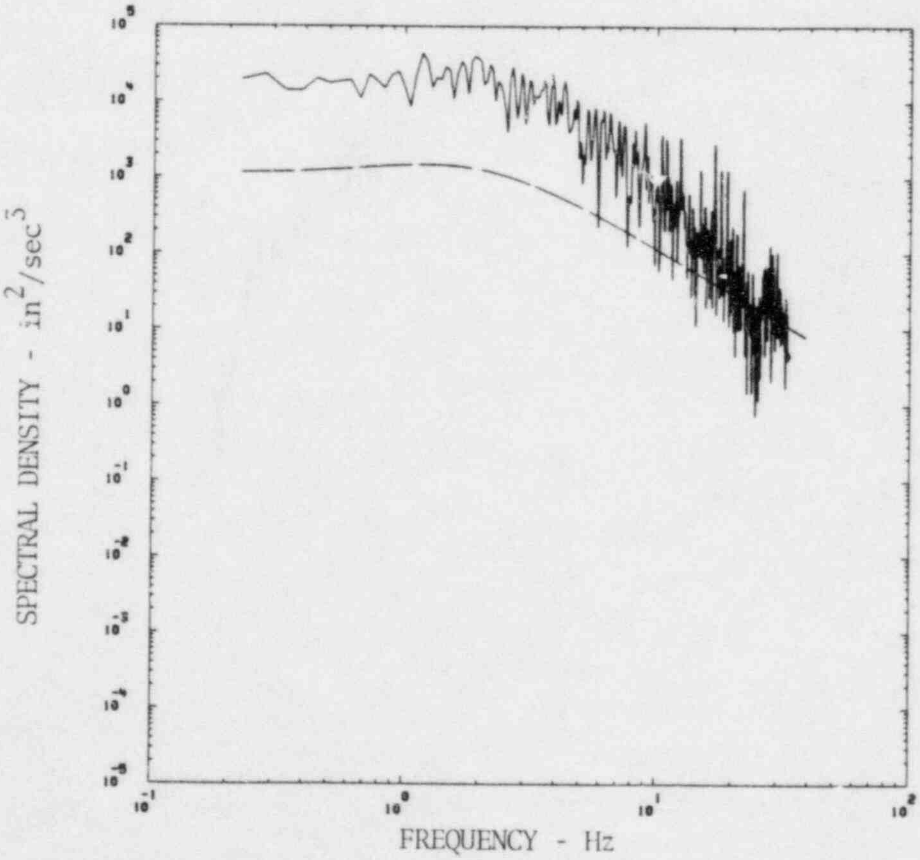

Fig. 3.6b - R.G. 1.60 Horizontal Time History 6
PSD vs NRC Proposed Criteria

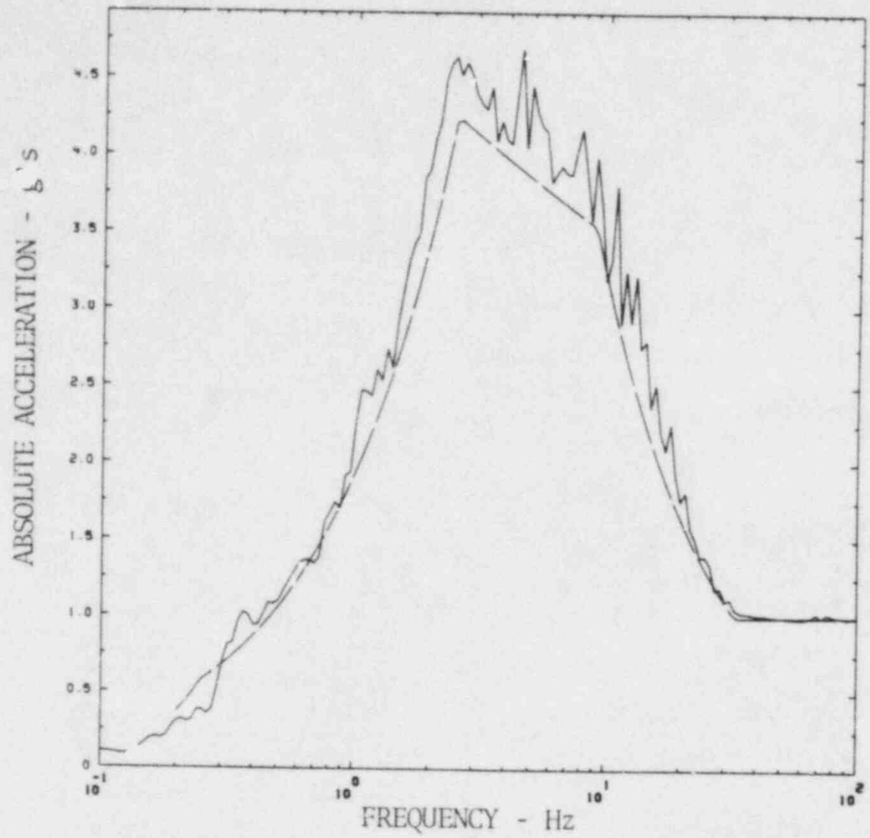

Fig. 3.7a - R.G. 1.60 Horizontal Time History 7
2% Response Spectrum vs R.G. 1.60

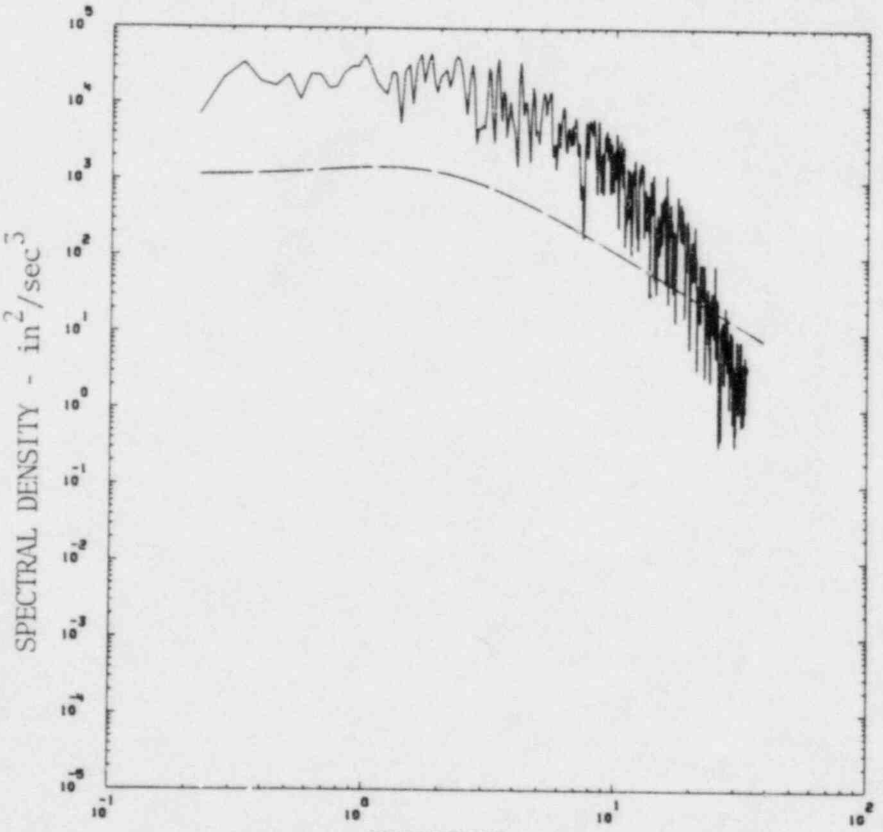

Fig. 3.7b - R.G. 1.60 Horizontal Time History 7
PSD vs NRC Proposed Criteria

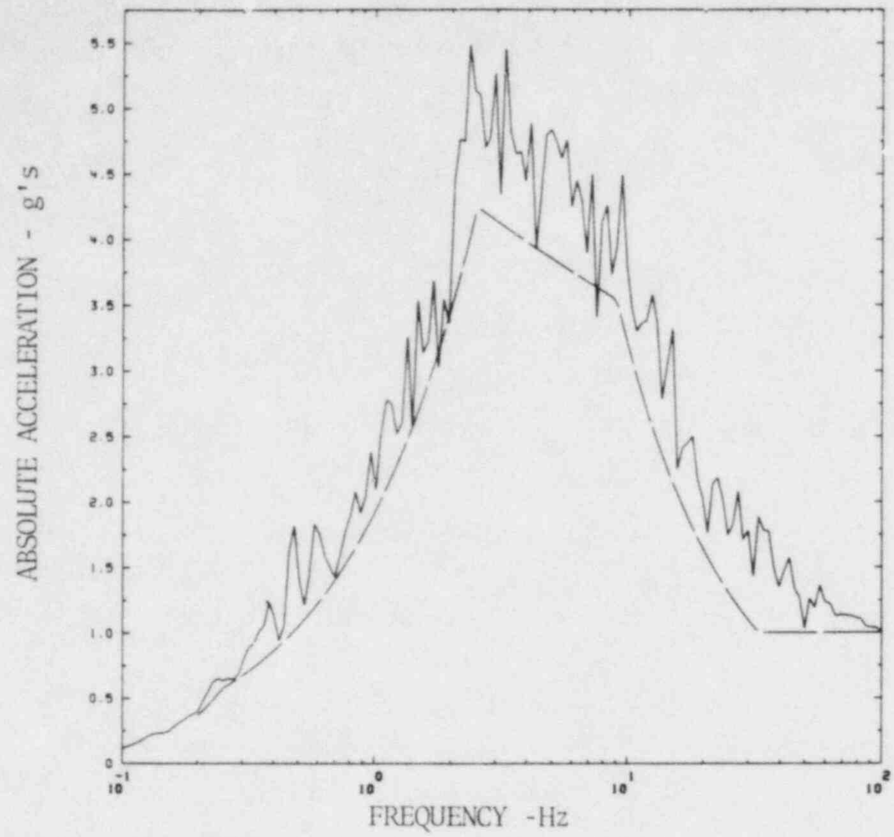

Fig. 3.8a - R.G. 1.60 Horizontal Time History 8
2% Response Spectrum vs R.G. 1.60

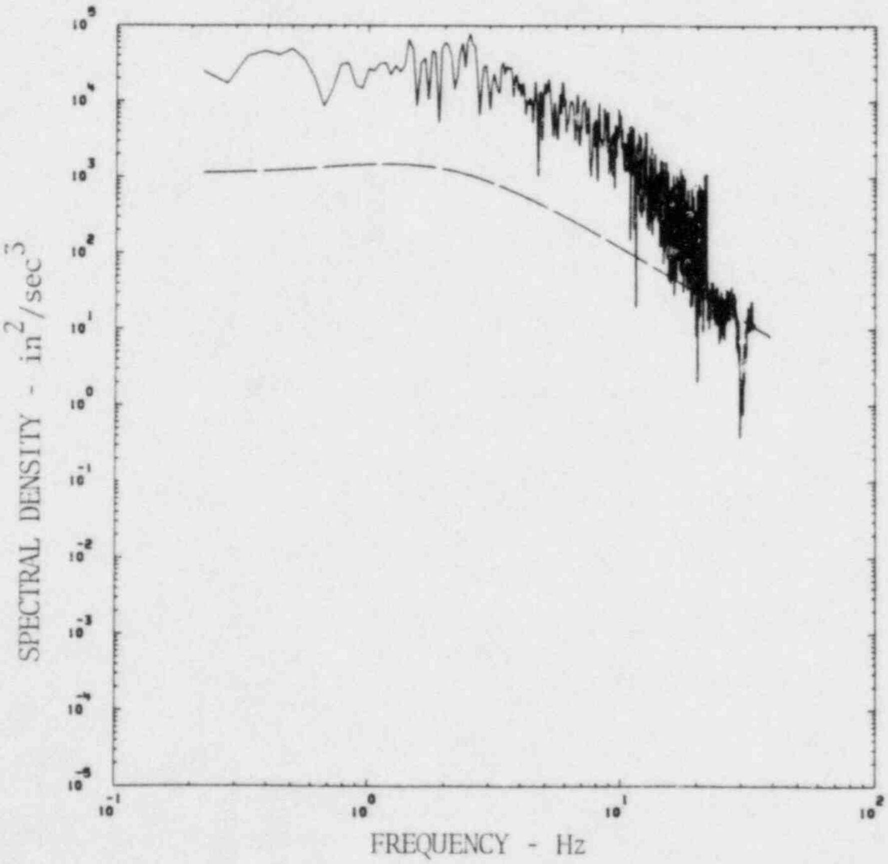

Fig. 3.8b - R.G. 1.60 Horizontal Time History 8
PSD vs NRC Proposed Criteria

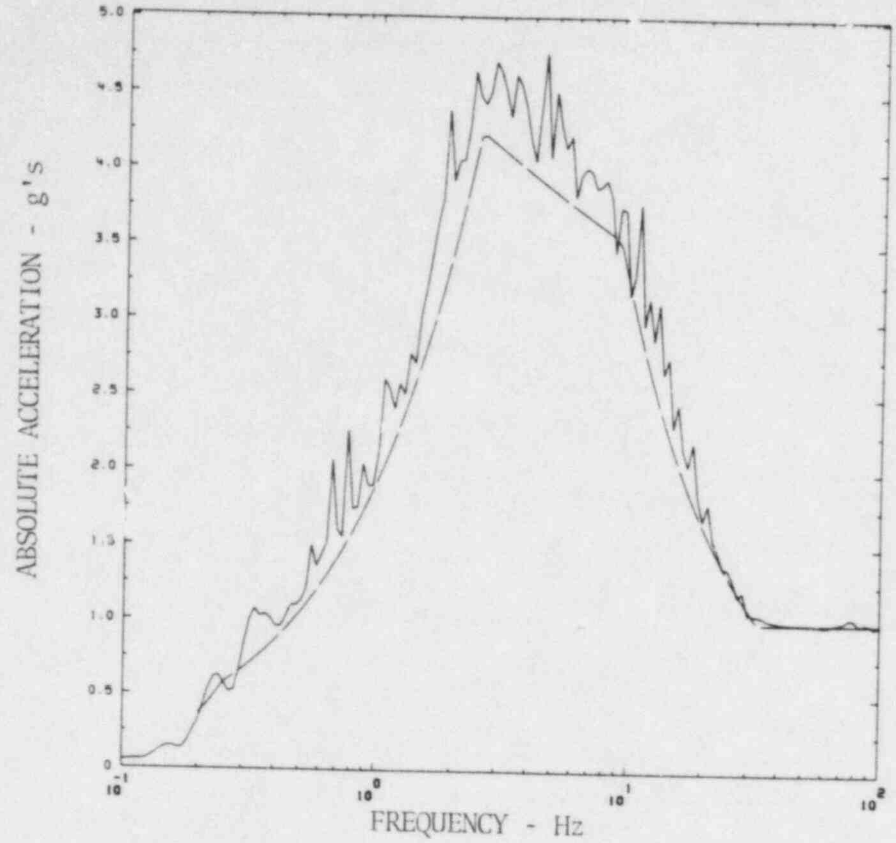

Fig. 3.9a - R.G. 1.60 Horizontal Time History 9
2% Response Spectrum vs R.G. 1.60

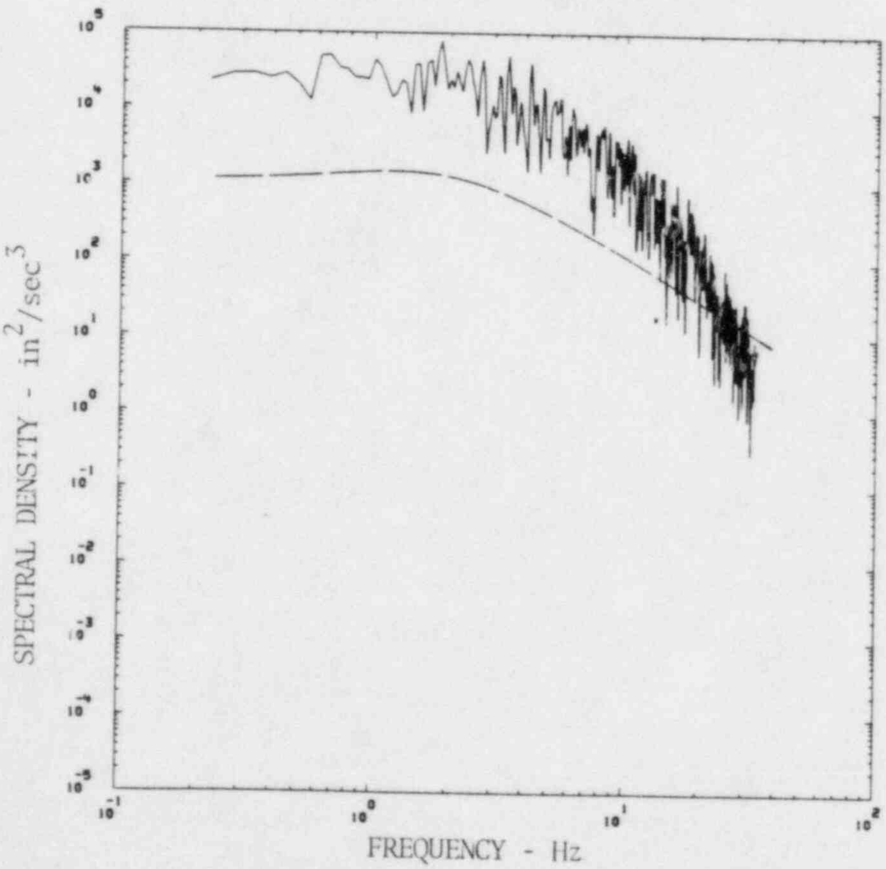

Fig. 3.9b - R.G. 1.60 Horizontal Time History 9
PSD vs NRC Proposed Criteria

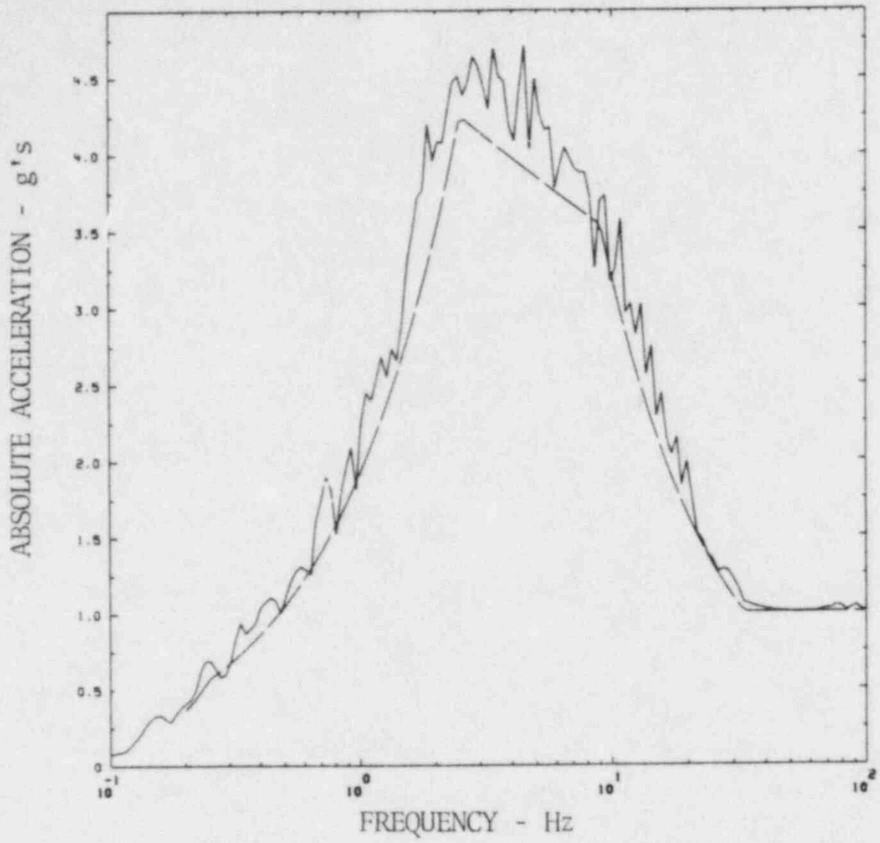

Fig. 3.10a- R.G. 1.60 Horizontal Time History 10
2% Response Spectrum vs R.G. 1.60

