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Recommendations for Resolution of Public Comments on USI A-40, "Seismic Design Criteria"

Office of Nuclear Regulatory Research

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ABSTRACT

In June 1988 the Nuclear Regulatory Commission (NRC) issued for public comment the proposed Revision 2 of the Standard Review Plan (SRP) Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3. Comments were received from six organizations. Brookhaven National Laboratory (BNL) was requested by NRC to provide expert consultation in the seismic and soil-structure interaction areas for the review and resolution of these comments. For this purpose, a panel of consultants was established to assist BNL with the review and evaluation of the public comments. This review was carried out during the period of October 1988 through January 1989. Many of the suggestions given in the public comments were found to be significant and a number of modifications to appropriate SRP sections are recommended. Other public comments were found to have no impact on the proposed Revision 2 of the SRP. Major changes are recommended to the SRP sections dealing with a) Power Spectral Density (PSD) and ground motion requirements and b) soil-structure interaction requirements. This report contains specific recommendations to NRC for resolution of the public comments made on the proposed Revision 2 of the SRP.

EXECUTIVE SUMMARY

In June 1988, the U.S. Nuclear Regulatory Commission (NRC) issued for public review and comment a proposed Revision 2 to the Standard Review Plan (SRP) sections dealing with seismic design criteria (Federal Register, June 1, 1988). These sections are:

Section 2.5.2: Vibratory Ground Motion
Section 3.7.1: Seismic Design Parameters
Section 3.7.2: Seismic System Analysis
Section 3.7.3: Seismic Subsystem Analysis

In response to this, NRC received comments from several organizations. Brookhaven National Laboratory (BNL) was requested to assist the NRC in resolving these public comments. This effort was supported by the Engineering Issues Branch, Division of Safety Issue Resolution of the Office of Nuclear Regulatory Research. As part of this effort, a consulting panel was formed (Dr. R.P. Kennedy, Prof. C.J. Costantino, Prof. M. Shinozuka, Dr. J.D. Stevenson and Prof. A.S. Veletsos) which was headed by Dr. A.J. Philippacopoulos. The review and evaluation of the public comments was initiated during October 1988 and was completed in January 1989.

As a result of this review, BNL and its consultants recommended major changes on the proposed Revision 2 to the SRP sections mentioned above. The recommended changes particularly affect the SRP areas dealing with a) ground motion requirements and b) soil-structure interaction requirements. BNL and its consultants strongly believe that the recommended changes will advance the licensing process in view of the developments in the seismic area over the last two decades and on the other hand they will provide an improved accountability of conservatism in the seismic design review process. In addition, it is strongly recommended that future research in the seismic area focus on a) development of PSD criteria for other than Regulatory Guide 1.60 design spectra and b) investigation of the spatial variation of free-field ground motions.

This report presents recommendations to the Nuclear Regulatory Commission (NRC) for resolution of the public comments on the proposed Revision 2 of the Standard Review Plan (SRP) specific sections mentioned above. In Section 1 we provide background material related to the review of the public comments by BNL and its consultants. In Section 2 we present a summary of the public comments on the proposed Revision 2 of the SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3. In Section 3 we provide an analysis of the pertinent issues and we present the basis of our recommendations. Finally, in Section 4 we present a summary of modifications to pertinent areas of the proposed Revision 2 of the SRP. BNL and its consultants strongly recommend that these modifications be implemented by the NRC.

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ACKNOWLEDGMENTS

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1.0 INTRODUCTION

During the first quarter of 1988, the Nuclear Regulatory Commission (NRC) prepared a Revision 2 to the NUREG-0800 (Ref. 1) Standard Review Plan (SRP) Sections 2.5.2 (Vibratory Ground Motion), 3.7.1 (Seismic Design Parameters), 3.7.2 (Seismic System Analysis) and 3.7.3 (Seismic Subsystem Analysis). The Revision 2 to the SRP was a result of many years' work carried out by the NRC and the nuclear industry on the Unresolved Safety Issues (USI) A-40: "Seismic Design Criteria." The background material related to NRC's efforts for resolving the A-40 issues is described by Shaukat, Chokshi and Anderson in NUREG-1233 (Ref. 2).

In June 1988, the proposed Revision 2 of the above mentioned sections of the SRP was issued by NRC for public review and comments. Around August 1988, comments were received from:

a) Sargent and Lundy Engineers (Ref. 3)
b) Westinghouse Electric Corporation (Ref. 4)
c) Stevenson and Associates (Ref. 5)
d) Duke Power Company (Ref. 6) and
e) General Electric Company (Ref. 7)

In October 1988, additional comments were provided by the Electric Power Research Institute (Ref. 8).

In September 1988, Brookhaven National Laboratory (BNL) as a contractor to the NRC was requested to assist the staff in resolving the public comments from the above six organizations. Specifically the project entitled: "Resolution of Public Comments for USI A-40 - Seismic Design Criteria" was issued to BNL with the following objectives:

1) Provide expert consultation in the seismic and soil-structure interaction areas for the review and resolution of the public comments on USI A-40 "Seismic Design Criteria."

2) Provide recommendations for possible modifications to the proposed revisions of the SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 and,

3) Investigate specific issues related to:

   a) Power Spectral Density (PSD) function, and
   b) Soil-Structure Interaction (SSI).

The above project was sponsored by the Engineering Issues Branch, Division of Safety Issue Resolution of the Office of Nuclear Regulatory Research. The NRC Project Manager was S.K. Shaukat, Engineering Issues Branch, Division of Safety Issue Resolution, Office of Nuclear Regulatory Research. Technical direction has been provided by N. Chokshi, Probabilistic Risk Assessment Branch, Division of Systems Research, Office of Nuclear Regulatory Research.

In order to accomplish the above objectives, a consulting panel was established in October 1988. The activities of the consulting panel were directed by Dr. A.J. Philippacopoulos of BNL. The members of the consulting panel were:

- Prof. C.J. Costantino, City University of New York
- Dr. R.P. Kennedy, Structural Mechanics Consulting, Inc.
- Prof. M. Shinozuka, Princeton University
- Dr. J.D. Stevenson, Stevenson and Associates
- Prof. A.S. Veletsos, Rice University

The responsibility of the consulting panel was to perform a detailed evaluation of all public comments and to draw conclusions with regard to their possible impact on the proposed Revision 2 of the SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3. For those cases in which such an impact was identified, specific recommendations are made for resolving the issue.

The review and evaluation of the public comments received by the six organizations mentioned above, was initiated during October 1988 and was completed on January 1989. The work accomplished during this period went far beyond the expected work requirements under this project. This was due to a major effort which was undertaken in order to resolve several issues associated with the public comments on the Power Spectral Density (PSD) requirement. Prof. M. Shinozuka and Dr. R.P. Kennedy carried out a detailed evaluation of various aspects related to the PSD issue. Numerical data were generated and several alternatives were considered. The results of this work are described in Appendix B. The effort by Prof. M. Shinozuka and Dr. R.P. Kennedy was extremely important in reaching consensus on the PSD issue.

The work conducted under this project for the resolution of public comments on SRP Revision 2 can be categorized into three phases. Phase I of the work reflects the preliminary stage of the review of public comments in which the major issues were identified. Phase II of the work was associated with the main portion of the review from which resolutions were prepared for most of the public comments except those related to the PSD requirement. Finally, Phase III of the work was devoted to efforts for resolving the PSD issue and reaching a consensus on the definition of the target PSD for Reg. Guide 1.60 type spectra. The above three phases of the work under this project were carried out during the period of October 1988 through January 1989. During this period, the consulting panel met twice. Members of the NRC staff attended both meetings.

A kick-off meeting was held at the White Flint North Building in Rockville, MD (October 6, 1988). The purpose of this meeting was to:

- Discuss the objectives of the work for BNL and its consultants.

- Discuss the approach for accomplishing the objectives.

- Give a preliminary assessment of the public comments to NRC.

The second and most significant meeting under this program was held on December 16, 1988 at the Nicholson Lane South Building in Rockville, MD. During this meeting recommendations for resolution of the public comments were presented to NRC. These comments were categorized as follows:

- Comments on Power Spectra Density (PSD) and seismic input requirements.
- Comments associated with proposed limits on various aspects of soil-structure interaction.
- Comments on modal combination and damping requirements.

Following the December 16, 1988 meeting, the work under this project was focused on the following items:

a) Preparation of Consultant Reports.

b) Efforts by M. Shinozuka and R.P. Kennedy to reach a consensus on the PSD issue.

The above two activities were completed by the end of January 1989. Finally, it should be mentioned that BNL and its consultants considered the majority of the public comments to be valid and, in addition, to have made significant impact on the seismic design process. In view of this, a set of modifications to the SRP Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 are recommended. These are presented in Sections 3.0 and 4.0 of this report.

2.0 PUBLIC COMMENTS ON PROPOSED SRP REVISION 2

The following is a selective summary of the public comments received by the NRC on the proposed Revision 2 of the SRP (Refs. 3-8).

2.1 Sargent and Lundy Engineers

- The differences between SSE and OBE should be clarified in the SRP.
- The vertical input should be defined as 2/3 of the horizontal.
- The following comments were made with regards to the PSD requirement:

  a) The 15% requirement in amplitude drop below the target PSD could force unnecessary conservatism.

  b) The target PSD above 10 Hz is questionable.

  c) The parameters defining the target PSD should be further examined in view of actual records.

  d) The frequency window of 0.05 Hz is questionable. A maximum frequency interval of 0.2 Hz with 25 second duration is recommended.

  e) The units of the PSD parameters should be consistent. SRP should state that the proposed PSD is a two-sided one.

  f) Two target PSD's should be specified for the horizontal and vertical analysis respectively.

- The requirement of minimum 5 time histories for multiple time history analysis is too high.
- The use of ASCE Standard 4-86 (Section 3.1.2.2, p. 10) damping requirement which correlates stress levels with damping values is recommended.
- The use of ASCE Standard 4-86 (Section 3.3.1.1, p. 25) definition of rock-like foundations is recommended.
- The requirement of enveloping the SSI results from half-space and finite boundary methods should be deleted.
- SRP should not require a limitation of hysteretic soil damping to 5%.
- Combination of modal responses according to ASCE Standard 4-86 is more appropriate and should be permitted.
- Item b on p. 3.7.2-11 of SRP should be deleted.

- Arbitrary limit on reduction of motion at foundation level should not be imposed.

- Limitation of total soil damping (material plus radiation) is not consistent with actual phenomena.