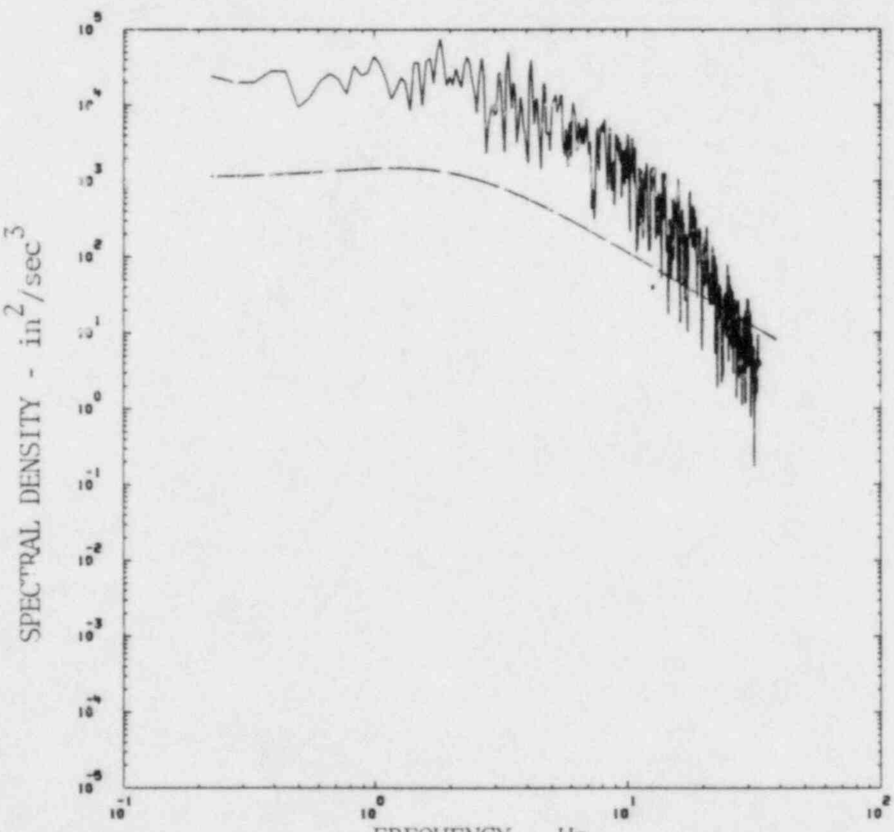

Fig. 3.10b - R.G. 1.60 Horizontal Time History 10
PSD vs NRC Proposed Criteria

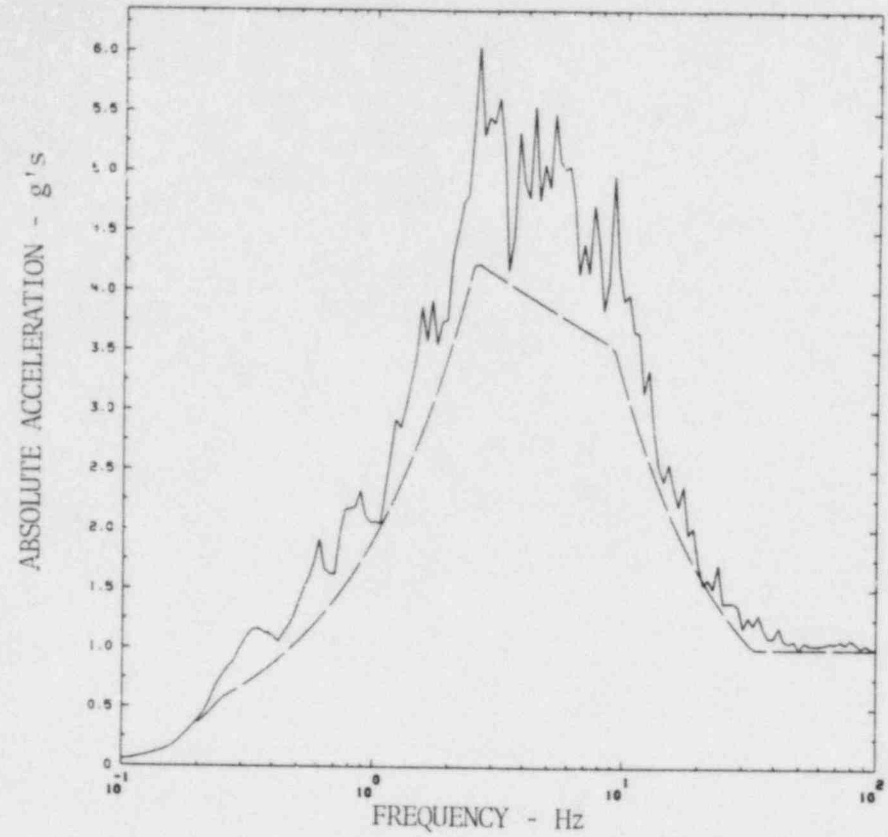

Fig. 3.11a - R.G. 1.60 Horizontal Time History 11
2% Response Spectrum vs R.G. 1.60

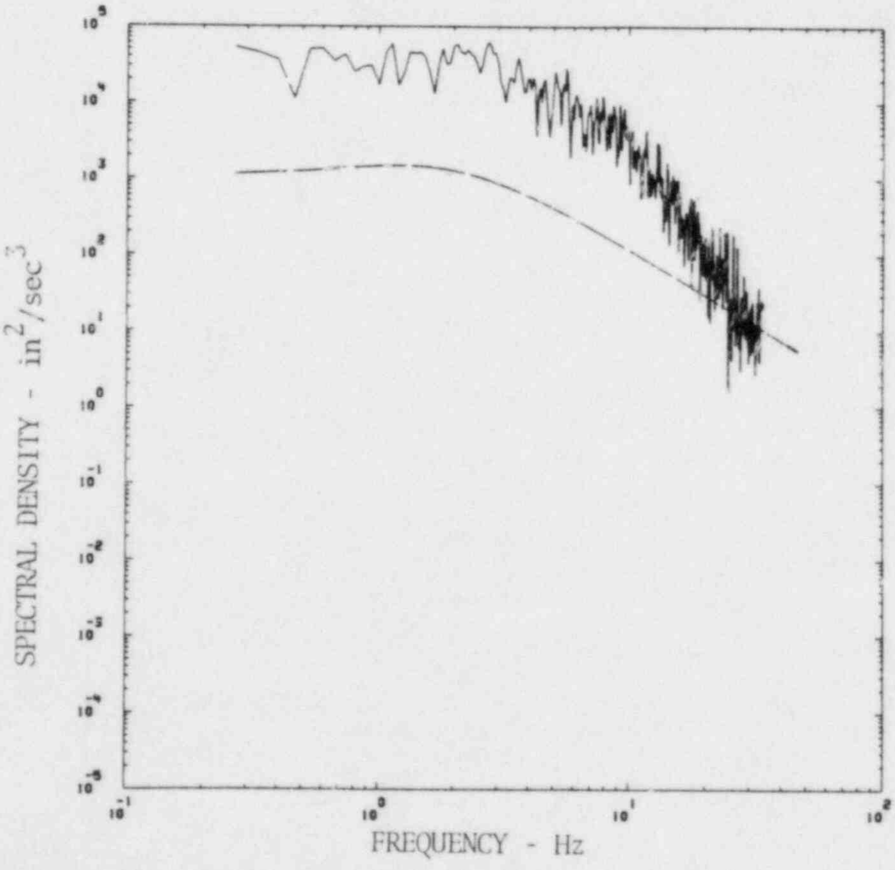

Fig. 3.11b - R.G. 1.60 Horizontal Time History 11
PSD vs NRC Proposed Criteria

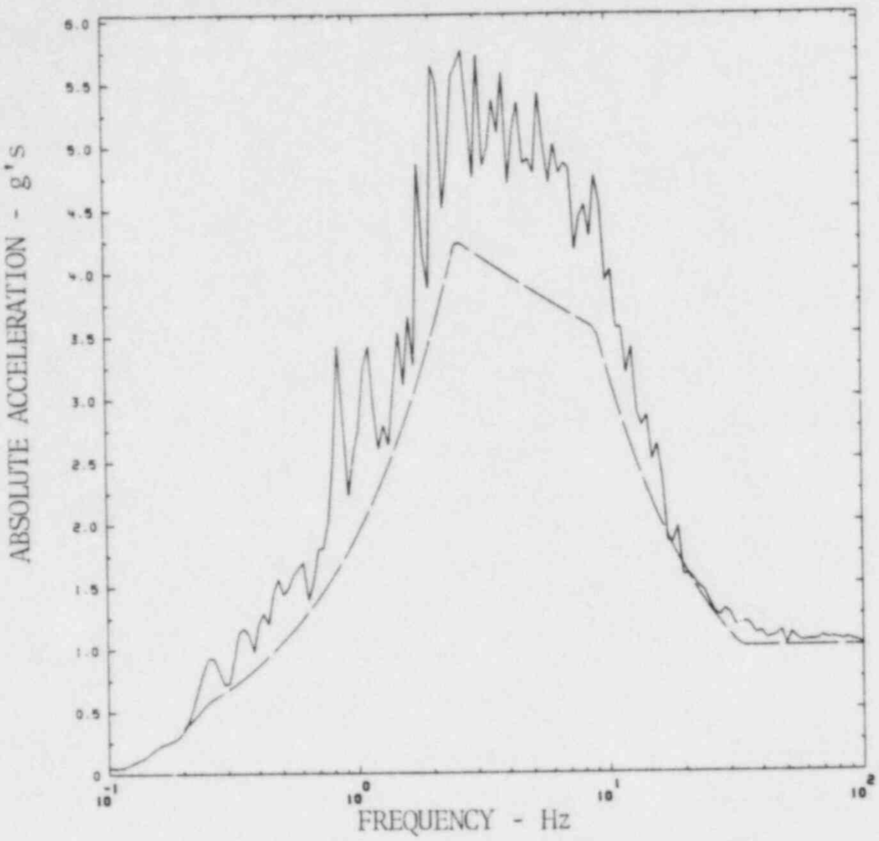

Fig. 3.12a - R.G. 1.60 Horizontal Time History 12
2% Response Spectrum vs R.G. 1.60

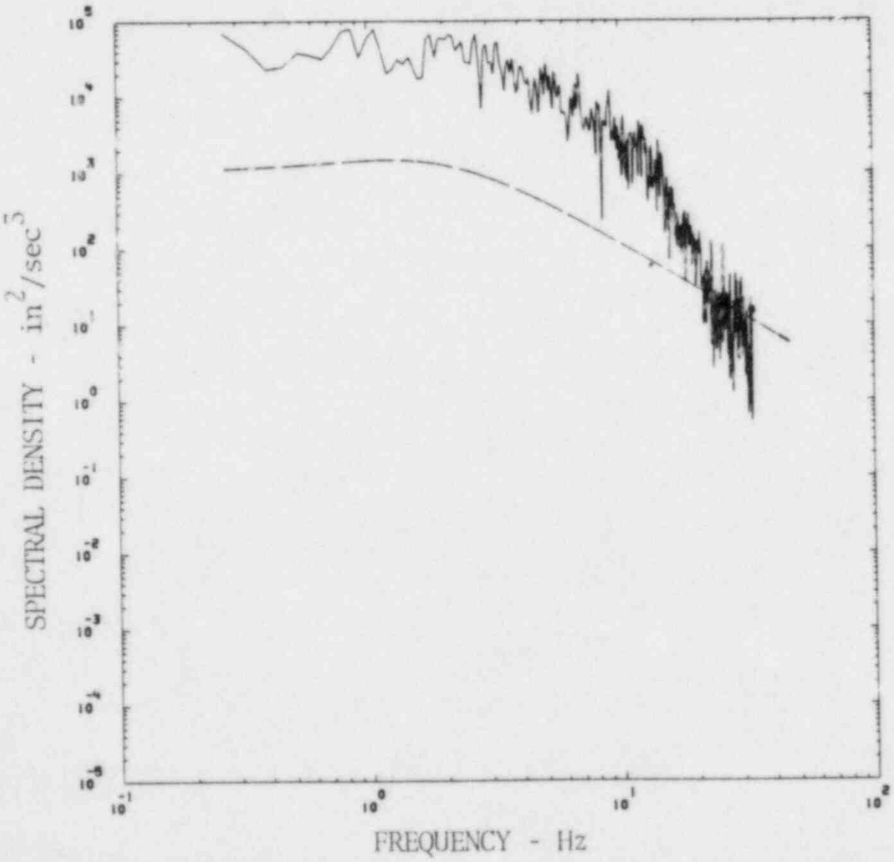

Fig. 3.12b - R.G. 1.60 Horizontal Time History 12
PSD vs NRC Proposed Criteria

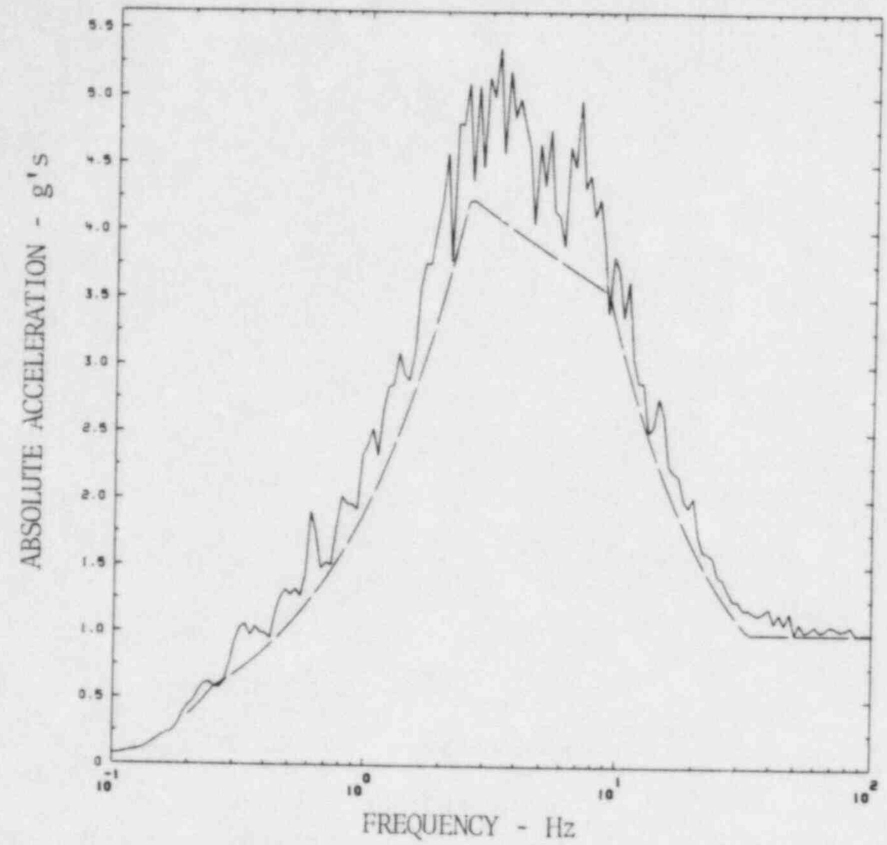

Fig. 3.13a - R.G. 1.60 Horizontal Time History 13
2% Response Spectrum vs R.G. 1.60

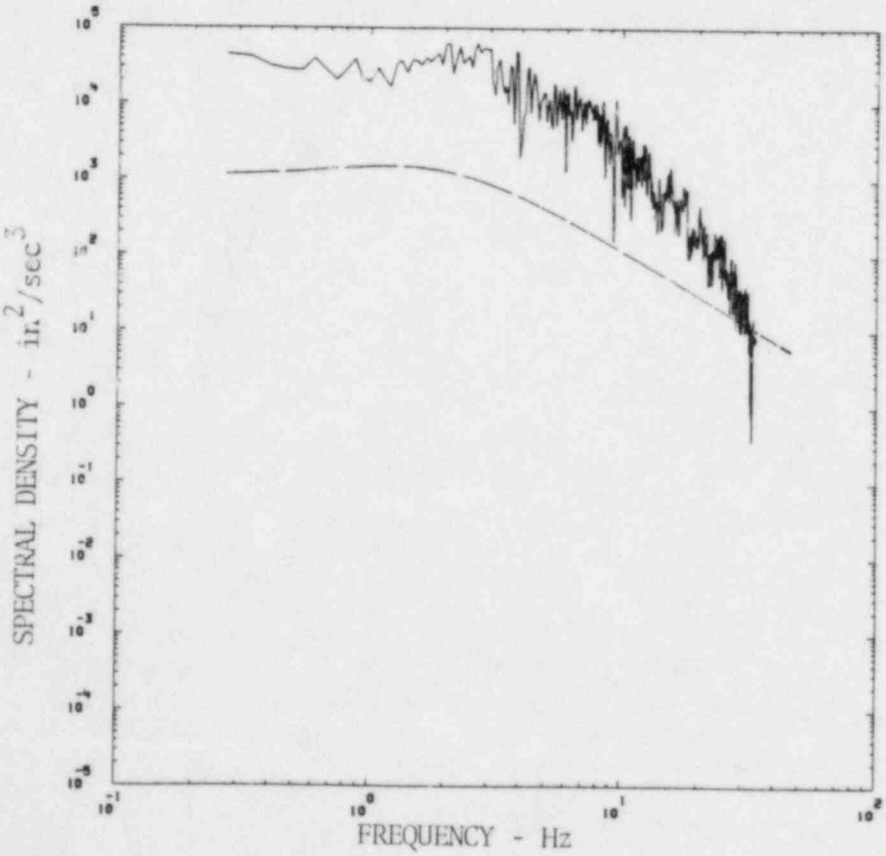

Fig. 3.13b - R.G. 1.60 Horizontal Time History 13
PSD vs NRC Proposed Criteria

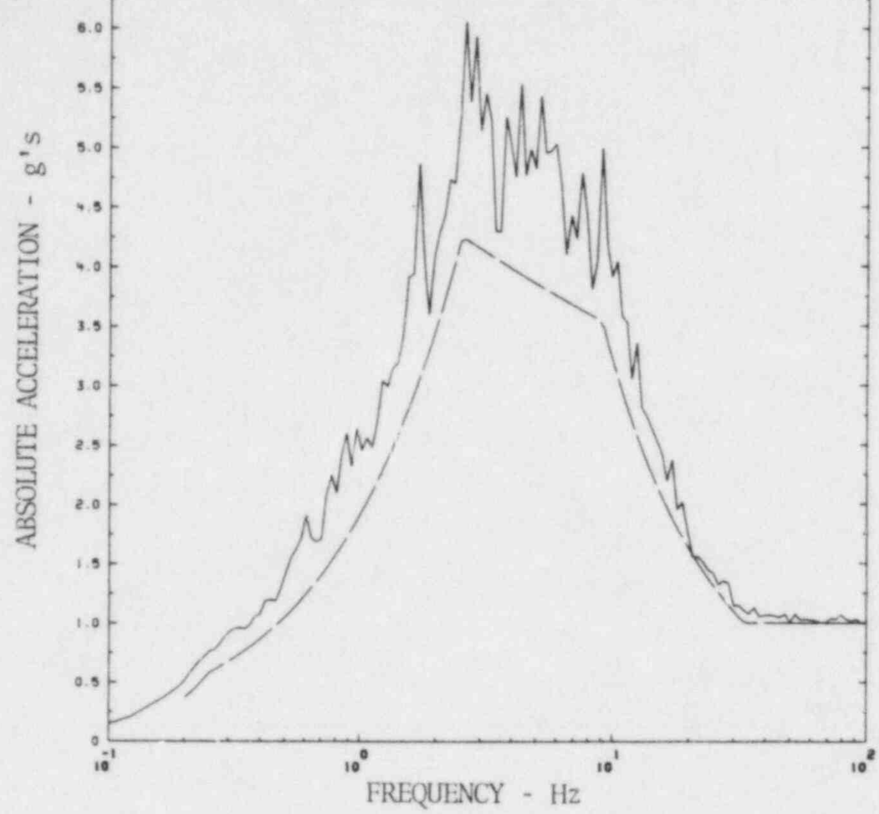

Fig. 3.14a - R.G. 1.60 Horizontal Time History 14
2% Response Spectrum vs R.G. 1.60

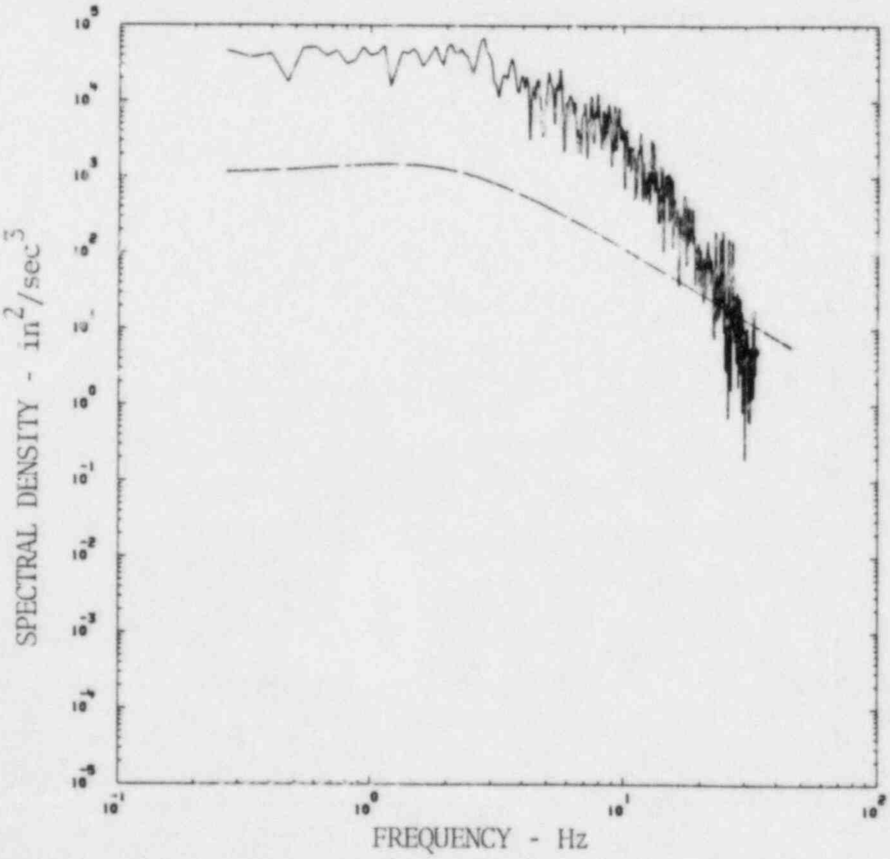

Fig. 3.14b - R.G. 1.60 Horizontal Time History 14
PSD vs NRC Proposed Criteria

* It is anticipated that the use of multiple time histories will lead to reduced loads in piping systems and equipment.

NUREG/CR-1161 presented two basic procedures for performing multiple time history analyses. They are briefly reiterated here:

Procedure 1:

* Given the design response spectra for the site of interest (e.g., R.G. 1.60), artificially generated time histories, whose mean response spectra match or exceed the mean of the design spectra, form the ensemble of earthquake motions. Each individual time history need not meet the enveloping criteria. However, the mean of the ensemble must. Using this ensemble of motions multiple analyses can be performed and mean response calculated and used in design. Although it is possible to probabilistically treat system uncertainties in this scenario, it is difficult to establish the nonexceedance probability of the result. If conventional analysis procedures were followed and only the time histories varied between analyses, the resulting mean response would reduce variability in design quantities due to time history variation. This in itself may be desirable but may not warrant the additional effort.

Procedure 2:

* Given the design response spectra for the site, establish an ensemble of earthquake motions whose 84% nonexceedance probability response spectra match or exceed the design spectra. The design spectra may be site independent, broad-band spectra or site specific spectra which are typically less broad-band. The ensemble of earthquake motions should be representative of real earthquakes. For example, the bases of R.G. 1.60 design spectra were response spectra of recorded motions. The design spectra were targeted to the 84% nonexceedance probability of the recorded spectra The resulting spectra were broad-band with different recorded motions contributing to different frequency ranges. The ensemble of recorded motions had few members, if any, with the broad frequency content of R.G. 1.60. The recorded motions constitute a possible set of motions to be used in the multiple analyses. Alternately, include artificially generated time histories whose 84% nonexceedance probability response spectra match or exceed the design spectra and whose variation is comparable to that of recorded data.

Two forms of site specific seismic input are used: 1) site specific response spectra or, 2) recorded ground motions typically from similar sites and for earthquakes of similar characteristics as the design event. In these cases, either the recorded motions or artificially generated time histories would constitute all or part of the ensemble.