- The shear modulus and damping should be limited to the values associated with strains which are consistent with those observed during earthquakes.

2.2 Westinghouse Electric Corporation

- Multiple time history analyses are not always needed. The required minimum of 5 sets "is unrealistic and unwarranted."

- PSD requirements will place added burden on the industry. They should not be imposed at this time for various reasons.

- More definitive acceptance criteria should be given with respect to the duration of the seismic input, i.e.,

  a) Minimum strong motion duration of 6 seconds.

  b) Total duration of 10-15 seconds.

  Choice of shorter durations with appropriate justification should be allowed.

- On the subject of high frequency mode combinations Westinghouse pointed out four references related to:

  a) Envelope seismic spectra analyses

  b) Seismic multi-spectra analyses

- On the subject of modal combinations, Westinghouse suggested that the procedures of Reg. Guide 1.92 are over conservative. SRP should be changed to include the algebraic sum method as per NUREG-1061 (Vol. 4).

- Westinghouse agrees with the SRP provision regarding a maximum 40% reduction of the surface free-field motion to the corresponding motion at the foundation level. Westinghouse suggested that this limitation will account for uncertainties due to wave type, angle of incidence and soil non-linearity. Furthermore, it is pointed out that this suggestion is in agreement with provisions given in ASCE Standard 4-86.

- Westinghouse agrees with the recommendation of the Senior Seismic Review Team (SSRT) regarding limits imposed on radiation damping. Specifically, frequency-independent radiation damping obtained from standard formulas should be limited to 75%. When layered soil profiles are of interest then the radiation damping should be the same with that computed with acceptable computer codes.

- Westinghouse suggested that when modal damping is used in SSI calculations in conjunction with modal superposition, the composite modal damping should be limited to 20%. It was further recommend that for higher composite damping, the direct integration method be used.

- Westinghouse recommended that the enveloping of the results of different SSI methods should be dropped.

2.3 Stevenson & Associates

- The proposed Revision 2 of SRP does not reflect the results contained in NUREG-1061 Vols. 1-5 which are specifically related to seismic design of piping.

- The proposed Revision 2 of SRP does not reflect contents of available standards such as:

  a) ASME Boiler and Pressure Vessel Code Section III Appendix N, "Dynamic Analysis Methods."

  b) ANSI/ASCE Standard 1-82, "N-725 Guidelines for Design and Analysis Nuclear Safety Related Earth Structures."

  c) ASCE Standard 4-86, "Seismic Analyses of Safety Related Nuclear Structures."

2.4 Duke Power Company

- Duke Power Company agrees with the use of site-specific spectra. They recommend that certain spectra be allowed for application to a number of sites for consistency with standard power plant design.

- Duke Power Company pointed out that it was not possible to investigate the PSD requirement since one of the references given in the SRP (Ref. 12) was not available at the time of their review.

- Duke power recommended that the backfit analyses for above ground tanks (rigid versus flexible wall assumption) be done using realistic allowable stresses (rather than code allowable) and by considering yielding for worst case type loads.

2.5 General Electric Company

- The 15% acceptance criterion for meeting the target PSD is unrealistic. GE recommended that the computed PSD at the major amplified frequency range of interest should reasonably envelope the target PSD.

- The cumulative PSD may be a more accurate measurement of energy than the conventional PSD.

- The 5% limit for hysteretic soil damping is too low. A 15% limit has been recommended in NUREG/CR-1161.

- Distinction of Alternates 1 and 2 in SSI analysis seems inadequate. Any state-of-the-art analyses should be acceptable provided that major uncertainties are accounted for.

- The vertical ground spectra should be 2/3 of the horizontal over the entire frequency range. This definition is consistent with recommendations of NUREG/CR-1161.

- A realistic limit for the reduction of the free-field with depth should be established by looking into more recorded earthquake data.

- No limit on radiation damping is needed provided that layering effects are properly incorporated into the analysis. When a layered halfspace is represented by a uniform halfspace having average properties then, the radiation damping may be over estimated. This can be improved by using refined methods.

- The following limits on soil moduli are agreeable to GE:

    a) Shear modulus reduction with strain should be limited to 40% of the low-strain value.

    b) Hysteretic damping increase with strain should be limited to 15%. This limit has been proposed in NUREG/CR-1161.

- GE recommended that the requirement of enveloping the results from the two SSI methods be deleted. Instead, any method should be acceptable provided that variations in soil properties are accounted for.

2.6 Electric Power Research Institute

- The OBE should not control the design and should be left with utilities to define.

- Although the use of various alternative approaches are encouraged in the design process, some of the restrictions imposed on the more realistic methods defeat the purpose of their use.

- The definition of the control motion either at the surface or at an outcrop is a major advance in the proposed SRP.

- More definitive guidelines are needed especially for Alternate 2 approach in SSI.

- Guidelines for establishing the importance of high frequency modes are needed.

- The extensive requirements associated with Alternate 2 SSI analysis may defeat the purpose of site-specific analyses.

- In the design of above ground tanks, soil-structure interaction criteria are required.

- The 40% limit on the reduction of free-field at the foundation level is not clear. Some Lotung data show even larger reductions. EPRI will provide specific recommendations on the amount of reduction with depth after completion of ongoing studies dealing with the Lotung data.