For these cases, uncertainties in the system can be treated explicitly. This is recommended. One method to do so is to: identify the parameters to be treated probabilistically; define probability distributions which describe uncertainty in their values; sample from these distributions (perhaps employing experimental design techniques such as stratified sampling, Latin hypercube designs, etc.); perform multiple analyses, each analysis is based on different seismic input and input parameter values; calculate response quantities of interest (in-structure response spectra, force quantities, etc.); and estimate the values of response for 84% nonexceedance probability to be used for design.

An illustrative example of the type of results one can obtain follows. Three Zion nuclear power plant piping systems were anlayzed by a "best estimate" procedure and by a design procedure with many aspects of the SRP. Table 3.1 itemizes the two methodologies. Note, the design procedure does not encompass all aspects of the present SRP. For example, Soil-Structure Interaction analysis was performed for only one set of soil properties. Differences between the design procedure used in this example and the SRP lead to smaller values of response as calculated by the design procedure, i.e., they are underestimates of the response the present SRP would yield. Results for piping system response are shown in Figs. 3.15. Inertial responses are compared. Table 3.2 itemizes the data of Figs. 3.15 in statistical form. The best estimate values compared there are at the 84% nonexceedance probability, which was selected as the design goal.

Several conclusions can be drawn from the data:

- Large conservatisms, relative to a specific design goal, exist in the dynamic analysis of piping systems by SRP procedures. A multiple analysis procedure permits quantification of this conservatism and permits explicit specification of the goal.
- Factors of conservatism vary significantly within a system and from system to system.

- The factors of conservatism displayed here are lower bounds because all aspects of the SRP were not implemented and aspects of the "best estimate" were, in some cases, selected conservatively (parameter variations, damping values, etc.).

In terms of value/impact, multiple time history analysis, when performed in conjunction with a design goal (e.g., 84% nonexceedance probability), leads to a balanced design in terms of risk and a uniform level of risk from plant-to-plant.

Table 3.1 Best Estimate and Design Procedure Used for Comparison

Best Estimate Chain

Seismic Input	SSI	Structure	Subsystem
•Ensemble of 90 EQ Simulations	•Frequency Dependent Impedances	•Models - Same	•Models - Same
•3 Components	•Embedment Effects	•Nominal Damping - Same	•Nominal Damping - Same
•Represents Best Estimate Seismicity for 0.18g SSE	•Soil Property Variations	•Frequency and Damping Varied	•Frequency and Damping Varied
			•Mean Inertial Response

Design Procedure

Seismic Input	SSI	Structure	Subsystem
•RG 1.60	•Model - Same as Best Estimate	•Constant Frequency and Damping	•Constant Frequency and Damping
•Artificial Time Histories Match or Exceed Design Spectra	•Constant Soil Properties		•SRP 3.7.2 Envelope Response Spectra
•3 Components			•Smoothed and Peak Broadened
•PGA = 0.18g Horiz.			•RG 1.92 Combination
•3 EQ Simulations			•Average of 3 Response Spectra Analyses

Table 3.2 Ratio of Inertial Responses

Design Procedure vs. Best Estimate (84% NEP)

RHR	Number of Components	Median Ratio	COV
Accelerations	28	2.2	.24
Displacements	51	.9	.18
Support Forces	15	2.0	.17
Piping Moments	22	2.4	.16
AFW			
Accelerations	50	4.7	.48
Displacements	63	2.1	.40
Support Forces	28	4.9	.44
Piping Moments	23	4.7	.24
RCL			
Accelerations	51	7.6	1.31
Support Forces	92	6.6	.59
Piping Moments	118	9.4	.49

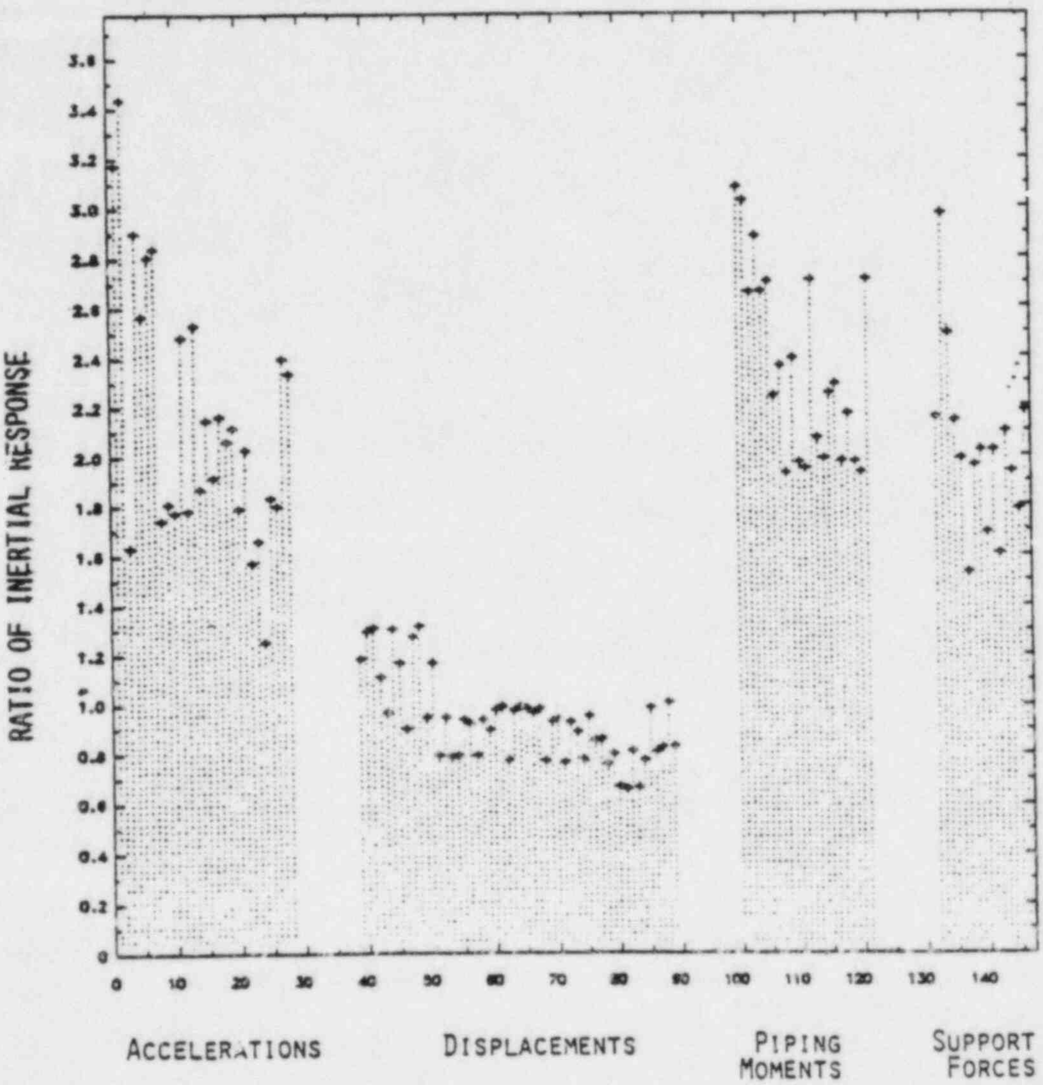

Fig. 3.15a - Ratio of Inertial Response Calculated by SRP Procedures vs Best Estimate 84% Non-Exceedance Probability Response -- Residual Heat Removal System

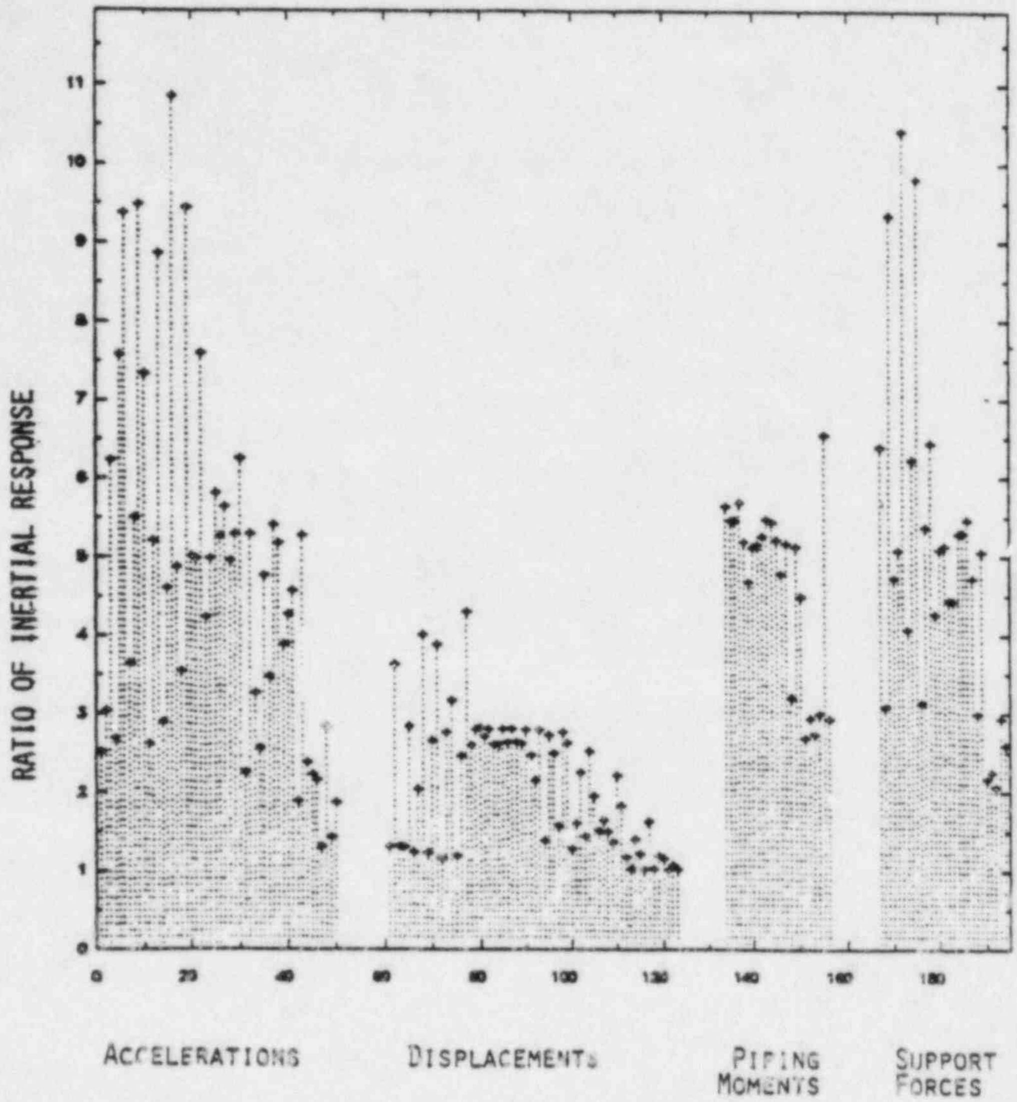

Fig. 3.15b - Ratio of Inertial Response Calculated by SRP Procedures vs Best Estimate 84% Non-Exceedance Probability Response -- Auxiliary Feedwater System

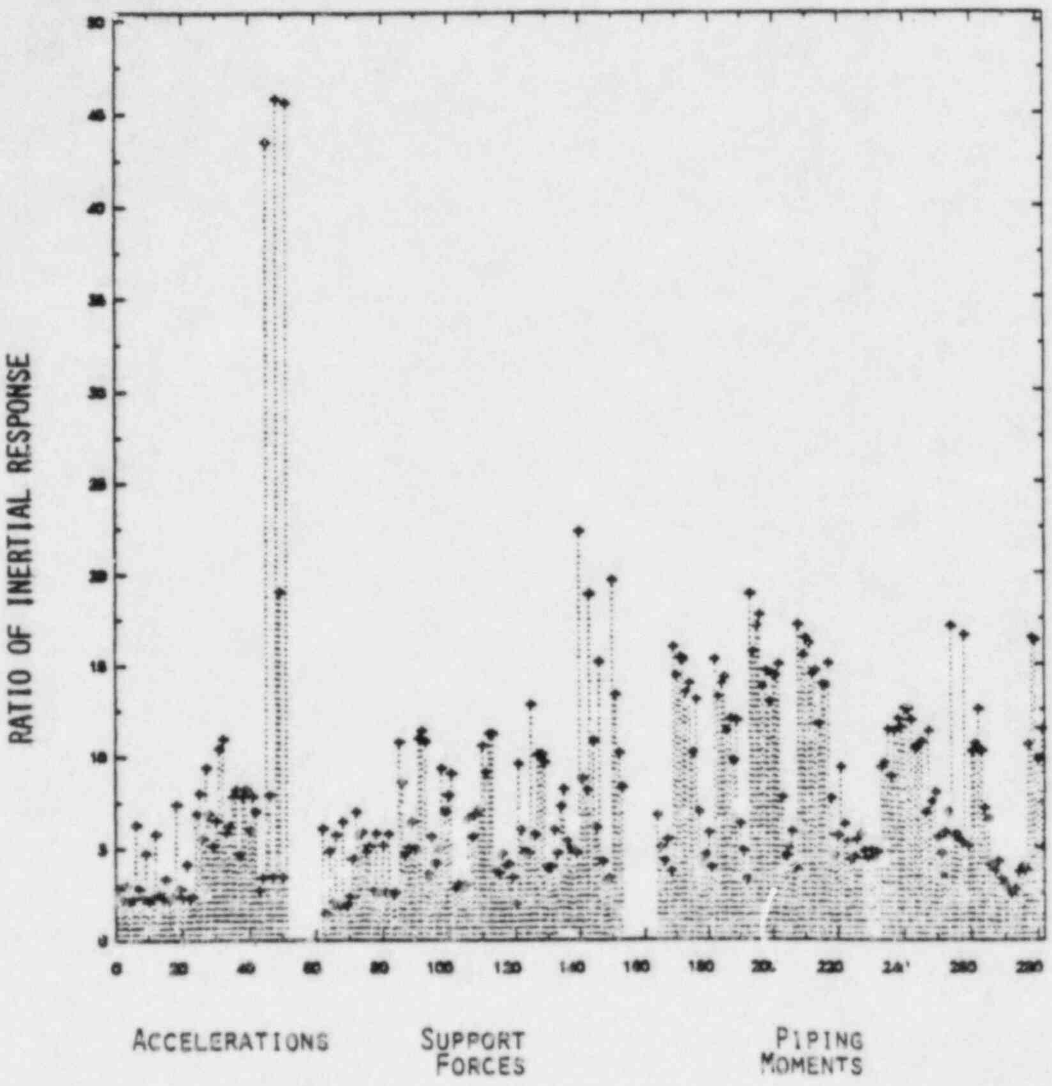

Fig. 3.15c - Ratio of Inertial Response Calculated by SRP Procedures vs Best Estimate 84% Non-Exceedance Probability Response -- Reactor Coolant Loop

3.1.3 Value/Impact Assessment

Judging by the results of the comparison, it would appear that the additional PSD requirement being proposed is currently being satisfied over the significant frequency ranges for structure response. Hence, only a small additional cost would be anticipated, i.e., the cost to compare PSD's and, perhaps, the cost of regenerating artificial time histories to meet the criteria in the frequency range greater than 20 Hz.

The cost of regenerating artificial time histories would represent a small, one-time analysis cost that would probably come out of overhead funds. Estimates received from the NRC staff indicate that additional review time associated with the PSD requirement would be approximately three man-weeks.

Thus, it is clear that analysis and review costs associated with this proposed new requirement are small (i.e., less than $5000).

The impact on design of the proposed change is expected to be minor. Also, in light of the potential conservatisms associated with artifical time histories discussed earlier, it appears that the original goal of R.G. 1.60 is being met or exceeded.

With regard to a reduction in variability due to artificially generated time histories, the additional PSD requirement may in general have little impact, since there is variability in the data-set of the 14 time histories evaluated and they meet the PSD over the frequency range of most interest.

3.2 Development of Floor Response Spectra and Effects of Parameter Variations on Floor Response Spectra

Proposed changes to SRP sections 3.7.2.II.5 and 3.7.2.II.9 dealing with acceptance criteria for the generation of floor response spectra and the consideration of parameter variation in developing floor response spectra are covered in the following changes.

3.2.1 Summary of NRC Proposed SRP Changes*

#1. Acceptance criteria is given for development of floor response spectra when a single artificial time history is used. All provisions of R.G.

* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

1.122 are required, as well as a justification for the use of a single artificial time history.

#2. The use of multiple time histories to generate floor response spectra is reviewed and accepted on a case-by-case basis. Particular items reviewed are:

- Number of time histories used.
- Procedures used to account for uncertainties.

#3. The use of direct solution methods for development of floor response spectra are reviewed and accepted on a case-by-case basis. Two key points are:

- The theoretical basis of the technique must be demonstrated.
- Selected comparisons are required between direct solution results and results from a time history approach.

#4. Acceptance criteria for considering parameter variations in developing floor response spectra are as provided in SRP section 3.7.2.II.5.

3.2.2 Technical Discussion of Issues

Currently, Sec. 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 at structure locations of interest. 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 synthetic time history of motion generally envelop the design response spectra for all damping values to be used. In addition, Sec. 3.7.2 of the SRP encourages the use of a time-history approach to generate in-structure 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."

In NUREG/CR-1161, it was stated that the use of time histories, for which the response spectra envelop the design response spectra for all damping values, tends to artificially introduce an added and unnecessary conservatism into the analysis of about 10%. 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 that reasonably closely envelops the design response spectra. Furthermore, it has also been observed that two different synthetic time histories, both of which result in response spectra that adequately envelop the R.G. 1.60 response spectra, can lead to in-structure spectra that may differ by a factor of two or more.

Because of these and other concerns, NUREG/CR-1161 made specific recommendations relating to the generation of in-structure response spectra, several of which are discussed below:

Floor Spectra - Single Artificial Time History

When developing floor spectra using a single artificial time history, all provisions of R.G. 1.122 are required. In addition, the single artificial time history used must meet the current acceptance criteria with respect to R.G. 1.60 response spectra, as well as meeting the proposed new PSD function requirement specified by the NRC staff. The technical issues relating to the proposed new acceptance criterion, have been discussed in detail in section 3.1.1 of this report, and the reader is referred to that section for further details.

Floor Spectra - Multiple Time Histories

As an option to the use of a single artificial time history to develop floor response spectra, the licensee is allowed to develop floor spectra through the use of multiple time histories. The use of this option is reviewed by the NRC staff on a case-by-case basis.

NUREG/CR-1161 suggested various acceptance criteria for the use of both real and artificial multiple time histories in the generation of floor spectra. The reader is referred to that document for a detailed discussion of this issue.

Although the use of multiple time histories for the generation of floor spectra represents more calculation than is typically required today, the economic impact is much less severe than might be expected. This is because one of the most significant costs is associated with mathematical model development rather than analysis. Furthermore, for various reasons, multiple analyses are often performed in present practice, though not required.

The overall benefits of the use of multiple time histories -- 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 -- are believed to significantly outweigh any disadvantages.

One example of the type of results to be expected, from the use of multiple time histories, is shown in Figs. 3.16 and 3.17. These plots compare in-structure spectra found using multiple time histories (30 records) and a best estimate approach to in-structure spectra found using a single artificial time history with peak-broadening techniques and meeting current SRP requirements. These plots were generated for the Zion plant. Figure 3.16 shows in-structure spectra plots calculated for a point on the containment structure. Figure 3.17 shows in-structure spectra plots for a point on the internal structure. Both figures show that the use of multiple time histories, with a best estimate approach, can lead to considerable reductions in spectral ordinates.

Additional studies in this area are needed to further quantify the potential gains associated with the use of multiple time histories.

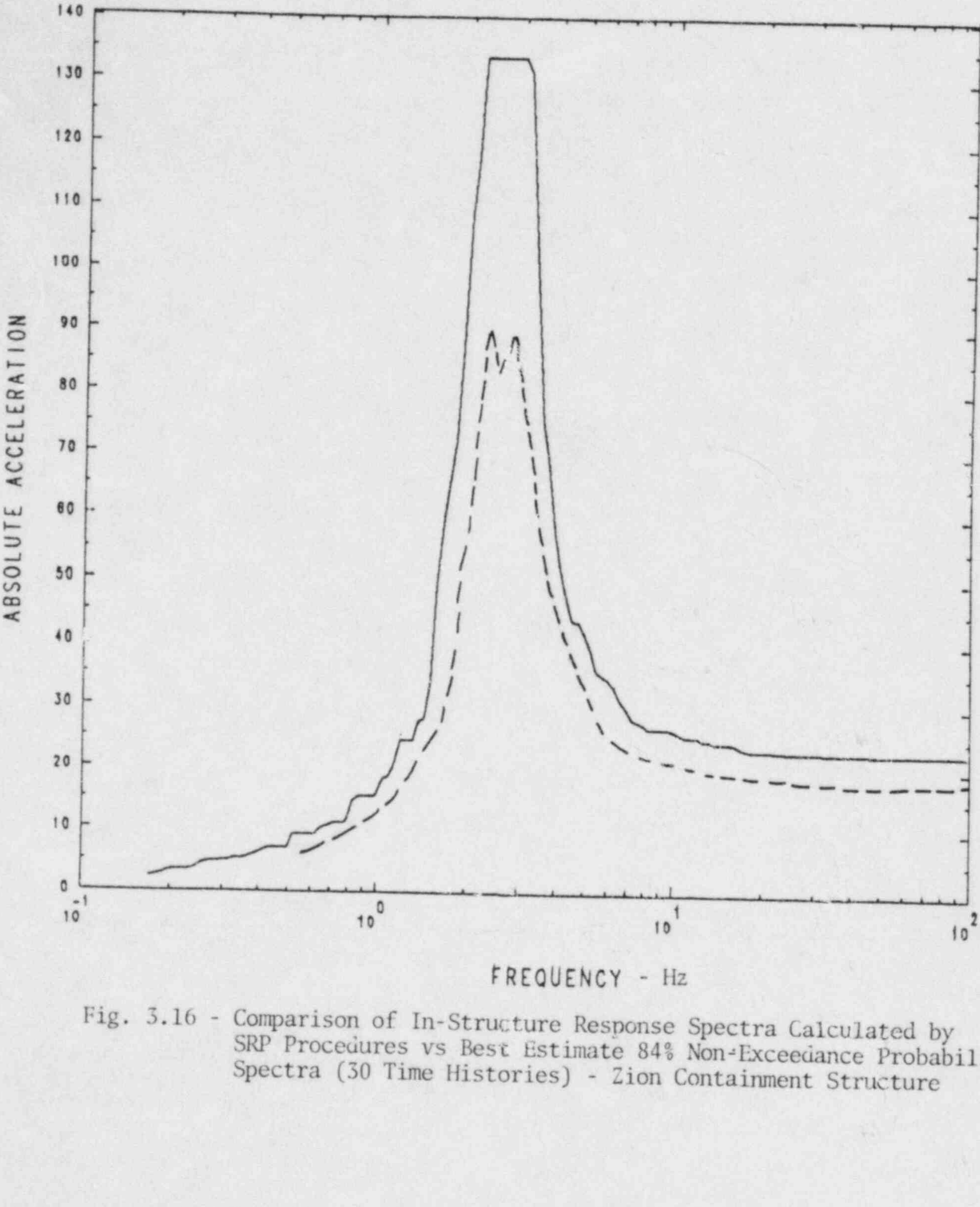

Fig. 3.16 - Comparison of In-Structure Response Spectra Calculated by SRP Procedures vs Best Estimate 84% Non-Exceedance Probability Spectra (30 Time Histories) - Zion Containment Structure

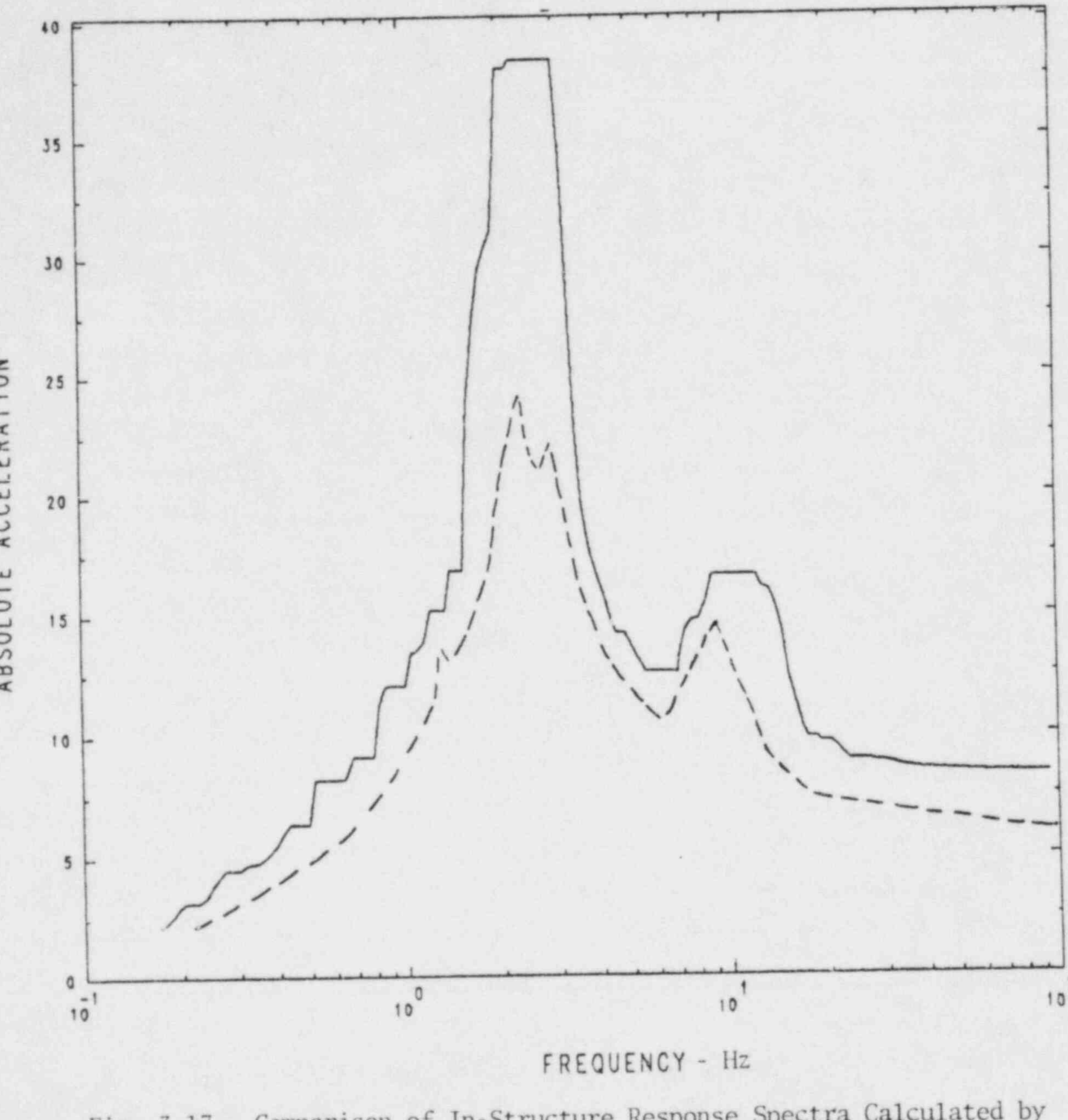

Fig. 3.17 - Comparison of In-Structure Response Spectra Calculated by SRP Procedures vs Best Estimate 84% Non-Exceedance Probability Spectra (30 Time Histories) -- Zion Internal Structure

Floor Spectra - Direct Solution Methods

Many algorithms are currently available that allow the generation of in-structure response spectra directly from the ground response spectra without time-history analysis. See for example: Singh, M.P. [1975, 1979]; Scanlan, et.al. [1977]; and Schnitz, D., et.al. [1977]. Because these algorithms are efficient, parametric studies are economical. These methods use the SRSS method for combination of components and produce smooth, realistic spectra.

At LLNL we are most familiar with the approach used by Singh. We have found that his method produces excellent, consistent, and repeatable results as compared to time-history approaches. His method is based upon the assumption that earthquake motions can be modeled as a homogeneous random process. The concept of a spectrum-consistent power spectral density function has been used in the development of this method. Figure 3.18 compares the two percent damped floor spectra generated at one level in Dresden 2 using Singh's method versus that obtained from an artifical time-history analysis. The artificial time history used closely approximated the Regulatory Guide 1.60 response spectra at each natural frequency of the structure. It generated a response spectrum which tended to be mean centered on the Regulatory Guide 1.60 spectrum as opposed to enveloping the Regulatory Guide 1.60 spectrum. Thus, no conservative bias was introduced by use of this time history. One can see the excellent agreement obtained between the floor spectrum for the Singh method and this artificial time history. This figure is representative of the results obtained for many other cases as well.

Although we are not as familiar with the other direct generation methods referenced, they are all based upon sound theoretical backgrounds and are suitable for adaptation on computers. We believe that direct generation methods, in conjunction with parametric studies, would reduce the uncertainties associated with in-structure spectra generated from synthetic time histories.

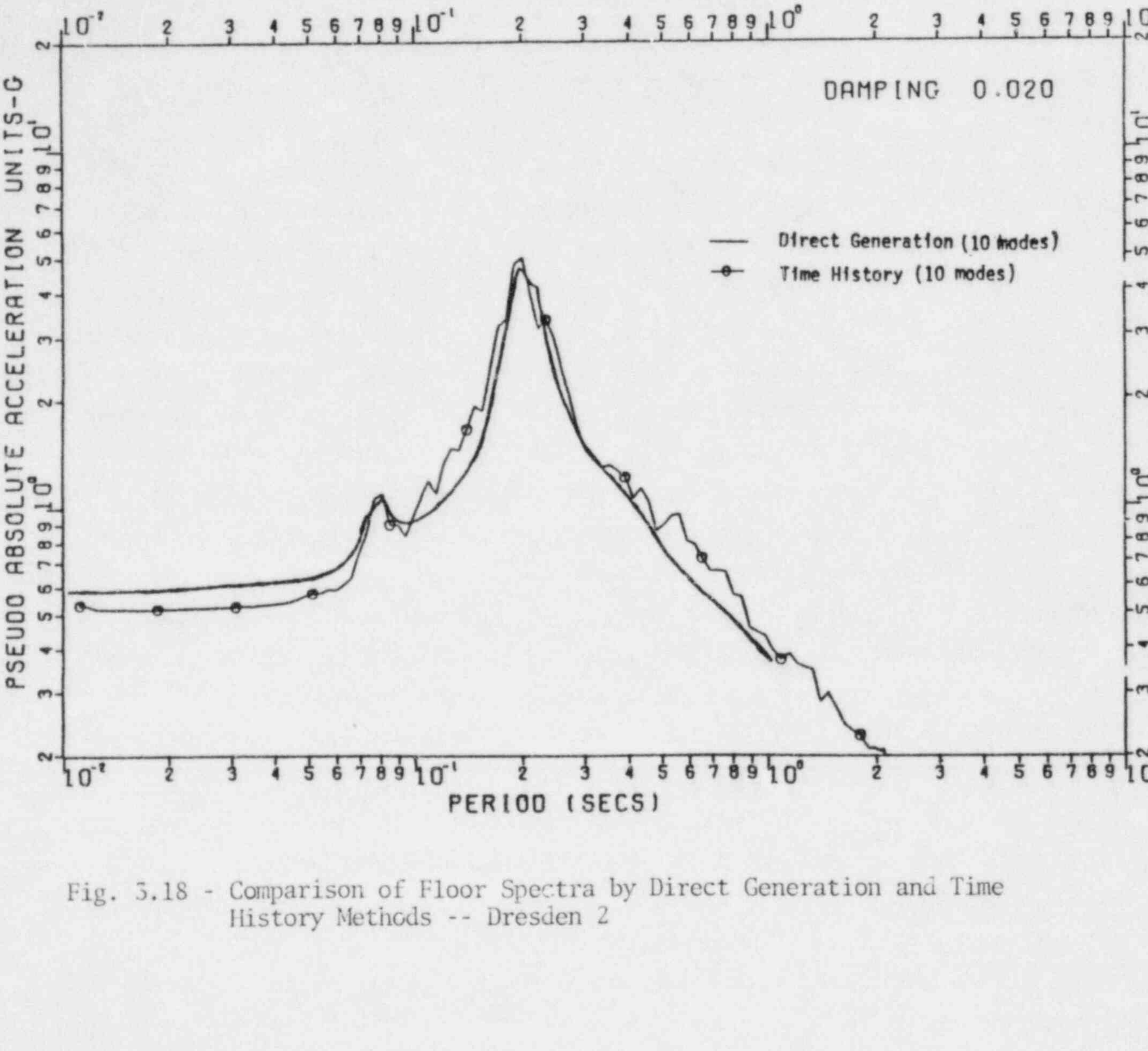

Fig. 3.18 - Comparison of Floor Spectra by Direct Generation and Time History Methods -- Dresden 2

Floor Spectra - Parameter Variations

Regulatory Guide 1.122 requires the broadening of in-structure spectra to account for uncertainties in the structural response characteristics. Such broadening 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 that lead to broadening of the in-structure spectra also lead to a reduction in the peak spectral amplitudes that have 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, in-structure spectra.

When in-structure spectra are generated using multiple time histories or direct generation techniques, it is possible to account for uncertainties directly through the variation of parameters (i.e., damping, stiffness, soil properties, etc.).

Studies performed by Smith, et.al. [1977], compared equal-probability-of exceedance in-structure spectra with deterministic in-structure spectra. The former 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 twice as high as those in the equal-probability-of-exceedance spectra. Thus, considerable conservatism is introduced within the broadened-peak region of the deterministic spectra. On the other hand, conservatism is reduced slightly at frequencies outside of the region of broadened peaks, i.e., outside modal frequences.

When multiple time histories or direct generation of in-structure spectra are coupled with structure and soil parameter variation, the mean of the resulting spectra will be flatter than current spectra -- the valleys raised, the peaks lowered -- and, as such, would represent a more rational seismic design basis for subsystem design than do deterministic in-structure response spectra.

3.2.3 Value/Impact Assessment

The value/impact assessment dealing with the development of floor response spectra and the effects of parameter variations on floor response spectra, is essentially the same as given in Section 3.1.3 of this report. This is so because the only new change that represents a requirement, and not an option, deals with the addition of the NRC staff's PSD function acceptance criterion for the use of a single artificial time history. Thus, no additional impact in engineering and review costs are anticipated. Also, as discussed in Section 3.1.3, the impact of the PSD requirement on design is expected to be minor, since it appears that the development of artificial time histories, using the current criteria, meets the proposed new PSD requirements for most of the frequency range of interest.

Although the use of multiple time histories or direct generation techniques for floor spectra development are left as options, it is our opinion that these procedures would lead to reduced loads in piping systems and equipment. Furthermore, the use of these approaches, coupled with parameter variations, should lead to a reduction in the variability of risk among plant sites.

3.3 Percentage of Critical Damping Values

3.3.1 Summary of NRC Proposed SRP Changes*

This change is a clarification of current NRC practice which allows the use of higher damping values than those given in R.G. 1.61, if documented test data are provided to support them. Additionally, a correlation between stress levels and damping values is required. Use of higher damping values is reviewed on a case-by-case basis.

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

3.3.2 Technical Discussion of Issues

The damping values currently given in R.G. 1.61, for use in seismic design, are based primarily on informed professional judgment and are considered to be conservative. Since these values were first proposed and accepted, a growing body of damping test data for nuclear power plant structures, piping, and equipment has accumulated. See, for example, Hart and Ibanez [1973], Hart, et.al [1973], Morrone [1974], Singh, et.al. [1980], Shibata [1981], Stephenson, [1980], Shibata, et.al. [1979], Ware [1982] and Coats [1982]. This body of test data supports the widely held belief that higher allowable damping values than those given in R.G. 1.61 are justified.

Piping Systems

There has been growing concern that seismic considerations dominate the design of piping systems to a greater extent than they should, compared to consideration of more normal or frequent loadings, such as those due to thermal effects. This concern is supported by studies on seismic margins and seismic risk, and, by an evaluation of the performance of piping systems in earthquakes. The conservative values of damping, specified for use in design, result in piping systems that have more supports than would be required if more realistic damping values were used. These stiffer systems, although highly resistant to dynamic loads, become more severely stressed during thermal growth transients. This illustrates the somewhat unique challenge in attempting to obtain an optimum or balanced design in nuclear power plants. The potential consequences of a design assumption or approach are often not fully recognized until a comprehensive risk assessment of the entire plant is performed. Such an assessment may very well show that strengthening one part of a plant may actually increase the risk of radioactive release.

This concern is plausible since there is a trade-off required between seismic and thermal loadings. The general design objectives are that for seismic loadings we would like a stiff piping system but for thermal loadings we would like a flexible piping system. The design is seen as a trade-off between these two opposing objectives. We have no basis to believe that the current safety requirements lead to an optimal design.

The concept of trade-offs was recognized some time ago and led to the development of the snubber for seismic restraint. If this approach works it might be ideal. It does not appear that it works well. Snubbers do not reliably perform as designed. As a reaction to this, or other issues, present practice leads to excess numbers of rigid restraints. There is, thus, a significant tendency to design piping sytems that are relatively rigid under thermal loadings. This violates one of the general design objectives.

This concern is corroborated by past experience. Piping systems in conventional power plants are not as restrained as are nuclear piping systems. This has not led to degraded performance during earthquakes.

This concern is also supported by current seismic PRA results. These results show that the failure of piping systems is not a significant risk contributor, with some exceptions which are a result of poor design practice. An example of poor practice is restraining a piping system which spans two structures in close proximity having independent foundations. It is noteworthy that this "poor practice" does not violate current safety requirements. This suggests, again, that the use of margin as a broad safety assurance measure may not be as effective as other, more focused, measures. However, the current seismic PRA results on piping systems are consistent with the perception that seismic considerations have been over emphasized to a significant degree in the design of piping systems.

A significant effort is being expended to address this concern through the Pressure Vessel Research Committee (PVRC). The efforts of the PVRC may succeed in modifying current seismic safety requirements. This may lead to an increase in the relative contribution of piping systems to seismic risk. Although the relative contribution of piping system failures in seismic PRA studies may be increased, this may not significantly increase the total seismic risk because piping does not appear to be a significant contributor to seismic risk. If this last point can be convincingly shown, then we may well conclude that seismic considerations in piping systems have been over emphasized to a greater degree than is necessary.

The small damping values which are currently being used for the seismic design and analysis of piping systems appear to be estimates based mostly on material damping and on low levels of excitation. From laboratory tests on pipe specimens, it is shown that these small material damping values are valid for low stress levels. However, for higher stress levels, considerably higher damping values have been obtained than those given in R.G. 1.61. Material damping would be applicable, for example, to small isolated piping whose vibrations do not appreciably affect the whole system and with little friction at the support connections. For larger piping systems where the structural response is interdependent, system damping appears to be predominant. This has been shown by the data obtained from the response of these components to actual earthquakes and to forced vibrations [Morrone, 1974].

For the primary coolant loop components of the San Onofre Nuclear Generating Station, damping values up to 2% of critical were obtained from forced vibration tests with very small displacements. These increased to a maximum of about 3.3% of critical resulting from the 1971 San Fernando earthquake. This damping value was obtained with a comparatively low level of excitation. For example, in the horizontal N33E trace, the maximum ground acceleration was only 0.012g at the San Onofre site. This value is approximately 1/16 of a high seismic region typical OBE maximum ground acceleration of 0.2g. Values of 3.2 - 8.6% of critical have also been obtained from Japanese tests of large and medium size piping [Akino, et.al. 1971].

Preliminary test results of experimental investigations of damping in nuclear power plant piping systems have been reported by Ware [1981]. The primary data source was tests conducted at the Heissdampfreaktor in Germany. Among other factors, the effect of excitation type, excitation level, pipe size, support type and response frequency were investigated. The conclusions reached by Ware, regarding these factors, indicate the following:

1. While the data was insufficient to conclude that damping was affected by the type of excitation, e.g., seismic or blowdown, there was a slight trend that direct methods of exciting the piping produced higher damping than did indirect methods where the excitation had to travel through soil, buildings, etc.
2. A trend was observed that higher excitation levels produced higher damping. However, this trend was quite weak with one test showing that increasing the force level by a factor of 10 resulted in increased damping levels by a factor of 2 or less.
3. No conclusion on the effect of the size of pipe on damping could be drawn. However, the type of pipe supports which was often determined by pipe size are a strong influence on system damping.

4. The data indicates that pipe support type is a strong factor in system damping. Rigid supports exhibit low damping, while systems with energy dissipating supports such as constant force hangers and snubbers have much higher damping.

5. There is a strong indication of increased damping with decreasing frequency below 20 Hz. This trend seems to fit well with mass proportional damping where damping is inversely proportional to frequency. Curve fitting techniques showed that mass proportional and power fits generally represented the data best.

Although most data seems to indicate that piping damping values increase with higher response amplitudes, some of the data is contradictory.

Recent studies made in Japan, as part of the Seismic Damping Ratio Evaluation Program [Shibata, et.al., 1981], have shown that, for piping systems having multiple supports, the damping ratio increases with vibration amplitude up to a point and then decreases with further increases in response amplitude. It is postulated that damping ratios are higher at low amplitudes of vibration because interface shear, slip effects and Coulomb type damping at support points dominate. As the amplitude increases, these sources of damping are overcome and material damping becomes predominant.

Even so, it is widely believed that higher allowable damping values would be more reasonable, and would be beneficial to the nuclear industry by reducing the number of required piping supports. More realistic stress analyses would be possible. Expenses would be reduced in design, analysis, procurement, and installation of supports. There would be less chance of a support malfunction since there would be fewer supports, and piping systems would undergo less stress when responding to thermal transients.

Structures

Damping affects not only piping systems in nuclear power plants, but all systems and structures. A comprehensive compilation of damping test data from real buildings, as a function of structure type and amplitude level, is contained in a study made by Haviland [1976]. In this report, 244 damping values for 139 buildings were collected from 39 references. The data represents fundamental mode damping values for steel, reinforced concrete, and composite structures subjected to small and large amplitude vibrations.

Vibrational sources for the small amplitude damping values include: underground nuclear events, mechanically forced by eccentric rotating mass or pull-release, small earthquakes, man-induced, wind-induced and ambient vibrations. Small amplitude vibrations account for 192 data points of the complete set. The remaining 52 damping values are associated with large amplitude data from two earthquakes. The first was an earthquake of magnitude 6.1, occurring on July 1, 1968, with an epicenter 45km northwest of Tokyo, Japan. The second was the San Fernando, California, earthquake of February 9, 1971, which registered 6.6 on the Richter scale with an epicenter approximately 45 km north of Los Angeles. These two earthquakes have essentially provided the only significant set of large amplitude building damping data available to date. Haviland statistically reviewed the complete data set of 244 values using two series of parameters, amplitude of motion and structural type, to produce a total of 12 histograms. Table 3.3 summarizes the statistics of the histograms. The following observations can be made from an examination of the Table:

- For each category of structural type, the mean value of damping increases with increase in amplitude of motion.
- The variance for each distribution is unique, which indicates that the data spread in each distribution varies significantly.
- The coefficient of variation for the six combinations of structural type and amplitude are similar, suggesting common parameters may be developed for a probabilistic damping model.
- The mean value for small amplitude motion of composite construction is greater than the mean for steel buildings but less than the mean for reinforced concrete buildings. This is expected, as composite materials typically display behavior ranging between the extremes of the individual components.
- The mean value for large amplitude motion of composite construction is less than the mean value for reinforced concrete and steel. This apparent anomaly may be due to the relatively small sample size.

A comparison of the sample means from Table 3.3 for large amplitude motions with the corresponding SSE level values specified in R.G. 1.61 shows good agreement. The small amplitude values from the Table were obtained from very small amplitude tests, producing stress levels well below working stress. Thus, a meaningful comparison of this data with the OBE values of R.G. 1.61 is not appropriate.

Table 3.3 Summary of Statistics for Histograms of Damping Determinations [Haviland, 1976]

	AMPLITUDE														
	SMALL					LARGE					ALL				
STRUCTURAL TYPE	n	$\overline{x}$	s^2	s	c.o.v	n	$\overline{x}$	s^2	s	c.o.v	n	$\overline{x}$	s^2	s	c.o.v
REINFORCED CONCRETE	104	4.26	10.49	3.23	0.76	17	6.63	17.99	4.24	0.64	121	4.60	12.06	3.47	0.76
STEEL	41	1.68	1.18	1.08	0.65	12	5.65	6.47	2.54	0.45	53	2.58	5.09	2.26	0.87
COMPOSITE CONSTRUCTION	47	2.72	1.31	1.14	0.42	23	3.23	3.08	1.76	0.54	70	2.89	1.91	1.38	0.48
ALL	192	3.33	7.36	2.71	0.81	52	4.91	10.71	3.27	0.67	244	3.67	8.45	2.91	0.79

n = sample size
$\overline{x}$ = sample mean (% critical damping)
s^2 = variance
s = sample standard deviation
c.o.v. = coefficient of variation

Test data, compiled by Hart and Ibanez [1973], Hart, et al. [1973], and Hornbuckle, Jr., et al. [1973], on nuclear containment structures indicates damping values considerably higher than those specified in R.G. 1.61. The testing procedures used consisted of harmonic vibrators and underground dynamite blasts. In all cases studied, the displacement amplitudes and accelerations were quite small. The relatively high damping values observed are probably associated with soil-structure interaction and embedment effects.