- EPRI is currently conducting additional tests (field and laboratory tests) to determine soil properties and their variation with strain in view of the results obtained in the blind predictions with Lotung data. When these efforts are completed EPRI will provide specific comments with regard to limitations on soil damping for SSI analysis.

3.0 PROPOSED RESOLUTIONS OF PUBLIC COMMENTS

The review of public comments was carried out by first examining all public comments contained in Refs. 3-8. Subsequently, it was focused on those comments which were judged to be more important in terms of impact on Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 of the proposed SRP Revision 2. These comments were classified conveniently into the following three categories:

- o Comments on input ground motion requirements.
- o Comments on soil-structure interaction requirements.
- o Comments on other issues.

Discussions and recommendations for their resolution are presented in Sections 3.1, 3.2 and 3.3 respectively of this report. The recommendations given in these sections are products of a) the reviews carried out by the consultants and described in the reports attached here to as Appendices A thru E; b) discussions between BNL and its consultants; and c) meetings between BNL, its consultants and the NRC staff. It should be realized that these recommendations involve some level of judgment resulting from the fact that the current state-of-the-art does not permit a complete resolution of certain issues. It is to be expected that refinements may be justified in these areas based on future research. Therefore, it is recommended that a mechanism be established for reviewing the SRP at some regular intervals (perhaps every five years).

3.1 Input Ground Motion Requirement

3.1.1 Power Spectral Density Requirements

The public comments reflect a strong response with respect to the PSD requirements described in the proposed SRP Revision 2 (SRP Section 3.7.1, Subsection I: Areas of Review, Item 1b: Design Time History, p. 3.7.1-4 and Subsection II: Acceptance Criteria, Item 1b: Design Time History, p. 3.7.1-8 through 3.7.1-11). It is the common understanding in the present review that the NRC's intent for requiring a PSD check on the design time history is to ensure that an adequate power distribution exists in the design time history throughout the frequency range of interest. Prior to implementing PSD requirements into the SRP, the usual procedure was to demonstrate that the design time history produces response spectra which closely match the design response spectra for all damping values employed in the analysis and over the frequency range of interest. The public comments made on the proposed PSD requirement ranged from clarification type to those expressing strong reservations regarding the target PSD function given on page 3.7.1-11 of the SRP Revision 2. Our review of public comments focused particularly into the PSD related ones and an intensive effort was made during the time frame of this review to provide recommendations for possible resolution of this issue. Specific aspects of this review are described in the following subsections.

3.1.1.1 Power Requirements for Design Time Histories

As indicated above, the understanding of the proposed SRP Revision 2 PSD requirement is that it was intended to provide power criteria for the design input time histories used to perform seismic evaluations so that possible power deficiencies are prevented. It should be made clear though at this point that in order to accomplish this objective, the PSD approach is not the only way but perhaps a convenient one. A more practical approach for implementation in the design practice is to provide criteria for preventing potential power deficiency at the response spectrum level. Specifically, another way for identifying lack of power in a design time history is to look at the low damped response spectra produced by this time history. It is realized that a response spectrum does not provide a direct definition of the input power since part of the latter is dissipated in the form of viscous damping which is conventionally employed for computing response spectra. On the other hand, low damped response spectra allow for more accessible information regarding the frequency distribution of the input power thus facilitating exercise of judgment. There is, however, a need for specific criteria.

In order to implement power requirements through response spectra, one needs to define how close the response spectra produced by the time history in question should match the corresponding design response spectra. Specifically, the following items have to be addressed:

- o What is the permissible frequency window for the damping considered?

- o What is the permissible amplitude difference at this window as well as in adjacent frequencies? (Lower power within a frequency window can be picked up by adjacent frequencies.)

In Appendix A it is suggested that possible answers to these questions are:

- o Frequency window: ± 20% centered at any spectral frequency.

- o Allowable differences: maximum 20% by average above the design spectrum within any frequency window and 10% maximum dip below the design spectrum at any frequency.

In Appendix A it is cautioned, however, that although the above seem to be reasonable values, the subject needs further investigation. On the other hand, in Appendix D it is suggested that the above requirements may not be difficult to implement if real time histories are employed to generate spectrum consistent time histories. Finally, in Appendix E, it is recommended that PSD criteria should not be required in the proposed SRP Revision 2 if the following two conditions in terms of response spectra are satisfied:

1. That the design time history satisfies the enveloping criteria for response spectra associated with equipment damping of 2% or less, whether the response spectra used in the analyses are of the broad-banded generic type (such as those of Reg. Guide 1.60) or site-specific.

2. That the enveloping criteria be defined as follows:

   - o no more than five points of the calculated spectrum fall below, and no more than 10% below the target spectrum

   - o the calculated spectrum does not exceed the target spectrum by more than 50% at any frequency

   - o the calculated spectrum lies at or above the target spectrum at all calculated structural frequencies of interest, and

   - o the calculated spectrum satisfies the specific frequency requirements of the current SRP.

In view of the above, it appears that although at this time qualitative descriptions are available, a more quantitative basis is required for implementing a power requirement through the response spectrum approach. It is recommended, however, that the discussions on this subject given in Appendices D and E be also considered by the NRC.

Turning now to a PSD approach for expressing power requirements on the design time history, the following items must be addressed:

a) Form of the target PSD function.

b) Criteria to meet the target PSD.

These two items have been addressed in the PSD requirements of the proposed Revision 2 of the SRP. However, both the target PSD as well as the criteria to meet the target PSD were questioned in the public comments. The following sections provide suggestions for resolution of the public comments on the proposed PSD criteria.

3.1.1.2 PSD Criteria of Proposed SRP Revision 2

The proposed SPR Revision 2 specifies that (p. 3.7.1-11):

"... Further, the computed PSD at no frequency should drop below 15 percent of the target value.

$$S_0(\omega) = S_0 \frac{1 + 4\xi_g^2 (\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\xi_g^2 (\omega/\omega_g)^2}$$

with $S_0 = 1{,}100$ $in^2/secs^3$ (this value corresponds to a peak acceleration of 1g), $\omega_g = 10.66$ rad/sec and $\xi_g = 0.9793$ ...."

The above requirements are based on the preliminary study reported in Ref. 12. In the latter study, the Kanai-Tajimi spectral density function was employed to produce ground acceleration time histories compatible with the Reg. Guide 1.60 design spectra. The response spectra produced by the time histories obtained from the above target PSD satisfy the Reg. Guide 1.60 requirements simply in the sense that they envelop conservatively the corresponding Reg. Guide 1.60 design spectra. This enveloping is associated with relatively large differences from a design standpoint especially at higher spectral frequencies (above 10 cps) where the response spectra produced from the target PSD lie much above the Reg. Guide 1.60 spectra. This may cause the following problem: If one starts with a time history having response spectra which match closely the Reg. Guide 1.60 spectra, then in order to satisfy the proposed target PSD requirements the time history may be forced with unnecessary conservatism beyond that embodied in the design spectra.

Specifically, a design time history which matches closely the design response spectra has to be subsequently modified so that its PSD meets a target PSD which in turn produces response spectra that are excessively conservative as compared to the design response spectra. In this awkward situation, the PSD requirement controls the design time history instead of the design response spectra controlling the time history. The fundamental role of the design response spectra is thus violated. This inconsistency which could force design response spectra is thus violated. This inconsistency which could force unnecessary conservatism is perhaps the main source of reaction in the public comments.

In order to resolve this issue it is recommended that the PSD requirements of the SRP Revision 2 be replaced with minimum PSD requirements. Minimum implies that they preserve the level of conservatism associated with the definition of the design time history through the design response spectra. Ideally, a minimum PSD requirement should reflect the same level of compatibility in terms of design response spectra to that of the design time history to the design response spectra. Practically speaking, a minimum PSD requirement must basically maintain the conservatism associated with design response spectra without artifically imposing additional one. In this context, the design response spectra are still the primary acceptance criteria while the PSD requirement is a secondary one which can be used to guard against unwanted (in terms of response) power dips in the input time history.

3.1.1.3 Minimum PSD Requirements

As part of the present review of public comments on the proposed Revision 2 of the SRP, Kennedy prepared initially the PSD requirement which is described in Appendix A (item 2: Earthquake Ground Motion Power Requirements). This requirement was developed on the basis of observations on the cumulative power spectral density functions of seven time histories (one synthetic of the Reg. Guide 1.60 type and six recorded earthquakes). The numerical results and the comparative plots which are presented in Appendix A demonstrate the consistency of minimum type PSD requirement and point out clearly the need for modifying the proposed Revision 2 to the SRP on this issue. Following this initial work, Kennedy and Shinozuka developed jointly a minimum PSD requirement for Reg. Guide 1.60 spectra (Appendix B). The procedure for developing this requirement is essentially similar to that proposed in Ref. 12 without the use of the Kamai-Tajimi PSD function as a target function. Pertinent details and definitions are presented in Appendix B and are not reproduced here.

The minimum PSD requirements proposed here by Kennedy-Shinozuka appear to be much more consistent than the PSD requirements of the proposed SRP Revision 2 (p. 3.7.1-10 and 3.7.1-11). It is recommended that the PSD requirements of the SRP Revision 2 be replaced with the minimum PSD requirements presented in the Appendix B of this report. This will help greatly in resolving the public comments on this issue.

3.1.1.4 Power Requirements for Multiple Time History Analysis

For the multiple time history seismic analysis option, the following suggestions were made by the consulting panel:

- The PSD provision of the proposed SRP Revision 2 (p. 3.7.1-11) regarding multiple time histories should be retained (Appendix A).

- o The PSD requirement should be applied only when multiple artificial time histories are used. It is not needed when multiple real or modified real ground motion histories are used (Appendix D).

- o The average of the individual PSD's should satisfy the target PSD (Appendix E).

It is logical to assume that the risk of missing power in the design input decreases with increasing number of time histories. Generally speaking, there is a sense of repetition when imposing a PSD check on a multiple time history seismic analysis. Perhaps, requiring a PSD check only when artificial time histories are employed in the multiple time history analysis could be a reasonable compromise.

3.1.1.5 Concluding Remarks on PSD Issue

First of all, it is recommended that the PSD criteria (target PSD function as well as requirements to meet the target PSD) of the proposed SRP Revision 2 be replaced with the minimum PSD criteria given in Appendix B of this report. The SRP should also clarify that the design response spectra are the primary acceptance criteria while the PSD requirement is a secondary one. Secondly, it is recommended that the following items be considered by the NRC:

- o PSD requirements for other types of generic broad-banded design spectra.