A summary of test data damping values from conventional structures, nuclear containment structures, and piping and equipment is given by Coats [1982].

Sensitivity studies of the effects of changes in damping values on response quantities were made by Smith [1977] as part of a study to assess conservatisms in NRC seismic design requirements. Smith analyzed a series of shear-beam models having a wide range of fundamental frequencies. These models were subjected to synthetic time histories whose response spectra were designed to comply with R.G. 1.60. A range of damping values from 1% to 10% of critical was used. Peak shear, moment, relative displacement, and acceleration, as well as, floor spectra were calculated for the various models and range of damping values. Results indicated that structural response is not very sensitive to the damping value used, with an X% change in damping producing an X/2% change in peak acceleration for damping values near 5% of critical. Smith's study also concluded that floor response spectra exhibit about 20% greater sensitivity to variations in damping values than did structural response, and that sensitivity of shear and moment values to changes in damping is much less than that of peak acceleration.

The implications here are that, while test data and engineering judgment would support the use of higher damping values than currently specified in R.G. 1.61, the application of higher damping values for design may not result in significant changes to most plant structures. However, design of piping systems may be affected in the number of piping supports required.

3.3.3 Value/Impact Assessment

No value/impact assessment of this change has been performed since the change involves a clarification and an option, and, because explicit recognition of existing stress criteria in R.G. 1.61 is not viewed by NRC staff as a change.

3.4 Soil-Structure Interaction

Clarifications are made to SRP Sections 3.7.2.I.4 and 3.7.2.II.4, regarding the kinds of uncertainties which must be addressed in soil-structure interaction analyses.

3.4.1 Summary of NRC Proposed SRP Changes*

#1. Specifically states that uncertainties in soil-structure interaction analyses must be recognized and addressed. These uncertainties include:

- transmission of input motion
- random nature of soil configuration and material characteristics
- uncertainty in soil constitutive modeling
- nonlinear soil behavior
- coupling between structures and soil
- lack of symmetry in soils and structures
- moisture content in soils
- loss of contact between soil and foundation

3.4.2 Technical Discussion of Issues

The response of a structure during an earthquake depends on the characteristics of the ground motion, the surrounding soil, and the structure itself. For structures founded on rock or very stiff soils, the foundation motion is essentially that which would exist in the soil at the level of the foundation in the absence of the structure and any excavation; this motion is denoted the free-field ground motion. For soft soils, the foundation motion differs from that in the free field due to the coupling of the soil and structure during the earthquake. This interaction results from the scattering of waves from the structural foundation and the radiation of energy from the structure due to structural vibrations. Because of these effects, the state

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

of deformation in the supporting soil is different from that in the free-field. In turn, the dynamic response of a structure depends on the characteristics of the underlying medium. For example, if the soil is soft, the coupled soil-structure system will exhibit a peak structural response at a lower frequency than will an identical rigidly supported structure.

A number of methods are available to analyze these effects. They generally fall into two categories: the direct method, which analyzes the idealized soil-structure system in a single step, and the substructure approach, which treats the problem in a series of steps: determination of the foundation input motion, determination of the foundation impedances, and analysis of the coupled system. Both methods can be discussed in terms of two basic elements: specifying the local free-field ground motion and idealizing the soil-structure system. The second of these elements involves modeling the configuration and properties of the soil, the geometry and stiffness of the structural foundation, and the complexities of the structure itself. Taken together, these two elements of SSI analysis comprise the following components:

Specifying free-field motion

- Describing the free-field ground motion entails specifying the point at which the motion is applied (control point), the amplitude and frequency characteristics of the motion, and the spatial variations of the motion. For both analysis methods, definition of the control motion depends on the assumed soil configuration, soil material behavior, and the wave-propagation mechanism at the site.

Idealizing the soil-structure system

- Idealize the soil configuration.
- Represent dynamic soil behavior. Three-dimensionality should be considered, as should nonlinear soil behavior. The latter effect is most often approximated by iterative linear analyses.
- Model the structure.
- Model the foundation. Two aspects are important -- geometry and stiffness. Partial embedment must also be considered.
- Model structure-to-structure interaction.
- Account for localized nonlinearities, primarily the effects of separation or debonding of soil and structure.

Uncertainties exist in each of the above steps of any SSI analysis. Many aspects of SSI are understood, and any valid method of analysis should be able to reproduce them. However, no deterministically exact solution of the physical SSI problem, in its entirety, can be obtained by existing techniques. As a consequence, any discussion of the accuracy of an analysis must take account of several factors. First, different analyses may have, as their aims, the prediction of different quantities, e.g., structural response or the state of stress at a point in the soil. Second, accuracy may be measured in either a probabilistic or a deterministic sense. Third, any estimate of accuracy should include a measure of the uncertainty in the results.

A thorough exploration of the uncertainties present in any SSI analysis can be found in the reports by Roesset [1980] and Johnson [1981].

FREE-FIELD GROUND MOTION

One of the largest sources of uncertainty in any seismic analysis lies in the specification of the free-field ground motion. Three aspects of the free-field motion are of particular interest -- the amplitude and frequency characteristics of the motion, the location of the control point, and the spatial variation of the motion. The free-field ground motion is typically specified by response spectra in three orthogonal directions (two horizontal and the vertical) anchored to a specified peak acceleration. These response spectra may be site independent, such as those of R. G. 1.60, or, in some cases, site specific. An alternative definition of the amplitude and frequency characteristics of the free-field ground motion is a suite of recorded ground motions judged to be appropriate for the site. For subsequent discussion purposes, let us assume response spectra define the amplitude and frequency characteristics.

Specification of the Control Point and the Spatial Variation of Motion

Once the design spectra are established, they are specified to act at a point denoted the control point. For the broad-band site independent response spectra, such as R.G. 1.60, the control point is most appropriately defined on a free surface -- a surface of the soil or rock outcrop. Specification of the control point at locations other than a free surface, such as within a soil column, can lead to unreasonable motions in the remainder of the soil column and at the surface.

The next aspect of importance to defining the motion is its spatial variation. In terms of SSI, the variation of the motion over the depth and width of the foundation is the key factor. For surface foundations, the variation of motion on the surface of the soil is important. For embedded foundations, the variation of motion over both the embedment depth and the foundation width should be known. Plane waves are normally assumed, which is appropriate considering typical distances from source to site.

The simplest and most often assumed mechanism of wave propagation at the site is vertically incident SH and P waves. For this case, the horizontal motion is transmitted entirely by SH waves and the vertical motion by P waves. All points on the surface of the soil experience identical motion. For a surface-founded structure, the foundation input motion is identical to the free-field ground motion on the surface of the soil. For embedded foundations, the input motion is composed of a horizontal excitation and a vertical translation for a vertical excitation. No torsional component is generated.

Two aspects of the spatial variation of motion can be considered: horizontal variation of motion and variation of motion with depth of soil. In both cases, the phenomenon which causes the foundation input motion to be different from the free-field ground motion is the fact that motion in the free-field varies from point-to-point at the same instant in time. Typical nuclear power plant structures have large, stiff foundations which effectively filter the point-to-point motion and are excited by its resultant.

<u>Horizontal variation of motion</u> -- Nonvertically incident waves lead to variation in motion over horizontal planes. Their effects can be visualized for a surface foundation. Translational motions are, in general, filtered and, hence, their frequency content is changed. An additional rotational component of motion is introduced. For example, nonvertically incident SH waves will cause horizontal translation and an induced torsional rotation of the foundation. Nonvertically incident P and SV waves will cause a vertical translation and an induced rocking of the foundation. To properly account for wave pasage effects, one must consider both the translations and the induced

rotations. In general, wave passage results in a reduced translation, partially compensated for by the the induced rotation. The effect of uncertainty due to nonvertically incident waves is adequately taken into account by design procedures such as the specified accidental eccentricity of 5% of structures' plan dimensions. [Smith, et.al., 1981, 1982, Vol. 4].

Variation of motion with depth of soil -- The spatial variation of motion over the depth of the embedded foundation is of importance to the seismic response of structures. This issue is intimately tied to the specification of the control point. It is our judgement that the control point should be located on a free surface of soil or rock. For deep soil sites or rock, the control point should be located at finished grade. For shallow soil sites, the control point may best be located on a rock outcrop, unless the specific frequency characteristics of the shallow soil layer are taken into account in the definition of the control motion. Specifying the control point at locations other than a free surface (e.g., at foundation level) ignores the physics of the problem and the source of data used in developing design ground response spectra. Once the control point and control motion have been established, the variation in free-field motion over the depth and width of the embedded foundation must be determined. As discussed earlier, the free-field ground motion varies from point-to-point at the same instant in time. This point-to-point variation is effectively filtered by the structure foundation, as in the case of horizontal variation of motion. The resultant excitation is, in general, a translation and corresponding rotation. To properly account for this aspect of wave passage, one must consider both the translation and induced rotation. One method to account for uncertainty in the spatial variation of motion with depth is through variation of soil properties. By maintaining a best estimate model of the phenomenon but shifting soil characteristics, the frequency content of the net input motion is shifted through the frequency range of interest.

SOIL PROPERTIES

A second, large source of uncertainties is related to the determination of the soil properties to be used in the analysis. This involves measuring soil properties in the laboratory and relating them to the properties in situ, establishing the soil configuration, determining the variation of soil properties with level of strain, and accounting for nonlinear soil behavior. Christian, et.al. [1980] present a useful summary of available techniques for determining soil properties, discussing, at the same time, some of the uncertainties relevant to SSI. The influence of true, nonlinear soil behavior on SSI has not been investigated to any significant extent. This effect can potentially cause large uncertainties.

In general, the stress-strain behavior of soil is strongly non-linear, anistropic, elasto-plastic, and loading-path-dependent. It is also dependent on previous loading states and the degree of disturbance to be expected during construction. Practically speaking, these effects are not quantifiable in the current state-of-the-art and hence, add to uncertainty in the description of soil stress-strain behavior. The following discussion concentrates on a linear viscoelastic material model (the one most often used to date). In this model, three parameters define soil behavior -- two elastic constants (usually shear modulus and Poisson's ratio) and material damping. One common method of accounting for this uncertainty in soil properties is an explicit soil property variation study during the SSI analysis.

Determination of Soil Properties and Configuration

Field exploration, which relies heavily on boring programs, provides information on the distribution of soils (horizontally and vertically) and produces samples for laboratory analysis. In addition, some dynamic properties can be measured in situ, notably the shear wave velocity (leading to a value for the shear modulus at low strains). The results of laboratory tests must be correlated with these in situ properties for the SSI analysis.

Three parameters define the soil stress-strain behavior for the viscoelastic model -- shear modulus, Poisson's ratio, and material damping. Poisson's ratio is normally assumed constant and independent of strain level; it is determined from laboratory tests. Soil shear modulus is acknowledged to degrade with increasing strain level. A primary objective of laboratory tests relative to SSI is to obtain the shear modulus degradation curve and material damping variation with strain. Cyclic triaxial, resonant column, or cyclic shear tests are performed in the laboratory. Each test yields somewhat different values for soil properties and may apply in different strain ranges. This requires interpretation and combination of results.

Cyclic triaxial and shear tests apply at strains of 10^{-2} percent and above, whereas resonant column tests apply for strains below 10^{-2} percent. At intermediate strain ranges, the results seldom match precisely and some interpolation is required. The conventional way of presenting the results of laboratory tests is in the form of curves of shear modulus and damping ratio versus average strain level. Variability in the data is considerable even for a single site. Once shear modulus degradation curves and material damping vs strain curves have been determined, it is necessary to correlate laboratory-determined low strain shear modulus values (G_{max}) with those in situ. In situ values of shear modulus are estimated by measuring shear wave velocities in the field -- the most common procedure being a "cross-hole" test. Laboratory values of low strain shear modulus are not identical to in-situ values, largely because laboratory samples are, unavoidably, disturbed samples. Laboratory-measured values of shear moduli at low levels of strain are typically smaller than those obtained in the field, often by a factor of two or three. Several procedures are available and used to adjust the shear modulus degradation and material damping curves vs strain level [Roesset, 1980]. Suffice it to say that, whichever procedure is followed, uncertainty is introduced into the process.

Nonlinearities

A one-dimensional linear analysis, based on shear modulus and on damping values that are estimated iteratively, is the simplest and most economical way of accounting for nonlinear soil behavior. The adequacy of this approach, which models the primary nonlinear behavior, has been generally confirmed, but differences from more detailed solutions have also been noted (Christian et.al., p. 28). An analogous two- or three-dimensional equivalent linearization technique, applied to the coupled soil-structure system, models secondary nonlinear behavior and might seem to offer greater accuracy. However, it also introduces additional uncertainties, notably in treating the different components of strain and in approximating three-dimensional states of stress. Neither of these two alternatives reliably estimates stresses or strains in the soil.

Time-domain analyses with nonlinear constitutive equations for the soil are not yet practical. However, continued research on these more exact solutions will provide invaluable insight into the effects of nonlinear soil behavior on SSI and into the validity of the simplified approaches now used. As noted by Christian, et.al. (pp. 34-35), linear analyses cannot adequately account for the sliding of massive structures on soil, partial slope failure, liquefaction, or excitations near 1g.

MODELING THE SOIL-STRUCTURE SYSTEM

Modeling the Structure Foundation

Three aspects of modeling structure foundations are important for the SSI analysis -- geometry, stiffness, and partial embedment. Foundations are typically embedded and structures are partially embedded. For some foundations, all three aspects can require three dimensions for a full description. Even for a relatively simple foundation (such as a containment building), the geometry of the foundation can be complicated; for example, comprising fuel transfer channels, prestress tendon galleries, and piping tunnels. In other cases, the foundation geometry can be extremely complicated. The ability to treat complicated geometries in their full generality is presently limited by cost considerations. Simplifying adds some uncertainty to the responses calculated. The flexibility or rigidity of the foundations poses a similar problem. Most foundations of the type common at a nuclear power station cannot be considered rigid by themselves. However, the cylindrical shell of a typical containment building and the walls of other structures significantly stiffen their foundations. Hence, in many instances, the assumption of a rigid foundation is reasonable when dealing with nuclear power plant structures.

Foundation embedment has a significant effect on SSI. In comparison with a surface foundation, both the foundation input motion and the foundation impedances change for an embedded foundation. Foundation input motion is dependent on the location of the control point and the assumed spatial variation of the motion as discussed earlier. For the control point on the surface of the soil and the assumption of plane waves (vertically or nonvertically incident), the foundation input motion is, in general, less than for a surface foundation, especially in the high-frequency range. Structural response is correspondingly reduced. The effect of embedment on the foundation impedances is generally to increase the real and imaginary parts with increasing embedment depth. The resulting impact on a structure response is a shift in the resonant frequencies of the soil-structure system and a reduction in the amplitude of response.

The two categories of SSI analysis techniques possess different capabilities of modeling structure foundations. Both methods require modeling assumptions to be made for foundations with complicated geometry and significant flexibility.

- In the substructure approach, surface foundations of arbitrary shape may be analyzed; however, embedded foundations are presently limited to simple shapes (axisymmetric). Most analyses using the substructure approach assume a rigid foundation.
- Most direct methods idealize the geometry of the soil-structure system as axisymmetric or two-dimensional plane strain for analysis purposes. In the latter case, slices through the structure-foundation system are analyzed, which requires determination of equivalent two-dimensional foundation dimensions. Flexibility of the foundation in the analysis plane may be included; however, physical aspects (such as shear walls) of the third dimension that serve to stiffen or increase the flexibility of the foundation should be considered. Complicated shapes in the plane of the analysis can be modeled.

In the context of uncertainties in structure response, one should consider the impact of making particular modeling assumptions on the behavior of the soil-structure system.

Modeling the Structure

There are uncertainties in modeling the dynamic behavior of structures, in general. One aspect of structural models is mentioned here (that is, the effect of using simplified structure models in the SSI analysis). The substructure approach typically poses no restrictions on the detail or sophistication of the structural model. The direct method, however, is often performed in two stages. The first stage determines the overall response of the coupled soil-structure system. A second-stage structural analysis is performed to obtain detailed structural response, using the results of the SSI analysis as input. In practice, the structure model used in the first-stage analysis is simplified, representing only the overall dynamic behavior of the structure. For simple structures such as a containment shell, this introduces minimal uncertainty into the process. For complicated structures whose dynamic behavior is not adequately represented by a small number of modes, this can add considerable uncertainty to the prediction of structure response (Maslenikov et. al., 1982; Kausel, 1980). Amplified response is predicted at frequencies associated with fixed-base frequencies of modes not included in the first-stage model. Ignoring the effect is conservative in most cases; in fact, it can be very conservative.

A second aspect of simplified structure models is the development of equivalent two-dimensional plane strain models when performing first-stage SSI analysis that introduces a presently unquantifiable uncertainty into the analysis.

Structure-to-Structure Interaction

During an earthquake, the vibration of one structure can affect the motion of another -- structure-to-structure interaction. It is of potential significance at a nuclear power station because of the small distances that often separate adjacent structures and the large massive structure foundation systems involved. Two characteristics of the structures and foundations affect structure-to-structure interaction -- the relative size of the foundations and the relative mass of the structures. In both cases, the larger of the two affects the smaller. One example (Maslenikov et. al., 1982) demonstrates the magnitude of the effect for the Zion Nuclear Power Station. Differences in peak accelerations of 30 percent and 50 percent in spectral accelerations (2 percent damping) were seen in the less massive structure.

Three points are worth special mention regarding this phenomenon. First, it is a three-dimensional phenomenon. Attempts to analyze it in two dimensions (e.g., by plane strain analyses) introduce uncertainties of unknown magnitude and effect. Second, the effect of structure-to-structure interaction may be overemphasized by linear analysis. Soil behavior in the immediate neighborhood of the structures is likely to be highly nonlinear. This may reduce the effect of the phenomenon. Third, structure-to-structure interaction may increase or decrease response of the structures, depending on the relationship between their dynamic characteristics and the free-field ground motion.

Localized Nonlinear Behavior

Localized nonlinearities arise from separation or debonding between soil and structure and the subsequent closing of gaps or sliding of structures. These phenomena and their effects on structural response are still being studied; however, two predictions have emerged. First, a reduction in the resonant frequencies of the soil-structure system is predicted. Second, analyses that assume linear soil material behavior predict the introduction of high-frequency structure response because of the impact caused when gaps close. This latter effect has been shown to be small, and likely will be reduced further when nonlinear soil behavior is taken into account.

Parametric studies for a range of soil conditions have established that structure forces determined from a linear analysis are conservative when compared with an analysis including the effect of soil-structure separation.

SSI MODELING CONSIDERATIONS

Idealizing the soil-structure system for SSI analysis was discussed in general terms. The effect of specific aspects of SSI on structure response was treated with minimal consideration given to the analysis techniques to be applied. Here, details of solution procedures are discussed. Recall, two categories of SSI analysis tecniques have been identified -- direct methods and substructure approaches.

Direct Methods

Several modeling issues can have significant effects on the results of SSI analyses performed by direct methods. Among these, the most important are the location of the bottom boundary defining a finite solid deposit, in contrast to a half-space; the location and types of lateral boundaries; and the use of two-dimensional rather than three-dimensional models. Note, as discussed earlier, that it is most convenient to interrogate solution procedures in terms of their ability to model force-displacement characteristics of the system. Hence, the following discussion is in terms of foundation impedances.

Location of the Bottom Boundary

Imposing a bottom boundary when no discontinuity physically exists affects both the real and the imaginary parts of the foundation impedances. The real parts increase slightly but, considering the uncertainties in the soil modulus, the effect is not likely to be significant if the boundary depth is larger than two or three foundation radii. The effect on the imaginary parts (radiation damping) is more important. When a finite stratum is considered, either because it represents the physical situation or because an artificial bottom boundary is necessary, there is no radiation damping below the fundamental frequency of the layer. For a half space, on the other hand, there is radiation damping at all frequencies. Inappropriate placement of the bottom boundary can therefore misrepresent the real situation, negating radiation damping when it exists or predicting it when it is not there.

A second effect of a finite soil layer is the appearance of marked oscillations in the frequency dependence of the foundation impedances. These oscillations are very large for elastic media, less significant in the presence of typical internal soil damping.

Lateral Boundaries

To some extent, the location of the lateral boundaries affects the real parts of the impedance functions (although, in general, to a much smaller extent than the bottom boundary). More importantly, it affects the radiation damping above the fundamental frequency of the layer and the variation of the impedance functions with frequency. The question of lateral boundaries is one of cost and computer capacity: By placing any boundary at a sufficient distance, the error in the solution can be kept within a desired tolerance (Roesset, p. 96).

Two-Dimensional Solutions

Much has been written about the validity of using two-dimensional or pseudo-three-dimensional models to reproduce a truly three-dimensional situation; however, no systematic evaluation of the approximation for a range of site conditions and parameter values has been performed. The degree of approximation depends on whether the width of the footing and the thickness of the soil slice are appropriately chosen. For the extreme case of an elastic half space, if the fundamental frequency of the two-dimensional model is low, the error can be large. Roesset (p. 97) reports the results of a study in which the width and thickness were selected so as to match the true impedances for dimensionless frequencies of 0.3 or higher. These results indicate considerable differences between the two-dimensional model and an exact three-dimensional solution, the former being unconservative. In practical cases, when a finite soil layer physically exists or when soil properties increase with depth, discrepancies are likely to be smaller if the parameters are chosen appropriately.

Some preliminary computations using approximate formulas seem advisable to justify a two-dimensional model. However, with some precautions to guarantee the adequacy of the model, it would seem that in many cases a two-dimensional model can provide an adequate solution.

Substructure Methods

Application of the substructure approach involves determining the foundation input motion, foundation impedances, and analyzing the combined soil-structure system. Simplifications are many times applied to each step. Some considerations are discussed here.

The use of simplified solutions for the foundation impedances corresponding to a uniform elastic layer or half space, and the use of the soil properties at a specific depth (typically between 0.5 and 0.75 radii) provide a reasonable approximation, provided the variation of soil properties is smooth. This approximation may not be appropriate for relatively thin layers of soil with very different characteristics.