- o PSD requirements for both horizontal and vertical cases should be specified.

- o PSD requirements for site-specific spectra.

- o The purpose of PSD functions in seismic analysis should be clarified. Should PSD representations of input motion be also used in conjunction with other aspects of seismic analysis?

- o The case of implementing power requirements directly at the level of the response spectrum should be further investigated.

3.1.2 Duration of Input Design Time Histories

In the public comments a suggestion was made to have explicit acceptance criteria in the proposed SRP Revision 2 for defining the duration of design time histories. Specifically, Westinghouse suggested the following:

- o Total duration: 10-15 seconds
- o Strong motion duration: 6 seconds (minimum)
- o Acceptance of shorter time histories with proper justification.

Based on our review of this subject, the following recommendation is made:

- o Strong motion duration: Minimum: 6 seconds
  Maximum: 15 seconds
- o Total duration: 10-25 seconds

Shorter or longer durations should also be accepted on a case-by-case basis.

3.1.3 Number of Time Histories for Multiple Time History Analyses

The requirement of a minimum five time histories which is specified in the proposed SRP Revision 2 (Section 3.7.1, p. 3.7.1-11) for the case of multiple time history analysis was questioned in the public comments. Sargent and Lundy suggested that this requirement be reduced to three time histories. On the other hand, Westinghouse suggested that "it is unrealistic, and unwarranted, to use five sets of time histories to perform a seismic analysis."

The recommendations made by the consulting panel on this subject are:

- o Kennedy recommended (Appendix A) that the provisions of the ASCE Standard 4-86 (Ref. 9, Section 2.3.1, p. 7, commentary Section 2.3.1, p. 45) are preferable to the response spectra and minimum number provision of the proposed SRP Revision 2.
- o Veletsos considers the proposed SRP Revision 2 requirement of minimum five time histories as "quite reasonable" while the ASCE Standard 4-86 provision on this matter as "inappropriate" (Appendix D, p. 5). He recommends that the minimum number of time histories may be reduced to four but no less than four.

From an overall prospective, the minimum number of time histories to be used in the multiple time history analyses:

- o Should not be high enough to discourage the use of the multiple time history option.
- o Should not be low enough so that the use of the multiple time history analyses option is unwarranted.

In making a decision on an acceptable minimum number of time histories for the multiple time history option of seismic analysis one needs to further consider how these time histories are required to match the design response spectra. According to the proposed SRP Revision 2, the acceptance criterion is "... if the average (or other appropriate statistical measure such as MSD) response spectra generated from these time histories envelope the design response spectra ." (p. 3.7.1-11). The following clarification with respect to this criterion is suggested (Appendix D):

> If a collection of artificial, real or modified real ground motion histories is used, the response spectra for the individual records need not separately match the design spectrum, but the spectrum for the ensemble of records corresponding to the mean plus one standard deviation (MSD) level of non-exceedance must match it. The response values considered for design in this option must be those associated with the MSD level of non-exceedance. Alternatively, one may initially adjust the intensities of the ground motion records so that the mean of their response spectra matches the design spectrum, and work with the mean values of the resulting responses. In either case, the match should hold over the entire range of frequencies and damping values of interest.

In addition, the PSD requirements to be imposed (if any) for the multiple time history option should be also factored into the decision for a minimum number of time histories. The provisions on the proposed SRP Revision 2 call for a power check based on an average PSD function. As indicated in Section 3.1.1.4, no consensus was reached here with respect to this item. In general, it appears that there is somehow a repetition in approach when imposing PSD requirements for a multiple time history analysis.

In view of the above, it is recommended that the minimum number of time histories required to perform a multiple time history seismic analysis be reduced from five to four. At this time, there is no sufficient basis for further reduction.

3.1.4 Ratio of Vertical to Horizontal Ground Design Response Spectra

The proposed Revision 2 of the SRP Section 3.7.1 has deleted the 2/3 acceptance criteria regarding the definition of the vertical design input from the corresponding horizontal (p. 3.7.1-8 of Ref. 1). Comments received by Sargent and Lundy as well as by General Electric suggest that the 2/3 provision for defining the vertical component of the ground motion be accepted in the SRP. Although the consensus reached from the review of the public comments is in agreement with this suggestion, a limitation of the rule for certain applications is recommended. Specifically, it is judged that the 2/3 acceptance criteria be applicable only to epicentral distances of 10 Km or more. For smaller epicentral distances the vertical component can exceed the horizontal. In such cases the 2/3 provision may lead to unconservative results and should be avoided. Instead, the definition of the vertical component should be subjected to a review on a case-by case basis.

It is recognized that the 2/3 rule for defining the vertical ground design spectra from the corresponding horizontal has been a subject of many discussions in the past. In view of the recent Revision 2 of the SRP Section 2.5.2, however, it is appropriate not to allow for this rule when the horizontal ground response spectrum is defined according to the provisions of Item 1 of Subsection 2.5.2.6 of the SRP. Specifically, if a site-specific approach is employed for deriving the horizontal ground design spectrum then the same process should be employed for deriving the vertical ground design spectrum without resorting to the 2/3 scaling approach.

In summary, the following recommendations are made with respect to the ratio of vertical to horizontal design response spectra.