Using approximate expressions to account for embedment or simple models with frequency-dependent springs, in contrast to a more accurate analysis, may again produce some variations in the values of the impedances but smaller variations in the structural response. Likewise, rectangular foundations (with aspect ratios less than four) and foundations of arbitrary shape can usually be modeled as circular foundations.

SOLUTION DETAILS

An aspect of the analysis that is rarely mentioned relates to details such as the time step of integration for solutions in the time domain, the frequency range and increment when working with Fourier transforms, and the mesh size for discrete models. Inappropriate choices for each can introduce errors in the results.

SUMMARY

The definition of the design earthquake (including its frequency characterisitcs, the types of waves, and the location of the control point) is without doubt the main source of variation in SSI analyses. Almost as important are the uncertainties involved in the estimation of soil properties in situ and their dependence on strain. Variations in the model used for the analysis of the soil-structure system can produce important differences in the results if inconsistent assumptions are made in some steps, or if serious mistakes are committed, say, in locating the bottom boundary or in applying an inappropriate theory to the physical situation.

3.4.3 Value Impact Assessment

The recognition of the uncertainties specified in the proposed revision to SRP Sections 3.7.2.I.4 and 3.7.2.II.4 does not constitute a new requirement, as the current SRP addresses the above uncertainties in various sections. For example, in the current acceptance criteria for SSI analyses (Section 3.7.2.II.4) two types of modeling techniques are required to address several of the above uncertainties. In the current SRP Section 3.7.2.II.9, peak broadening of floor response spectra is required to address, in some measure, uncertainties in the soil properties and SSI analysis. Thus, the list of uncertainties to be addressed in the proposed revision of SRP sections is for clarification and editorial purposes, in the opinion of the NRC staff, and does not require a value/impact analysis.

3.5 Seismic Analysis Methods

An editorial change to SRP Section 3.7.2.II.1(4), dealing with the adequacy of the number of degrees of freedom used in analysis, is made.

3.5.1 Summary of NRC Proposed SRP Changes*

#1. This change specifies that, as an alternative, the number of degrees of freedom may be taken as twice the number of modes and that the adequacy of the number of modes is discussed in subsequent sections of the SRP. Previously, the alternative approach required that the number of degrees-of-freedom be taken as twice the number of modes with frequencies less than 33 cps.

3.5.2 Technical Discussion of Issues

The use of an adequate number of masses or degrees of freedom in dynamic modeling is essential to insure that calculated response quantities are representative of responses anticipated from the actual structure or system being modeled. If the number of masses or degrees of freedom are inadequate, mode shapes, frequencies and response quantities may give misleading and erroneous results.

Current NRC requirements specify that the number of degrees of freedom is considered adequate when additional degrees of freedom do not result in more than a 10% increase in response. Alternately, the number of degrees of freedom may be taken equal to twice the number of modes having frequencies less than 33 cps. The proposed change would modify this last acceptance criteria to allow the number of degrees of freedom to be taken as twice the number of modes used in the analysis. The adequacy of the number of modes used in the analysis is acceptable if inclusion of additional modes does not result in more than a 10% increase in responses.

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

The dynamic modeling of structures and systems is as much an art as it is a science. Given the same structure, no two analysts are likely to model it in exactly the same way.

The details required in a mathematical model to predict the structural dynamic response behavior depend on the complexity of the real structure and the design requirements. The information required from the dynamic analysis is a primary consideration in constructing the mathematical model. If displacements or clearances at specific locations are a concern, then enough joints or degrees of freedom must be included in the model to provide the required information at points of interest. Also, model refinement may be desired or necessary at locations having discontinuities or where force quantities are changing rapidly. For structures such as containment, biological shield, and reactor pedestal structures having continuous mass distributions, enough mass points should be chosen so that the significant vibration modes can be adequately defined and the dynamic response can be accurately predicted. Of course, increasing the number of degrees of freedom in the model will also result in increased computational effort and cost. The analyst is always faced with the task of attempting to balance and optimize the mathematical refinement of the model with economic considerations.

In engineering practice, the lumped-mass beam approach is widely used. The beam is selected such that the significant stiffnesses are properly represented. The approach is quite convenient and straight-forward. Its properties can often be chosen so that its natural frequencies match those of a more refined-3-dimensional finite element model. In many cases for specific structures, the accuracy with the lumped-mass beam approach is dictated by the total number of masses or degrees of freedom chosen. As an example, Bechtel Corp., [1974] studied a cylindrical containment structure with two different methods, i.e., the constant mass method and the constant member length method. It was found, as a rule-of-thumb, that the maximum error in frequency, for a given mode, associated with using the lumped-mass beam model is always within 10 percent so long as the number of masses or degrees of freedom used is at least twice the mode number. However, as the complexity of the structure being modeled increases, the applicability of this simple rule-of-thumb is questionable.

An excellent discussion of seismic analysis methods and structural modeling techniques is given by Healey, et.al., [1980] and Singh, et.al., [1980].

3.5.3 Value/Impact Assessment

No value/impact assessment of this change has been performed since this change is considered editorial in nature by the NRC staff. It is also our judgment, based on current engineering practices, that this change would have no impact on seismic analysis methods nor on structure modeling techniques currently used for the design and analysis of Category I and applicable non-Category I structures and plant equipment.

3.6 Seismic Analysis Methods and Combination of Modal Responses

Proposed changes to SRP Sections 3.7.2.II.1.(5) and 3.7.2.II.7 dealing with methods for combining high frequency mode responses ($\geq$ 33 Hz) with responses from lower frequency modes (< 33 Hz) are discussed and evaluated.

3.6.1 Summary of NRC Proposed SRP Changes*

Changes to the above SRP sections require that adequate consideration be given to responses associated with high frequency modes. Acceptance criteria for consideration of high frequency modes is contained in a new appendix to SRP 3.7.2.II.7.

3.6.2 Technical Discussion of Issues

Background

In a 1979 submittal for the Lawrence Livermore National Laboratory A-40 Program effort [Coats, 1980], Dr. R.P. Kennedy, of Structural Mechanics Associates, demonstrated the inaccuracies associated with the use of the SRSS combination method** for high frequency modes (modes in excess of about 33 Hz).

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

** The SRSS combination method as referred to herein means the conventional square-root-sum-of-squares method as modified for closely-spaced modes.

The SRSS combination of modal responses is based on the premise that peak modal responses are randomly time phased. However, at frequencies approximately ecual to the frequency at which the spectral acceleration, S_a, returns to the peak zero period acceleration, ZPA, or 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. Recent research work has been conducted in this area, the results of which will be summarized in the next section [Hadjian, A.H., 1981; Gupta, A..K., and Cordero, K., 1981; and Gupta, A.K., and Chen, D.C., 1982]. In general, 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, suggested by Kennedy, appears to be the simplest and most accurate one for incorporating responses associated with high frequency modes (beyond about 33 Hz), and is the procedure given in the proposed new appendix to SRP Section 3.7.2.II.7.

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.
2. For each degree-of-freedom (DOF) included in the dynamic analysis, determine the fraction of 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:

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

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, in the direction of the earthquake input motion
$\phi_{m,i}$	is the eignevector value for mode m and DOF i

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

$$K_i = F_i - \bar{\delta} \tag{2}$$

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 this fraction $|K_i|$ exceeds 0.1, one should include the response from modes higher than those included in Step 1.

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. This is equivalent to a 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:

$$P_i = ZPA * M_i * K_i \tag{3}$$

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

Note that for rotational degrees of freedom, F_i, K_i, and $\phi_{m,i}$ have the units of 1/length, M_i is the mass moment of inertia associated with DOF i and P_i is a pseudo-static inertial moment. The structure is then statically analyzed for this set of pseudo-static inertial forces applied at all of the degrees-of-freedom to determine the maximum responses associated with the high frequency modes not included in Step 1.

4. The total combined response to high frequency modes (Step 3) is 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.

Alternately, one can compute modal responses for a sufficient number of modes to ensure that an inclusion of additional modes does not result in more than a 10% <u>increase</u> in responses. 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 total response from the combined higher modes are then combined SRSS with the total response from the combined lower modes.

Recent Research

The publication and dissemination of NUREG/CR-1161 has resulted in new research on the combination of higher frequency modes, including Hadjian [1981], and Gupta [1981, 1982]. This new research has indicated that Kennedy's 1979 recommendations, which are the basis of the NRC proposed SRP changes, did not go far enough. Basically, the problem with the SRSS response combination method and the transition to algebraic summation occurs at frequencies well below that at which the spectral accleration, S_a, returns roughly to the ZPA. Whereas Kennedy illustrated that the SRSS method should not be used at frequencies above 33 Hz for the USNRC R.G. 1.60 spectra, this newer research illustrates that the same problems extend down to lower frequencies as well.

Either the Hadjian or Gupta approach could be incorporated into Kennedy's 1979 recommendations for changes in the SRP. Both approaches incorporate the idea that the total response is made up of two parts consisting of a damped periodic relative response, R^p, and a rigid response, R^r. The total damped periodic relative response, R^p, is obtained by the current SRSS method of combining modal "relative" responses based upon the assumption that the phasing of these "relative" responses are uncorrelated with each other. The total rigid response, R^r, is obtained by algebraic summation of modal "rigid" responses because this rigid portion of total response is all in-phase with the ground motion. In understanding these methods, three frequencies need to be defined:

f^1 = lower frequency below which rigid and damped periodic relative responses are not additive. Below this frequency the separation into rigid modal responses and damped periodic modal responses is unnecessary and the total modal responses can be combined by the SRSS method.**

f^2 = upper frequency above which the separation into damped periodic relative modal response and rigid modal response is unnecessary, and above which the total response should be treated as being in-phase (rigid) and should be combined algebraically.

f^3 = frequency at which spectral acceleration, S_a, roughly returns to the ZPA.

Gupta defines f^1 and f^2 by:

** It should also be noted that the SRSS method is also inaccurate at very low frequencies. This problem is of little importance to stiff nuclear power facilities and is not addressed herein.

$$f^1 = \frac{S_{amax}}{2\pi S_{vmax}} \tag{4}$$

$$f^2 = \text{FLSP} + 8 \text{ Hz} \tag{5}$$

where S_{amax} and S_{vmax} are the maximum spectral acceleration and velocity, respectively, and FLSP is the frequency at the last significant peak in the spectral acceleration response spectrum. The frequency f^1 may be thought of as a corner frequency between the velocity and acceleration response domains. For a given response spectrum, f^1 is uniquely defined. Based on the R.G. 1.60 response spectrum, f^1 is 2.0 Hz at 0.5% damping, 1.7 Hz at 5% damping, and 1.5 Hz at 10% damping. The frequency f^2 is not uniquely defined for most spectra. Given the same spectrum, different users will obtain substantially different estimates of f^2 depending upon what is taken as the last significant peak. Furthermore, the definition of f^2 appears to be very arbitrary. Based upon Kennedy's review of Gupta's results [1982], he recommends that a preferable definition for f^2 would be:

$$f^2 \simeq f^3 \tag{6}$$

The Gupta method is relatively insensitive to the definition of f^2 and the substitution of f^3 for f^2 as indicated by Equation (6) does not reduce the accuracy of his method, but does provide a more unique definition for f^2. With this modification (Equation 6), f^2 would be 33 Hz for the R.G. 1.60 response spectrum. For the real time histories used in Gupta's studies [1982], f^2 would lie between 10 and 25 Hz when defined by Equation 6.

Hadjian indicates that f^1 lies between 2 and 3 Hz for the 1% damped R.G. 1.60 spectrum and arbitrarily assigns an f^1 value of 2.5 Hz. Hadjian does not need to explicitly define an f^2. However, his approach is consistent with f^2 being defined by Equation (6), i.e., f^2 equals 33 Hz for the R.G. 1.60 spectrum.

Hadjian demonstrates that the separation into a relative response component (combined SRSS) and a rigid response component (combined algebraically) is only important for structures which contain multiple (more than one) significant modes with frequencies greater than 10 Hz for the R.G. 1.60 spectrum.

In other words, with the R.G. 1.60 spectrum, for frequencies below 10 Hz, the SRSS modal response combination method is perfectly adequate and these improvements are unnecessary. As Kennedy showed in 1979, above 33 Hz, SRSS is not acceptable and algebraic summation should be used. Between 10 Hz and 33 Hz, a transition zone exists in which a portion of the modal responses should be combined SRSS and a portion should be combined algebraically for the R.G. 1.60 spectra. For other spectra, these transition frequencies would differ somewhat.

Gupta Approach

1. Separate the total individual modal responses, R_i, into a rigid response, R_i^r, and a damped periodic relative response, R_i^p, by:

$$R_i^r = \alpha_i \; R_i \tag{7}$$

$$R_i^p = \sqrt{1 - \alpha_i^2} \;\; R_i \tag{8}$$

$$\text{where } \alpha_i = \frac{\log f_i/f^1}{\log f^2/f^1}, \text{ except } 0 \leq \alpha_i < 1 \tag{9}$$

Thus, at $f_i \leq f^1$, $a_i = 0$, and at $f_i \geq f^2$, $a_i = 1.0$

2. The damped periodic relative modal responses, R_i^p, are computed for modes with frequencies below f^2, and are combined SRSS to obtain the damped periodic relative response, R^p. The rigid modal responses, R_i^r, are computed for modes with frequencies above f^1, and are combined algebraically to obtain the rigid response, R^r. Note that modes with frequencies above f^3 do not have to be computed. Rather, Kennedy's 1979 recommendations can be used to accurately incorporate the effects of all such modes.
3. The total response, R, is obtained by the SRSS combination of R^p and R^r.

Hadjian Approach

1. For modes with frequencies below f^1, the total modal responses are computed using the conventional pseudo spectral acceleration, S_{a_i}. These modal responses are combined by the SRSS method to obtain the total response, R_L, for all modes with frequencies less than f^1.
2. For frequencies above f^1, an "effective relative' spectral acceleration, S'_{ar_i}, is obtained by:

$$S'_{ar_i} = S_{a_i} - (ZPA) \tag{10}$$

which assumes that the relative response is in-phase (additive) with the rigid response. Next, an "effective relative" response is computed for each mode using S'_{ar_i} in lieu of S_{a_i}.

Note that S'_{ar_i} becomes zero at frequency f^3. Thus, only modes up to frequency f^3 need be considered. All modal responses computed in this step are combined by the SRSS method to obtain the damped periodic relative response, R^p, which is based on the assumption that phasing of these relative response modes is uncorrelated.

2. The rigid response, R^r, is computed by Kennedy's 1979 recommendations except that only modes with frequencies below f^1 are used to compute F_i (see Equation 1). The combined rigid response, R^r, for all modes with frequencies above f^1, is obtained from a static analysis using the pseudo-static inertial forces given by Equation (3).
3. The total response, R_H, for all modes with frequencies higher than f^1 is obtained by the absolute sum combination of R^p and R^r. One must use an absolute sum combination of R^p and R^r to be consistent with the in-phase (additive) assumption upon which Equation (10) is based.
4. The higher frequency total response, R_H, and the lower frequency total response, R_L, are combined SRSS under the assumption that responses in these two frequency ranges are uncorrelated.

Comparison of Hadjian and Gupta Approaches

The Hadjian and Gupta approaches can be directly compared by casting the Hadjian approach into the same format as the Gupta approach. There are basically two differences. First, the Hadjian approach is consistent with α_i being defined as:

$$\alpha_i = 0 \text{ for } f_i < f^1$$
$$\alpha_i = \frac{(ZPA)}{S_{a_i}} \text{ for } f_i > f^1 \tag{11}$$

whereas Equation (9) is used to define α_i for the Gupta approach. Secondly, the Hadjian approach assumes in-phase (additive) phasing between the rigid response and the "effective relative" response whereas Gupta assumes uncorrelated phasing. Therefore, in the Hadjian approach:

$$R_i^P = (1 - \alpha_i)\, R_i \tag{12}$$

whereas Equation (8), based on SRSS combination, is used by Gupta to obtain R_i^P. Because of the use of Equation (12) to obtain R_i^p in the Hadjian approach, one must combine the total relative response, R^p, and total rigid response, R^r, by absolute summation. In the Gupta approach, these two response components are combined SRSS to be consistent with Equation (8). These are the only differences in the two approaches.

The Gupta approach appears to lead to slightly better accuracy than the Hadjian approach. The Hadjian approach is slightly easier to use.

The Hadjian approach appears to contain a fundamental inconsistency in its logic. First, it assumes that all "effective relative" modal responses, R_i^p, are in-phase (additive) with the corresponding rigid modal responses, R_i^r. This assumption is the basis for Equation (12). Next, it assumes that all rigid modal responses, R_i^r, are in-phase with each other, which is the basis for algebraic summation of the rigid modal responses, R_i^r, to obtain the total rigid response, R^r. However, it also assumes all "effective relative" modal responses, R_i^p, are uncorrelated with each other, so that they may be combined SRSS to obtain the total "effective relative" response, R^p. It is inconsistent to assume the relative modal responses are uncorrelated with each other (SRSS combination) and yet are in-phase with the rigid modal responses (Equation 12), which are all in-phase with each other (algebraic summation). This fundamental inconsistency could easily be corrected in the Hadjian approach through the use of Equation (6) to define R_i^p and through the SRSS combination of the total relative, R^p, and total rigid, R^r, responses. Equation (11) for defining α_i could be retained. If this change were made, the only differences between the modified-Hadjian approach and the Gupta approach would be the use of Equation (11) to define α_i versus the use of Equation (9) and different definitions for f^1, assuming the Gupta approach adapts Equation (6) to define f^2. Although a thorough study has not been made, it appears that the modified-Hadjian approach would be more accurate than the originally proposed Hadjian approach, and would have essentially equal accuracy with the Gupta approach, which is indicative of the lack of sensitivity associated with the definition of α_i.

For the R.G. 1.60 spectra, it appears that any approach which uses Equations (7) and (8), and defines α_i so as to be less than about 0.6, at frequencies below about 10 Hz, and greater than about 0.8, at frequencies above about 25 Hz, should lead to reasonable results. In other words, below 10 Hz responses should be predominantly SRSS combined and above 25 Hz responses should be predominantly algebraic sum combined. Between 10 and 25 Hz, a transition zone should exist. These frequency ranges are for the R.G. 1.60 spectrum. For other spectra, these frequency ranges would shift somewhat.

Recommendations

Based on the recent studies made by Gupta and Hadjian, it is clear that Kennedy's original 1979 recommendation, on combining high frequency modal responses, were a step in the right direction. Kennedy currently believes that some modifications to his original recommendations are appropriate. To this end, he suggests the following revisions to the SRP are appropriate:

1. The SRP should indicate that the SRSS method of modal response combination is adequate, so long as the structure model does not contain more than one significant mode at a frequency higher than that associated with the highly amplified spectral acceleration response domain (approximately 10 Hz for the R.G. 1.60 spectrum).

2. The SRP should indicate that either the algebraic sum method or the absolute sum method of response combination must be used for modes with frequencies greater than that at which the spectral acceleration roughly returns to the peak-zero period acceleration (ZPA). Kennedy's 1979 recommendations and the proposed NRC Appendix A to SRP Section 3.7.2 accomplish this goal.
3. The SRP should require a gradual transition from the SRSS response combination method to the algebraic sum response combination method over a frequency range. This transition should lead to predominantly SRSS response combination at frequencies below that at which the 5% damped spectral amplification factor drops to about 2.5 (approximately 10 Hz for the R.G. 1.60 spectrum) and predominantly algebraic sum combination at frequencies higher than that at which the 5% damped spectral amplification factor drops below 1.25 (approximately 25 Hz for the R.G. 1.60 spectrum). No single transition method such as the Gupta or Hadjian method should be specified and any reasonable method to define this transition should be allowed.

3.6.3 Value/Impact Assessment

The SRSS response combination method can lead to significantly unconservative computed responses near the base of stiff cantilever structures and near supports for stiff components such as a stiff piping system. This unconservatism only occurs near supports. Away from supports, the SRSS response combination method can lead to significant conservatism. For the R.G. 1.60 spectrum, the SRSS response combination method will tend to underestimate responses near supports for structures which contain more than one significant mode at frequencies exceeding 10 Hz. If only one significant mode exceeds 10 Hz, no problem exists. The problem of underestimation becomes most severe when the structure model contains more than one significant mode at frequencies exceeding 25 Hz for the R.G. 1.60 spectrum. The degree of unconservatism depends upon the importance of these high frequency modes on total response. Generally, the level of unconservatism is negligible and of academic interest only. However, for very stiff structures, such as are sometimes encountered in nuclear plant designs, the level of unconservatism can be severe.

Based on Dr. Kennedy's experience and a review of the Hadjian [1981] and Gupta [1981, 1982] studies, it is judged that under fairly extreme but realistic situations the ratio of SRSS computed to actual responses might be as low as:

Response Quantity	Ratio SRSS-Computed to Actual Response
Acceleration	0.60
Inertial Forces	0.60
Base Shears	0.75
Base Overturning Moments	0.90

These levels of unconservatism would only occur near the supports of structure models which contain more than one significant mode at frequencies above 25 Hz. Note that the unconservatism is most severe for accelerations and inertial forces. The underprediction of base shears and overturning moments is much less, because in these cases the SRSS method leads to overprediction of responses away from the supports and this reduces the unconservatism of base shears and moments at supports.

Actually, an experienced or cautious analyst would catch these levels of unconservatism in the results. This level of unconservatism has been observed, only when the SRSS computed accelerations near supports were less than the ZPA of the support. Any analyst who makes this check would realize an analytical problem existed and would correct for it by adding in static inertial accelerations or would perform a time-history analysis. Thus, it is doubtful that such large unconservatisms would exist in any analysis or design performed by an experienced or cautious analyst using the SRSS method. However, such unconservatism might exist in "cookbook" analyses performed by an analyst who was overly trusting in the accuracy of his computer program.

The above discussion addresses the potential underestimation of responses using the SRSS method. This unconservatism is expected to impact only stiff piping systems and some walls.

The Zion seismic PRA performed by the SSMRP at LLNL has determined that piping systems do not contribute significantly to risk during earthquakes. Also, it is not anticipated that piping systems, in general, are so stiff as to be a point of concern. This leaves only walls as our principal component under investigation.

For Zion, the SSMRP PRA has determined only one wall to have a significant impact on risk, the auxiliary building shear wall. This wall has a number of pipes and control/power cables penetrating it. As a conservative measure, it was assumed that failure of the wall would result in failure of those systems which are dependent on these penetrations. Thus, the wall is an important singleton in some accident sequences.