- The vertical ground design spectrum should be taken as 2/3 of the horizontal over the complete frequency range of interest provided that the epicentral distance of the design earthquake is more than 10 Km. For smaller epicentral distances the definition of the vertical ground design spectrum should be reviewed and accepted on a case-by-case basis.

- The 2/3 scaling rule should not be permitted for cases in which the horizontal ground design spectrum is generated using the site-specific approach described in Item 1 of SRP Subsection 2.5.2.6. In such cases, the same procedure should be followed for generating both the horizontal as well as the corresponding vertical ground design spectra.

3.2 Soil-Structure Interaction Requirements

3.2.1 Justification of Fixed-Base Analysis

The following specifications are given in the proposed SRP Revision 2 concerning the justification for performing a fixed-base analysis:

p. 3.7.2-9

> "For sites where SSI effects are considered insignificant and fixed base analyses of structures are performed, bases and justification for not performing SSI analyses are reviewed on a case-by-case basis."

and

p. 3.7.2-10

> "For structures supported on rock, a fixed base assumption is acceptable. A comparison of interaction frequencies and the fixed base frequencies can be used to justify the fixed base assumption."

In response to request for public comments, Sargent and Lundy suggested that the provisions of the ASCE Standard 4-86 (Ref. 9) could be used as one acceptable basis for justification of a fixed-base analysis. These provisions are:

> 3.3.1.1 Fixed-Base Analysis - A fixed-base support may be assumed in modeling plant structures for seismic response analysis when the site soil conditions are rock-like beneath the foundation. A rock-like foundation is defined by a shear wave velocity of 3,500 ft/sec (1,100 m/sec) or greater at a shear strain of 10^{-3} percent or smaller when considering preloaded soil conditions due to the structure.

The suggestions provided by Sargent and Lundy on this matter are found generally acceptable. Specifically, the following are recommended:

- o The ASCE Standard 4-86 definition of rock-like materials be adopted in the proposed SRP Revision 2.
- o Acceptability of fixed-base assumption should be primarily addressed by comparison of interaction and fixed-base frequencies.
- o Justification of the fixed-base assumption of the ASCE Standard 4-86 be acceptable by the proposed SRP Revision 2 as an option for cases in which the fixed-base structural frequencies are 10 cps or less.

3.2.2 Enveloping Requirement of Alternate 1

The proposed SRP Revision 2 (Section 3.7.2, p. 3.7.2-9) requires the enveloping of the results from the two SSI methods.

- o In the review efforts of Task Action Plan A-40 (Ref. 10) it was recommended that the enveloping of the two methods should not be required.
- o In the SSI Workshop (Ref. 11) it was recommended that this enveloping requirement be dropped.
- o In the public comments by Sargent and Lundy, Westinghouse, General Electric and EPRI, it is recommended that this requirement be deleted from the proposed SRP Revision 2.
- o In the present review of the public comments on the proposed SRP Revision 2, it is unanimously recommended that this requirement be deleted from the proposed SRP Revision 2.

The skepticism in the regulatory community which led years ago to this requirement has been recognized. There is really no longer a need for this requirement.

3.2.3 Variation of Soil Properties for SSI Analysis

Sargent and Lundy suggested that the low strain values mentioned in Item 2, p. 3.7.2-12 of the proposed SRP Revision 2 be defined. This is a valid point and it is further related to Item 4 on the same page, concerning requirements for variations in soil properties for SSI analysis.

Soil properties are usually handled in the SSI analysis by either of the following approaches:

- o The shear stress and the material damping are computed iteratively through the use of appropriate shear modulus (G) versus shear strain (γ) and damping (β) versus shear strain (γ) curves (e.g., SHAKE, FLUSH). In this case a set of such curves are entered into the SSI calculation.
- o The soil is represented as a linear viscoelastic material (i.e., CLASSI or similar solutions which are based on continuum models). In this case, a single set of shear modulus and damping are entered into the analysis (i.e., G, β are taken as constants).

In the second of the above approaches, however, some representative values of (G, β) in terms of shear strain should be employed in the analysis. These values should be defined according to the effective shear strains (taken usually as 65% of corresponding maximum values) obtained in the soil profile through the free-field analysis of the design ground motion. It is recommended that this clarification be made on page 3.7.2-11 of the proposed SRP Revision 2 so that the provisions are not interpreted as not allowing for any reduction of the shear modulus at seismic strain levels (p. 18 Appendix A).

With respect to the variation in soil properties, the following clarifications are given (Appendix E).

- <u>Definition of best estimate, upper and lower bound cases:</u>

  The upper bound shear modulus at low strain can be taken as twice the best-estimate value while the lower bound shear modulus can be defined as one-half this value, provided that this range of variability suitably encompasses the scatter typically found in the field program. The average shear modulus degradation (G/G_{max} vs peak shear strain) and hysteretic damping ratio (D vs peak shear strain) curves, as defined in ASCE Standard 4-86 can be determined from the laboratory testing program, together with typical data available for similar soils. These curves can then be used in the iterative pseudo-linear analyses to determine shear moduli and hysteretic damping ratios compatible with the effective shear strains computed in the free-field for the input seismic motions for all soil layers for each of the three cases of interest. These properties can then be used directly in the SSI computational model.