As stated in the technical discussion, the SRSS prediction of base shear is estimated to be about 25% low in the worst case. Accordingly, if the proper method of mode combination were applied, we might expect wall strength to improve by, at most, about 33%. To measure the impact of such an improvement, we increased the auxiliary building shear wall median strength by 33% and examined the reduction in total risk.

Table 3.4 presents the results of our calculations. The first column, 'EQ Level', lists the earthquake level at which the other variables are calculated. Although a range of rock outcrop accelerations is shown, the other columns present point values.

During the calculation of component responses, 30 random samples are made from the acceleration range shown. Then, 30 responses are calculated using time histories characterized by these 30 samples. These responses provide the data from which a response distribution is constructed. This response distribution is then combined with a fragility curve to obtain a component failure probability. Likewise, the response and fragility curves are used, together with correlation data and random component failure probabilities, to obtain safety system failure and accident sequence probabilities. For a more detailed discussion, see [Bohn, et al, 1983].

The remaining variables are subscripted with either '0' or '1'. A '0' subscript column represents the original value before the component strength was modified. A '1' subscript column represents the post-modification value.

The column 'P-Fail' represents the conditional component failure probability. By 'conditional' we mean the probability <u>does not</u> include the probability of occurrence of an earthquake in the specified range. The column

'P-CM' represents the annual probability of core melt. The column 'MRem' represents the total contribution to risk, in Man-REM/year, from the specified earthquake level. Both the core melt and Man-REM values are unconditional. That is, they do include the annual probability of occurrence of an earthquake in the specified range.

Table 3.4
Effect of Increased Shear Wall
Strength on Total Risk

EQ Level	P-Fail$_0$	P-Fail$_1$	P-CM$_0$	P-CM$_1$	MRem$_0$	MRem$_1$
.06-.10g	0.	0.	3.7E-8	3.7E-8	2.9E-3	2.9E-3
.10-.20g	1.33E-9	1.94E-12	1.8E-8	1.8E-8	1.5E-3	1.5E-3
.20-.32g	6.96E-6	2.09E-8	2.8E-5	2.8E-5	5.3E+0	5.3E+0
.32-.42g	7.43E-4	5.03E-6	1.2E-6	1.2E-6	3.7E+0	3.7E+0
.42-.53g	2.76E-2	1.59E-3	5.2E-7	5.2E-7	1.2E+0	1.2E+0
.53-.69g	8.76E-2	7.22E-3	2.0E-7	1.9E-7	5.9E-1	5.9E-1

P-Fail = conditional probability of Shear Wall collapse
P-CM = annualized probability of core melt due to an earthquake of this level
MRem = total risk in Man-REM/year from an earthquake within the given level

From the table, it is clear that the increased strength reduces the probability of failure of the wall at all levels. However, the smaller failure probability has an insignificant effect on total risk.

Although the auxiliary building shear wall is the most significant component which is relevant to this task, it is not a major contributor to risk. From the table, it can be seen that, even before the wall is strengthened, the conditional failure probability is negligible at the two lowest earthquake levels. Other structural failures exist which dominate. They are the uplifting of the containment basemat and the collapsing of the service water cribhouse roof.

Plants different from Zion may not be subject to these other structural failures. For those plants, the failure of a wall similar to the auxiliary building shear wall at Zion could be a dominant contributor to risk. The severing of electrical and fluid lines and the impacting of debris on adjacent equipment can be important common mode failures of vital safety systems. Thus, strengthening vital walls should be considered an important seismic safety improvement.

The impact of the proposed SRP changes would be to eliminate this possible, but generally unlikely, source of unconservatism in design. The change would make clear the cause of this unconservatism and would eliminate the need for the use of approximate methods, which have been used to correct this deficiency in the SRSS combined response. Once computer programs were modified, the added analytical costs and engineering efforts would be small. Costs associated with computer program modifications would most likely be absorbed by overhead funds. Furthermore, no construction changes in future plants are anticipated as a result of the proposed revisions. In general, seismic shear stresses in reinforced concrete walls are well below allowables and a 33% increase in these stresses would not affect wall design. It is also believed that the maximum increase in the base-of-wall overturning moments of approximately 10% would not lead to any appreciable changes in wall reinforcement.

Estimates received from the NRC staff indicate an additional two man-weeks of effort might be required to review changes in analysis resulting from the adoption of this proposed new requirement. The total cost increases associated with this requirement are not expected to exceed $5000.

3.7 Methods of Seismic Analysis of Above-Ground Tanks

Proposed changes to SRP Sections 3.7.3.I.14, 3.7.3.II.14, and 3.7 3.III.14, dealing with design requirements for above-ground tanks, are discussed and evaluated.

3.7.1 Summary of NRC Proposed Changes*

1. Changes to the above SRP sections require that dynamic effects and tank flexibility be considered in the analysis of above ground tanks. Specific acceptance criteria are given.

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* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

3.7.2 Technical Discussion of Issues

Background

Although there are many different configurations of liquid storage tanks in use, ground supported circular cylindrical tanks are by far the most common. These tanks have been popular because they are simple in design, efficient in resisting primary hydrostatic pressures, and are easily constructed. Historically, as the number of these tanks increased, their effectiveness in resisting seismically induced loadings became of increasing concern. The actual performance of liquid storage tanks during seismic events indicated their behavior was not being adequately predicted by the relatively simple assumptions used in their design.

The earlier commonly used method of analyzing tanks for seismic response was based on the "Housner-Method", contained in TID-7024 [Holmes and Narver, 1963]. This approach considered the tank to be rigid and focused attention on the dynamic response of the contained fluid. Basically, Housner formulated an idealization for estimating liquid response to seismically induced motions. He divided the hydrodynamic pressure of the contained liquid into two components: (1) The "impulsive" pressure caused by the portion of the liquid accelerating with the tank; and (2) The "convective" pressure caused by the portion of the liquid sloshing in the tank.

The 1964 Alaska earthquake caused the first large scale damage to tanks of modern design, and initiated many investigations into the dynamic characteristics of flexible tanks. These studies generally showed that the seismic response of flexible tanks may be substantially greater than that of a similarly excited rigid tank. More recent evaluation techniques [Veletsos, A.S., and Yang, J.Y., 1976, Veletsos, A.S., 1974, and Haroun, M.A., and Housner, G.W., 1981] have attempted to account for tank flexibility in seismic design, and have indicated that, for typical tank designs, the modal frequency of the 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. Additional studies by Haroun [1982], on unanchored oil storage tanks damaged during the Imperial Valley Earthquake of 1979, indicated that tanks with large "liquid depth-to-radius" ratios frequently suffered structural damage while shell damage in large capacity tanks, which tend to have a large radius and a small depth-to-radius ratio, is less common. Haroun found that overturning moment appeared to have been a critical factor in tank damage during earthquakes. The computation of such moments depends mainly on: (1) The ground acceleration; (2) the assumptions regarding liquid-shell interaction; and (3) the support condition. Experience data as well as analytical studies have demonstrated that tank flexibility can amplify overturning moments considerably.

Current Practice

Despite the growing body of experience data on tank failures, and the increasing number of analytical and experimental studies being made, recent developments in tank design improvements are slow to gain general acceptance into current seismic design codes. This is primarily due to the complexity of computing the dynamic characteristics of tanks. However, some codes have recognized the importance of the effects of wall flexibility and adopted an increase in the maximum ground acceleration to an "ad hoc" value representing the short period amplified acceleration due to shell deformation.

Since the proposed changes to the SRP deal with modifications to the design of above-ground Category I and safety related tanks for Nuclear Power Plants, we conducted an informal survey of major tank fabricators and designers to assess the methods currently used to design such tanks for seismically induced forces. This survey included both steel tanks and reinforced concrete tanks. Survey results are summarized in Table 3.5.

A total of fourteen organizations responded to our survey which included A/E firms, reactor vessel vendors, and tank fabricators. Responses from the following organizations are included in Table 3.5.

- NuTech - San Jose, CA.
- Bechtel Power Corporation - San Francisco, CA.
- Chicago Bridge and Iron Co. - Chicago, IL.
- Stone & Webster - Boston, MA.
- Westinghouse Nuclear Division - Pittsburgh, PA.
- Standard Oil Co. - San Francisco, CA.

- Pittsburgh Des Moines Steel Co. - Des Moines, IA.
- Buffalo Tank - Buffalo, N.Y.
- Combustion Engineering - Windsor, CT.
- Nooter Corp. - St. Louis, MO.
- GATX Tank Erection Corp. - Chicago, IL.
- Brown Minneapolis Tank - Minneapolis, MN.
- General Electric - Sunnyvale, CA.
- Richmond Engineering Co. (RECO) - Richmond, VA.

Responses have not been identified by organization in Table 3.5 so that privacy may be respected.

In summary, the results of the survey indicated the following:

- Nearly all organizations contacted said that they do account for tank flexibility in designing steel tanks to resist seismic forces.
- Virtually none of these organizations indicated much experience with the design of reinforced concrete tanks. One firm indicated that concrete tanks are considered rigid, while another indicated flexibility of concrete tanks is considered.
- Many of the firms indicated that they have no idea about the impact on cost due to changes in design from rigid to flexible tanks. The majority of those who would hazard an opinion felt that the increased costs were small.

The results of the survey seem to suport the contention that the proposed changes to the SRP on tank design do not represent changes to the industry, but merely reflect current industry practice. As such, the actual cost impact on future plant design would be small.

Table 3.5 SUMMARY OF TANK SEISMIC DESIGN SURVEY

1. Designer/Manufacturer

   A/E Firm "A"

   Steel Tank

   Most of the design done by tank vendors. API Standard 650 method is most common. This uses quasi-static coefficients for impulsive and convective forces.

   Reinforced Concrete Tank

   No good feeling about R.C. tanks. However, size should govern the method of design. Large radius and low height tanks should be designed as flexible ones.

   Comments

   No idea about impact on cost since vending companies do the design work.

2. Designer/Manufacturer

   A/E Firm "B"

   Steel Tank

   Steel tanks are designed as flexible structures. Less than 20% of water mass is lumped with the cylindrical shell to determine natural frequency and corresponding spectral acceleration . Remaining (about 80%) water mass is assumed to be sloshing at a low frequency of < 1 Hz.

   Reinforced Concrete Tank

   Should be designed as rigid structure.

   Comments

   No idea about cost difference.

Table 3.5 continued

3. Designer/Manufacturer

A/E Firm "C"

Steel Tank

Follows TID-7024 Procedures, which assume the tank to be rigid. The tank frequency is decoupled from the sloshing liquid frequency to calculate forces for design of anchors.

Reinforced Concrete Tank

Not too familiar with design method of R.C. tanks. Nevertheless, due to lack of ductility, should be designed as rigid structure.

Comments

No idea of cost impact.

4. Designer/Manufacturer

A/E Firm and Vendor "D"

Steel Tank

Use Housner-Haroun criteria for design. This method treats the tank as flexible and uses amplified spectral acceleration and sloshing effect. Sloshing force is usually small, if there is enough free board. If the tank is full, the roof needs to be designed for sloshing force.

Reinforced Concrete Tank

No experience with R. C. design, so no comment.

Comments

Engineering cost does not vary with the method of design. Construction cost will be affected in case of small tank. The tanks in nuclear power industry come under this category. In large tank (100 dia x 100 height), the cost does not vary much. For small tanks, if design acceleration is 1g or less, costs will not be affected. If design acceleration is greater than 1g, then tank construction costs will increase due to additional anchorage requirements and increased wall thicknesses. No estimate on magnitude of cost increase given.

Table 3.5 continued

5. Designer/Manufacturer

   A/E Firm "E"

   Steel Tank

   API Standard 650 method is used for design. The latter uses equivalent static load coefficients for seismic forces, which effectively account for tank flexibility.

   Reinforced Concrete Tank

   Concrete tanks not designed.

   Comments

   There is no appreciable increase in the engineering cost due to revised API Standard. Fabrication cost goes up slightly.

6. Designer/Manufacturer

   A/E Firm "F"

   Steel Tank

   Primarily, ASME Code Section 3 is used. It does take into consideration the sloshing effect and amplified spectral acceleration using flexible design approach.

   Reinforced Concrete Tank

   Do not deal with concrete tanks.

   Comments

   There is not much difference in engineering or fabrication cost between the old "rigid" tank design and the new "flexible" design approach.

7. Designer/Manufacturer

   Tank Vendor "A"

Table 3.5 continued

Steel Tank

Have not designed steel tanks for nuclear power plants for many years. In the old method for E. Q Zone 4, 35% of total operating load was assumed to be acting at the c.g. as seismic load. This took care of sloshing effect too. For lateral uplift, 25% of operating load was assumed to be acting. The base was designed for 125% of operating load.

Reinforced Concrete Tank

Don't deal with concrete tanks.

Comments

No idea of cost change.

8. Designer/Manufacturer

A/E Firm and Vendor "G"

Steel Tank

Since 1979 using API Standard 650 which assumes flexible tanks.

Reinforced Concrete Tank

Don't deal with R. C. tanks.

Comments

Engineering cost increase is minimal (4-5 extra man hours). Fabrication and erection cost goes up with flexible tank assumption, but no idea of percentage rise.

9. Designer/Manufacturer

A/E Firm "H"

Steel Tank

Safety related tanks designed per ASME Code Section 3, Class III. Sloshing effect has been taken into design consideration since mid or late 60's. Since last decade, no order received from nuclear power industry.

Table 3.5 continued

Reinforced Concrete Tank

No experience

Comments

No idea of cost impact.

10. Designer/Manufacturer

A/E Firm "I"

Steel Tank

Use Mc Auto STRUDL code to design tanks. It does take into consideration lumping the liquid and tank to calculate natural frequency. Corresponding spectral acceleration is used to calculate seismic forces. The code also considers sloshing effect of liquid.

Reinforced Concrete Tank

No experience.

Comments

Due to new method, which came into effect in 1978, the engineering cost has doubled. Fabrication cost has not altered much, except that cost of anchoring goes up.

11. Design/Manufacturer

Tank Vendor "B"

Steel Tank

Don't design any tanks for nuclear power industry. For other industries ASME, API, AWWA codes, as specified by the client, are used.

Reinforced Concrete Tank

No idea

Comments

No idea about cost impact

Table 3.5 continued

12. Design/Manufacturer

    Tank Vendor "C"

    Steel Tank

    For petroleum industry, use API Standard 650. For nuclear power industry, use ASME Code Section 3. Sloshing effect has always been considered.

    Reinforced Concrete Tank

    Do not deal with concrete tanks

    Comments

    No idea of cost increase.

13. Designer/Manufacturer

    A/E Firm "J"

    Steel Tank

    ASME Code Sec. 3 is used for category I tanks. API code is used for oil industry.

    Reinforced Concrete Tank

    Don't deal with R. C. tanks.

    Comments

    No change in engineering cost. Cost in fabrication goes up mainly due to Q.A.requirements. The increase is about 20%.

14. Designer/Manufacturer

    A/E Firm "K"

    Steel Tank

    Only indirectly involved in tank design. The latest state-of-the-art is compiled in a draft ASCE report "Fluid Structure Interaction during Seismic Excitation" which recommends flexible tank design similar to Housner-Haroun method.

Table 3.5 continued

Reinforced Concrete Tank

The same analyses method as for steel tanks.

Comments

For flexible tank analysis the analysis cost goes up by 25-30% compared to rigid tank. Fabrication cost should be about 10-15% above the rigid tank design.

3.7.3 Value/Impact Assessment

In the previous section, we indicated that field-erected steel storage tanks have had relatively poor performance in major past earthquakes. The primary reason for this appears to be the underestimation of fluid-induced impulsive forces on the tank, largely due to the once common practice of assuming these tanks to be rigid and thus using the ZPA to compute these impulsive forces. In fact, these tanks typically have impulsive mode frequencies between 2 and 20 Hz, with frequencies in the 4 to 10 Hz range being most common. The use of a spectral acceleration in lieu of the ZPA would result in the computed impulsive forces being increased by a factor of 2.5 to 3.0 for typical cases. These impulsive forces are the largest seismic-induced forces on the tank. As a result, this change would generally increase the total computed seismic forces on these field-erected steel storage tanks by a factor of about 2.0 to 2.5.

Increasing computed seismic forces by a factor of 2.0 to 2.5 would reduce the risk of failure of these tanks in major earthquakes.

For an SSE of 0.3g or less, it is estimated that these changes will result in a slight increase in shell thicknesses throughout the tank height, moderate increases in the shell thickness for the bottom shell course, and moderate increases in the number of hold-down straps or anchor bolts. The increased construction cost should certainly be less than 5% of the total tank cost. Engineering costs would increase about $500 per tank for the additional analysis required. In other words, increased costs would be minor. These shell thickness changes are moderate since current thicknesses are controlled by static loads, not seismic.

With a higher SSE than 0.3g, changes would be more significant and might even result in changes in the overall size and shape of the tank being required. Thus, the cost impact could be significant in these higher seismic regions. However, it is anticipated that the majority of future plant construction will take place predominantly in the eastern United States, where SSE design levels are not anticipated to exceed 0.3g. Thus, considering all future plant construction, increased costs should be minor.

In fact, the actual cost impact on future plant design could be considered to be zero, for all practical purposes, since flexible tank design considerations are already being implemented by major designers and fabricators. Thus, future tank design would incorporate the essence of the proposed SRP change whether or not these changes are implemented.

There are many tanks in a typical nuclear power plant. These tanks are required for storing liquids and gases. Most of these tanks play little if any role in the safety systems. However, some tanks are essential to the successful operation of major safety systems. Some examples of these essential tanks are: diesel generator fuel oil tanks; refueling water storage tanks (RWST); boron injection tanks; condensate storage tanks; accumulator tanks; pressurized nitrogen tanks; and city water tanks. Failures of these tanks can seriously jeopardize, if not completely defeat, the successful operation of one or more safety systems.

The extent to which a tank failure impacts safety depends on many factors: the function of the system containing the tank (e.g., Chemical and Volume Control as opposed to Safety Injection); the redundancy in number of a certain tank (e.g., only one diesel generator fuel tank as opposed to three); the existence of backup systems available to offset loss of a particular safety system (e.g., the use of the Condensate Transfer Pumps to offset the loss of the Auxiliary Feedwater system); and so on.

Among the various tanks essential to plant safety, only some fall into the category being investigated as part of this task; particularly, above ground fluid containing vertical tanks.

SECONDARY CONDENSATE STORAGE TANK

In order to quantitatively estimate the impact of an increase in tank strengths on the total risk from a nuclear power plant, we made use of the seismic PRA performed for the Zion Nuclear Power Plant as part of the SSMRP at LLNL [Bohn, et al, 1983]. In that study, the only tank which made any significant contribution to risk was the secondary condensate storage tank (SCST), which is part of the power conversion and auxiliary feedwater systems (AFWS). (The refueling water storage tank at Zion is part of the structure of the auxiliary-fuel-turbine building and is not relevant to this task.)

The SCST is used as a reservoir for steam turbine condensate, which is normally returned to the steam generators as feedwater. In the event of failure of the main feedwater system, the auxiliary feedwater system is used to feed the steam generators. The auxiliary feedwater pumps draw suction from the SCST. If the tank supply is not available, the pumps will automatically draw from an alternate source, the service water system.

As stated above, the use of a spectral acceleration in lieu of the ZPA, when computing the impulsive forces on a tank, is expected to increase the total computed seismic forces by a factor of 2.0 - 2.5. Accordingly, if this method of calculation were used in the tank design stage, we would expect the tank median strength to be higher by a corresponding amount. This, of course, assumes that the seismic forces are the predominant forces applied during the design analysis.

We increased the median strength of the secondary condensate storage tank by 150%. Table 3.6 presents the results of our calculations.

The first column, 'EQ Level', lists the earthquake level at which the other variables are calculated. Although a range of rock outcrop accelerations is shown, the other columns present point values.

During the calculation of component responses, 30 random samples are made from the acceleration range shown. Then, 30 responses are calculated using time histories characterized by these 30 samples. These responses provide the data from which a response distribution is constructed. This response distribution is then combined with a fragility curve to obtain a component failure probability. Likewise, the response and fragility curves are used, together with correlation data and random component failure probabilities, to obtain safety system failure and accident sequence probabilities. For a more detailed discussion see [Bohn, et al, 1983].

The remaining variables are subscripted with either '0' or '1'. A '0' subscript column represents the original value before the component strength was modified. A '1' subscript column represents the post-modification value.

The column 'P-Fail' represents the conditional component failure probability. By 'conditional' we mean the probability does not include the probability of occurrence of an earthquake in the specified range. The column 'P-CM' represents the annual probability of core melt. The column 'MRem' represents the total contribution to risk, in man-rem/year, from the specified earthquake level. Both the core melt and Man-REM values are unconditional. That is, they do include the annual probability of occurrence of an earthquake in the specified range.

Table 3.6
Effect of Increased Secondary Condensate
Storage Tank Strength on Total Risk

EQ Level	P-Fail$_0$	P-Fail$_1$	P-CM$_0$	P-CM$_1$	MRem$_0$	MRem$_1$
.06-.10g	5.05E-3	2.65E-7	3.7E-8	3.7E-8	2.9E-3	2.9E-3
.10-.20g	1.61E-1	1.05E-3	1.8E-8	1.8E-8	1.5E-3	1.5E-3
.20-.32g	6.43E-1	3.89E-2	2.8E-5	2.8E-5	5.3E+0	5.3E+0
.32-.42g	8.89E-1	1.51E-1	1.2E-6	1.2E-6	3.7E+0	3.7E+0
.42-.53g	9.75E-1	3.38E-1	5.2E-7	5.2E-7	1.2E+0	1.2E+0
.53-.69g	9.81E-1	4.86E-1	2.0E-7	2.0E-7	5.9E-1	5.9E-1

P-Fail = conditional probability of failure of the SCS Tank due to rupture
P-CM = annualized probability of core melt due to an earthquake of this level
MRem = total risk in Man-REM/year from an earthquake within the given level

The results show that, although tank failure probability decreases with the increased strength, the total risk is unaffected.

There are several reasons for the small impact on risk. The first and most significant is that the risk is dominated by two structural failures, the uplifting of the containment basemat and the collapsing of the service water cribhouse roof. The second is the use of a bleed and feed operation to mitigate the loss of auxiliary feedwater. A third reason is the use of the service water system as a backup source of feedwater during a loss of flow from the SCST. In other words, although the SCST was the most significant tank at Zion, it was not significant in the overall risk calculation.

Obviously, the Zion plant cannot be representative of the entire range of plants which will be affected by the Standard Review Plan. Thus, our conclusions regarding the effect of strengthening tanks may not be applicable to all plants. One way to "unfold' some of the specifity of our calculation is to look only at the specific component performance in evaluating the benefit of improving strength.

In Table 3.6, the columns labeled "P-fail" contain the conditional component failure probability for the secondary condensate storage tank. From the table, it is clear that strengthening the tank reduces the component failure probability significantly. Thus, postulating a more significant role in total risk contribution, as may be true at some plants, results in the conclusion that including seismic forces during tank design can appreciably reduce total risk.

REFUELING WATER STORAGE TANK MODIFICATION STUDY

During the value/impact assessment, the only above ground, free standing vertical tank which was studied was the Secondary Condensate Storage Tank (SCST) because it was the only tank which made a significant contribution to total risk in the Zion study. There are other tanks of this type at the Zion site. However, the other tanks either were less crucial to safety or were less likely to fail than the SCST.

The A-40 value/impact study is generic in that it relates to changes to the Standard Review Plan and, therefore, must address issues as they relate to all future plants. The analysis which LLNL made was based on the Zion study, which was limited to one power plant. This plant is typical but does, of course, have some features which are uncommon among other plants. One such feature is the refueling water storage tank (RWST).

The RWST at Zion Unit 1 is part of the structure of the auxiliary-fuel-turbine building. It is very different from an above ground, free standing vertical steel tank. It tends to be much stronger than such tanks and, consequently, exhibited good reliability during the analyses performed in the Zion study. Accordingly, it was not a significant contributor to the total risk identified by the study.

The NRC staff wished to know what the total risk from Zion Unit 1 would be if the RWST were of the above ground, free standing vertical steel tank design. The NRC also wanted to know how sensitive the total risk at Zion was to changes in the strength of such an RWST.

Objectives

The objectives of this study were to determine:

1. the total risk from Zion Unit 1 assuming an above ground, free standing vertical steel tank design for the refueling water storage tank
2. the sensitivity of total risk to changes in the strength of this hypothetical refueling water storage tank

The remainder of this section deals with Objective #1. After completing this step, we felt that it would be inappropriate to continue with Objective #2 at this time. A discussion relating to Objective #2 is given below under Future Effort.

The assessment was made using the Base Case identified in the Zion Seismic Risk Study. The Base Case assumes a capability for performing Bleed and Feed core heat removal. It also includes dominating structural failures, such as containment basemat uplift and collapse of the service water crib house roof.

Modification of Original Zion Model

The functional behavior of a free standing RWST should be identical to that of the actual tank which exists at Zion. Therefore, there were only two modifications made to the Zion model. Since the tank would be free standing, we assigned a new response to the RWST. Secondly, we modified the RWST fragility to reflect the completely different structure of the hypothesized tank.

The response chosen for the free standing RWST was the free-field peak horizontal response at the Zion site.

The hypothesized tank is of the above ground, free standing, vertical steel storage tank design. To arrive at a fragility for this tank, we consulted with Dr. Robert Kennedy of Structural Mechanics Associates. From our discussions with Dr. Kennedy, we concluded that the hypothetical RWST would have a median strength of .42g; with the standard deviation of the logarithm of strength being .3.

Results with Original RWST

The Base Case results using the original RWST are presented in Tables 3.7 and 3.8. Table 3.7 shows the probability of core melt as a function of the release category. Table 3.8 presents the risk to the public as a function of the same variable. The release category definitions are those used in the Zion study, which were in turn taken from the WASH-1400 study [U. S. NRC, 1975].

Notice that most of the core melt probability occurs in release category 7. Upon examining the risk table, however, we find that most of the risk occurs in release categories 2 and 3. This is a result of the fact that categories 2 and 3 are high consequence release categories (see Table 3.9). In contrast, release category 7, is the lowest consequence release category.

Table 3.7
Core Melt Probabilities(yr^{-1})

Release Category	With Original RWST	With Free Standing RWST
1	3.0e-8	2.8e-8
2	4.8e-7	5.1e-7
3	5.0e-7	7.1e-8
4	2.4e-10	1.0e-11
5	9.7e-10	7.0e-11
6	1.6e-7	3.3e-7
7	2.2e-6	4.4e-6
TOTAL	3.6e-6	5.2e-6

Table 3.8
Risk to Public (Man-REM/yr)

Release Category	With Original RWST	With Free Standing RWST
1	1.6e-1	1.5e-1
2	2.4e+0	2.5e+0
3	2.8e+0	3.9e-1
4	5.4e-4	2.6e-5
5	9.7e-4	7.0e-5
6	2.5e-2	5.0e-2
7	5.3e-2	1.0e-1
TOTAL	5.3e+0	3.2e+0

Table 3.9
Public Consequences of
the WASH-1400 Release Categories
(From NUREG-2800)

Release Category	Man-REM / Reactor-Melt
1	5.4E+6
2	4.8E+6
3	5.4E+6
4	2.7E+6
5	1.0E+6
6	1.5E+5
7	2.3E+4

Results with Free Standing RWST

Tables 3.7 and 3.8 present the results of the Base Case involving the hypothesized free standing vertical RWST. In Table 3.7, notice that nearly all of the core melt probability results in release category 7.

Examining Table 3.8 reveals that release category 2 is, by far, the largest contributor to risk of all the release categories; contributing 2.5 Man-REM/yr out of a total of 3.2 Man-REM/yr.

Comparison of Original Base Case with Modified Base Case

Examining the marginal probabilities of failure presented in Table 3.10, it is clear that the above ground, free standing, vertical steel refueling water storage tank is more susceptible to failure during earthquakes than the actual RWST in use at Zion Unit 1. As expected, we find that the introduction of the weaker tank into the SSMRP model of Zion results in an increase in the likelihood of core melt; from 3.6E-6/yr to 5.2E-6/yr. However, we find, somewhat unexpectedly, that the total risk to the public is actually reduced by this modification. This is an artifact of the release category assumptions made. In order to better understand this phenomenon, we must examine the major contributing terminal event sequences for both cases.

Table 3.10.

Marginal Failure Probabilities

Earthquake Level	Original RWST	Free Standing RWST
1	0.	.05
2	1.3E-9	.12
3	7.0E-6	.68
4	7.4E-4	.94
5	2.8E-2	.97
6	8.8E-2	.99

Changing from the original to the free standing RWST, reduces risk by approximately 2.1 Man-REM/yr. Nearly all of this reduction can be accounted for in release category 3. Notice that with the original RWST this value is 2.8 Man-REM/yr. However, with the modified RWST, it is less than .4 Man-REM/yr.

There are three accident sequences which are primary contributors to release category 3 with the original RWST. They are: Small LOCA-21, Small-small LOCA-21, and Medium LOCA-21, all with a DELTA containment failure mode. (The DELTA mode represents containment overpressure due to steam buildup.)

Each of these sequences requires success of the Emergency Coolant Injection (ECI) system. Examining Table 3.10, the free standing RWST has a much higher likelihood of failure than the original RWST. As a result, the probability of success of the ECI system drops greatly, since the RWST is a singleton for the ECI system. This leads to a dramatic reduction in the probability of a core melt in release category 3 with the free standing RWST: from 5.0e-7 to 7.1e-8. This reduction of core melt probability in release category 3 translates into a 2.4 Man-REM/yr drop in risk from release category 3.

The obvious question then is: How does the core melt probability get changed, and why are the consequences so much less severe?

The answer can be seen by examining Table 3.7. For the free standing RWST, we see that the total core melt probability has increased by roughly 1.6e-6/yr. The bulk of this increase occurs within release category 7.

For release category 7, we find that the major contributing terminal event sequences are: Small-small LOCA-13, Class 2 Transient-04, and Small LOCA-13, all with EPSILON containment failure modes. (The EPSILON mode represents containment rupture due to basemat melt-through.) The sequences Small LOCA-13 and Small-small LOCA-13 are characterized by success of the containment cooling functions and failure of Emergency Coolant Injection system. The sequence Class 2 Transient-04 represents a failure of both the Auxiliary Feedwater system and the Bleed and Feed capability along with successful containment cooling.

The probability of each of these sequences is increased by the high probability of failure of the free standing RWST, acting through both the ECI system and the Bleed and Feed capability. The failure of the RWST has a much smaller impact on the containment cooling function because the fan coolers, which do not depend upon the RWST, are considered adequate for removing steam from the containment atmosphere. Since the accident sequences involve success of the containment cooling function, they are characterized by a high probability of containment failure due to basemat melt-through and a low probability of containment failure due to vessel steam explosion.

In order to help understand the results from the SSMRP model, consider the following simplified example:

Assume that there is only one type of initiator, X. Further, assume that there are only two safety systems A and B. In order to prevent an accident, both A <u>and</u> B must perform sequentially. Failure of either system A or system B leads to an accident. The event tree for this simple model is shown in Figure 3.19.

Assume that the consequences of sequence #3 are of some magnitude C, and also that sequence #2 has consequences of magnitude 10C. Then, if A fails with probability .1 and B fails with probability .9, the total risk (assuming that A and B are independent) is:

$$\text{Risk} = P_A * C + P_{(\text{not } A)} * P_B * 10C$$

$$= (.1)C + (.9)(.9)10C = 8.2C$$

Now, if P_A <u>increases</u> to .9, then the total risk becomes:

$$\text{Risk} = (.9)C + (.1)(.9)10C$$

$$= (.9)C + (.9)C = 1.8C$$

Thus, <u>increasing</u> the failure probability of system A, <u>decreases</u> the total risk.

The situation for our Zion model is analogous. Failure to inject coolant leads to an early melt followed by a basemat melt-through. This type of containment failure results in a release which is the least harmful to the public. In contrast, successful injection leads to the potential for unsuccessful decay heat removal and, ultimately, to a containment failure due to overpressure. This type of containment failure has more damaging consequences for the public. Radionuclides, rather than being trapped in the soil, are instead cast into the atmosphere and dispersed. As stated earlier, this is an artifact of the model and may not represent the true situation.

<u>Conclusions</u>

The results presented in Table 3.7 demonstrate that the refueling water storage tank is not a dominant contributor to core melt probability at Zion.

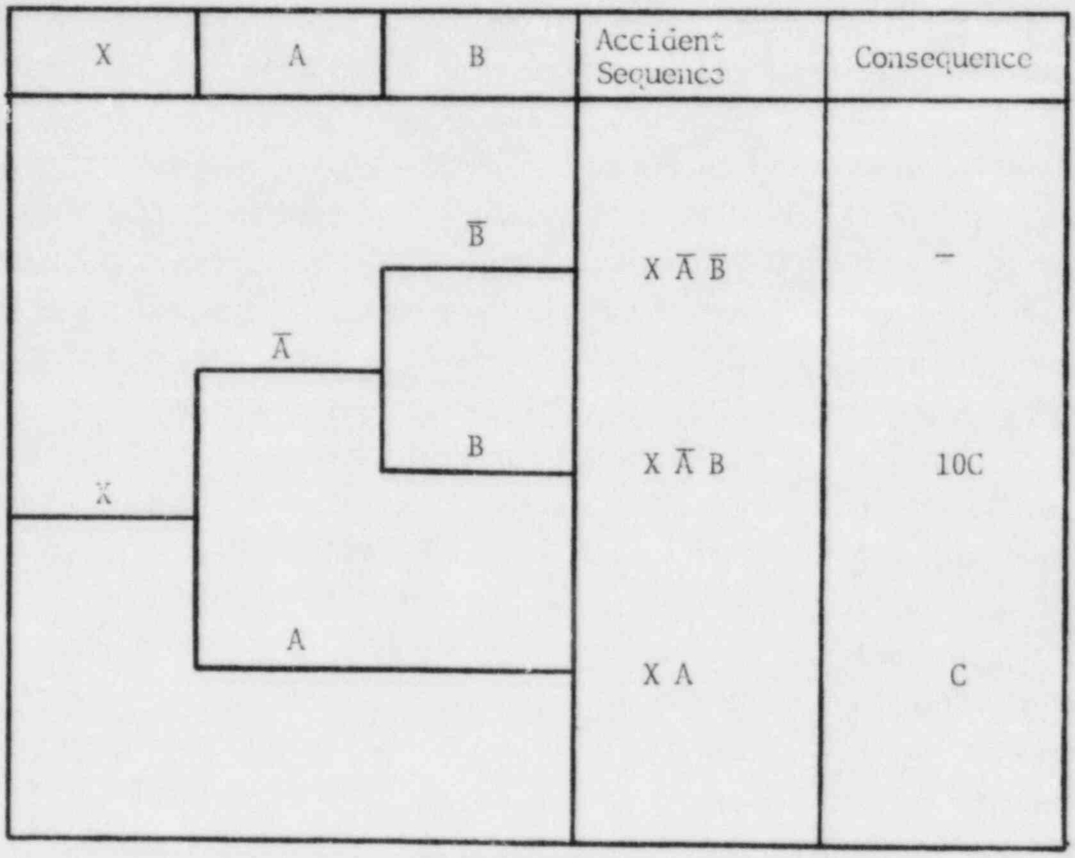

Fig. 3.19 - Event tree for a simplified system.

Clearly, weakening the RWST increases the core melt probability, but not by a significant amount. This result is largely dependent on assumptions regarding major structural failures.

The results relating to risk certainly run counter to expectations. It is not likely that risk decreases as a result of a weakened RWST but the system models and release category assignments in the current Zion model are insufficient to permit a definitive statement on this matter.

In order to illustrate the importance of the containment failure mode and release category assignments, an additional calculation has been made. The containment failure modes for accident sequences Small LOCA - 13, Small-small LOCA - 13, and Class 2 Transient - 04 were changed from a basemat melt-through to rupture due to steam overpressure. Also, release category 7 was replaced in those sequences by release category 2. These sequences were chosen because of their dominance in the case of a free standing RWST.

We recalculated core melt probability and risk at earthquake level 3, both with the original RWST and with the hypothetical free standing RWST. Level 3 was chosen because it is the major contributor to core melt frequency, which is unaffected by the containment failure mode and release category assignments. For earthquake level 3, we found that the original RWST design had a total core melt probability of 1.4e-6 per year, and a total risk of 3.8 Man-REM/yr. Changing to the free standing RWST, the probability of core melt increased, to 3.2e-6. However, because of the new assumptions regarding containment failure and release category, we found that the total risk had increased to 12.1 Man-REM/yr.

This clearly demonstrates that the value/impact of strengthening refueling water storage tanks depends heavily on the assumptions made regarding post-core melt phenomena.

Future Effort

The conclusions based on Man-REM/yr releases are due, in large part, to the assignment of containment failure modes and release categories to each of the accident sequences. Much of what has been used in the Zion model was borrowed from WASH-1400. That study was performed in the early 70's. Since then, much has been learned regarding containment failure modes and radionuclide behavior both during and after a reactor core melt.

As part of the SSMRP validation, a study will be made of the containment failure and release category assignments. This validation may result in changes to the Zion model which could reapportion risk among the terminal event sequences, possibly affecting the conclusions contained in this report.

After the completion of the SSMRP containment consequence model validation, it will be possible to complete Objective #2: to find the sensitivity of total risk to changes in the strength of a hypothetical, free standing refueling water storage tank at Zion.

COST ESTIMATE

Estimates received from NRC staff members indicate an additional review time of two man-weeks might be required to review analysis results associated with the adoption of this proposed new requirement. This probably represents the only real cost impact associated with this requirement since, as previously indicated, current tank design practice already reflects the design approach being proposed. Thus, it is estimated that the total cost associated with the implementation of this proposed new requirement would not exceed $5000.

3.8 Category I Buried Piping, Conduits and Tunnels

Proposed changes to SRP Sections 3.7.3.II.12.(1) and 3.7.3.II.12.(3) are presented and discussed.

3.8.1 Summary of Proposed SRP Changes*

1. Changes to the above SRP sections consist of deleting the existing statement that inertial effects of earthquake loadings on buried systems and tunnels should be accounted for, and recognizing that the real problem is that these structures are subjected to relative displacement-induced

* See Proposed Revision 2 to SRP Sections 3.7.1, 3.7.2, and 3.7.3 for complete text of changes.

strains. The new requirements state that the following loadings must be considered:

a. Ground-shaking-induced loadings
   - Relative deformations produced by passage of seismic waves or by differential deformation between soil and anchor points.
   - Lateral earth pressures acting on structures.

b. Seismic-induced loadings
   - Abrupt differential displacement in zone of earthquake fault breakage.
   - Ground failures such as liquefaction, landsliding, lateral spreading, and settlement.
   - Transient recoverable deformation or shaking of the ground or anchor points relative to the ground.

3.8.2 Technical Discussion of Issues

During an earthquake, permanent ground deformations can be caused by faulting, soil liquefaction, slope instability, ground compaction and lateral spreading. Damage to buried pipelines and systems can be caused by permanent ground movements of this type or by ground-shaking induced loadings. For instance, surface faults, landslides and local compaction of the ground in the 1971 San Fernando earthquake caused the rupture and/or buckling failures of water, gas, and sewage lines [Airman, T., 1983]. Although relatively old and/or corroded pipelines have been damaged by wave propagation [Steinbrugge, K.V., et.al., 1970], seismic ground shaking alone generally cannot be expected to cause any major failures in properly designed, manufactured and laid out welded steel pipelines. This conclusion is in complete agreement with Youd [1973]. After examining the 1971 San Fernando earthquake effects in detail, Youd concluded that strong and ductile steel pipelines withstood ground shaking but were unable to resist the large permanent ground deformations generated by faulting and ground failures. Recent investigations suggest that the most important parameter affecting the performance of an underground pipe crossing a fault is the angle of the pipeline/fault intersection [Eguchi, R.T., et.al., 1981]. Continuous pipelines of constant cross section can provide good resistance to ground shaking and ground failure, if compressional strains are kept below the yield point of the material [Hall and Newmark, 1978 and Kennedy, et.al., 1979]. This can be accomplished by crossing faults at right or oblique angles so that lengthening, rather than compression, results.

Damage observations and research in lifeline earthquake engineering in the Peoples Republic of China [Fu-Lu, 1983], resulting from major earthquakes such as the Haicheng Earthquake of 1975 and the Tangshan Earthquake of 1976, have led to the following conclusions:

1. Damage to pipelines is caused by three major effects, namely: the effect of wave propagation; the effect of tectonic movement including fault movement, landslides etc.; and the effect of nonuniformity or nonhomogeneity of ground soil, including liquefaction.
2. Far away from the fault and landslide zone, the damage occurs least in bedrock, moderately in coarse-grained and firm soils, and most frequently and heavily in fine-grained and soft soils. Furthermore, the damage is maximum in regions of abrupt transition of soil types.
3. Pipelines parallel to the direction of wave propagation are more heavily damaged than those normal to the direction of propagation.
4. Pipelines with rigid joints fail more frequently than those with flexible joints.
5. In relation to pipe size, smaller pipes are more liable to break than larger ones, but this is not the case under certain circumstances. In other words, there are contradictions.

Furthermore, analysis of data from instrumental measurements made during earthquakes has indicated the following response behavior of buried pipelines:

1. Buried pipelines move closely with the ground in both longitudinal and lateral directions during seismic wave propagation.
2. The response behavior of buried pipelines depends largely on the ground displacement characteristics. Axial stress and strain are predominant over the bending ones.
3. Pipelines move with the ground as long as the adhesion/friction between the pipelines and surrounding soil is not lost.

Damage observations of buried pipelines, from the two aforementioned major earthquakes, indicated that pipe joints were key links in pipelines. Rigid joints, or the portions not far from them, were easily broken. They were either pulled out, crushed, bent or sheared off, while flexible joints were seldom damaged. This may be due to the fact that joints are weaker in comparison with the pipe segments, or that the seismic wave responses at joints become more intensive owing to diffraction, reflection and stress concentration [Fu-Lu, 1983].

Good jointing techniques and practices will allow pipelines to change length, rotate or bend without leakage or failure. Backpacking or softened trench techniques, shallow pipe burial above and below ground, and supporting piping above ground can also provide additional measures to prevent pipe deformation and failure during earth movement [Ford, D.B., 1983 and Kennedy, 1979].

One area of concern, for both above-ground and buried piping, is the poor practice of introducing "hard" spots in pipe runs where they enter or are connected to structures in close proximity having independent foundations. This is a relatively common situation and one that exists at the Zion Nuclear Power Plant. Results of the probabilistic risk assessment (PRA) performed on this plant in the SSMRP Phase II report [Bohn, et.al., 1983], indicate that the risk in terms of Man-REM/yr from Release Category 3 (medium LOCA) is due almost entirely to small LOCA sequences, which are caused by the failure of pairs of pipes between the reactor and AFT buildings. These pairs of pipes fail due to differential motion between the buildings. Failure of any one of these pipe pair combinations causes failure of both emergency core injection and the RHR system. Approximately 30% of the total risk (2.7 Man-REM/yr out of 9.6 Man-REM/yr) is due to failures of pairs of pipes between the reactor and AFT buildings, for the base-case Zion risk analysis.

This emphasizes the importance of applying sound engineering judgment and practices in the design of underground piping and utilities. Specific design criteria for the design of long, buried structures, continuously supported by surrounding soil, and the connection of such structures into buildings or other effective anchor points has been given by Kennedy in his 1979 submittal to NUREG/CR-1161 and is contained in that document. Another useful source of design and research information on buried structures is contained in the collection of papers from the 1983 International Symposium on Lifeline Earthquake Engineering [Airman, 1983], several papers of which have already been referenced.

3.8.3 Value/Impact Assessment

No value/impact assessment is required since the proposed changes to the SRP are considered clarifications of existing NRC criteria.

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13. ABSTRACT (200 words or less)

In October, 1981, the Nuclear Regulatory Commission approved a reorganization that resulted in the establishment of the Committee to Review Generic Requirements (CRGR). The charter for the CRGR requires that written justification accompany all proposed new regulatory requirements submitted to the CRGR for review. At the request of the Nuclear Regulatory Commission's Generic Issues Branch, Lawrence Livermore National Laboratory has provided the required written justification to accompany proposed new regulatory requirements to SRP Sections 3.7.1, 3.7.2, and 3.7.3. These proposed new requirements are the result of technical studies performed, as part of the Unresolved Safety Issues (USI) A-40 program, by LLNL and others. NUREG/CR-1161, "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," by LLNL, provided the technical resolution to USI A-40 and was the basis for the proposed new recommendations. The report contained herein presents a technical evaluation and value/impact assessment of the proposed new requirements.

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