- <u>Criteria for the lower and upper cases:</u>

  First, the lower bound shear moduli should not be less than the moduli required for an acceptable foundation design, that is, lead to static settlements much greater than considered acceptable for normal foundation design. Secondly, the upper bound shear moduli should not be less than the best estimate shear moduli defined at low strain (G_{max} defined at 10^{-4} percent effective shear strain) for all soils.

3.2.4 Limit on Soil Damping of Hysteretic Type

The Revision 2 of the SRP states that the internal soil damping of the hysteretic type is "not expected to exceed about 5% of critical" (SRP Section 3.7.2, p. 3.7.2-12). Public comments made with respect to this limitation suggest that the value of 5% is too low and should be increased to 15% which was also recommended in Ref. 10. The maximum value of 15% is also found to be acceptable in the present review and it is recommended that the provision 2 on page 3.7.2-12 of the SRP be changed to allow for a 15% limit on the material (hysteretic) soil damping in place of the current 5% requirement.

It is further recommended that a definition of the hysteretic soil damping be provided in the SRP to avoid confusion with regard to the 15% value. According to the published literature on the SSI subject, the material soil damping for hysteretic behavior can be expressed in terms of the specific loss factor $\Delta W/W$ as

$$\beta = \frac{1}{4\pi}\,\frac{\Delta W}{W} = \frac{\omega G}{2G} \qquad \text{(I)}$$

and

$$\tan\delta = \frac{1}{2\pi}\,\frac{\Delta W}{W} = \frac{\omega G'}{G} \qquad \text{(II)}$$

G = shear modulus
G' = shear viscosity
ω = circular frequency
Note that $\tan\delta = 2\beta$

Any of the above two relationships i.e., (I) or (II) can be used in the SRP to define material attenuation relationships for hysteretic soil behavior. It must be made clear, however, that the recommended 15% limit on the hysteretic type soil damping implies that β must be equal or less than 0.15 or $\tan\delta$ must be equal or less than 0.30.

3.2.5 Limit on Reduction of Ground Motion with Embedment

The reduction of ground design motion for embedded structures received special attention in the public comments. Four out of the six organizations which provided comments to NRC on the Revision 2 of the SRP expressed different opinions on this subject. A brief description follows:

- Sargent and Lundy suggested that arbitrary limits on the reduction should not be imposed.

- Westinghouse agrees with provisions 3.3.1.2(b) of the ASCE Standard 4-86 (Ref. 9) which states that:

  "Variation of amplitude and frequencies content with depth may be considered for partially embedded structures. The spectral amplitude of the acceleration response spectra in the free-field at the foundation depth shall be not less than 60% of the corresponding design response spectra at the finish grade in the free-field."

- General Electric suggested that a realistic limit on the allowable reduction should be established by looking into more data.

- EPRI suggested that the limit of 40% reduction of the translational ground motion is not clear. They are currently investigating this issue using the Lotung data and are expecting to provide final recommendations at the completion of the work.

Looking back at the effort under Task Action Plan A-40 for resolving this issue in late seventies, it was made clear at that time that the review team was dealing with a controversial subject and no consensus could be reached (p. 20 of Ref. 10). In the present work this subject was reconsidered in view of the public comments on the proposed Revision 2 of the SRP. The consulting panel conducted a detailed review of the comments made by General Electric, EPRI, Sargent and Lundy and Westinghouse on this subject. Specifically, the following issues were considered:

- o Should a limit on the reduction with embedment be required?

- o If so, then:
  - What is the amount of allowable reduction?
  - What is the form of the reduction?

The following views were expressed by the consulting panel:

- o The spectral amplitude of the acceleration response spectra in the free-field at the foundation depth shall not be less than 60% of the corresponding design response spectra at the finish grade in the free-field. [Section 3.3.1.2(b) of ASCE Standard 4-86]. This recommendation is discussed in Appendix A (p. A-19) of this report.

- o The reduced motion should not be less than 70-75% of the corresponding surface motion and should not be permitted if rotational components are ignored. The reduction should refer to the horizontal component of the foundation input motion. This recommendation is discussed in Appendix D (p. D-11) of this report.

- o A limit on the reduction is not generally needed. If a limit of the reduction is to be imposed, then the reduced motion should be limited to 60% of the design ground motion. This recommendation is discussed in Appendix E (p. E-11) of this report.

It is clear from the above that no consensus among the members of the consulting panel was reached with respect to the reduction of motion with embedment. As indicated previously, a similar conclusion on this subject was also obtained in Ref. 10. There are, however, the following differences:

First, among other options, the case of not limiting the reduction with embedment was considered in the present review. Specifically, it is recommended in Appendix E (p. E-11) that if the kinematic and inertial aspects of the SSI process are properly addressed in the analyses, then there is no need to place a limit on the reduction.

Secondly, while the range of the allowable reduction is the same with that of Ref. 10, specifically 25-40%, the proposed options with respect to the form of the reduction are:

1) The reduction should refer to the difference between the surface motion and the corresponding motion in the free-field at the foundation level.

2) The reduction should refer to the difference between the foundation input motion for a surface supported structure and the corresponding foundation input motion of the embedded structure.

Based on the above, the options proposed here with respect to the form of the reduction are only two as compared to the three cases given in Ref. 10. The case of reduction with respect to the foundation mat was unanimously rejected at the SSI Workshop (Ref. 11).

Perhaps the main source of the continuing confusion on this matter is because we are still having difficulties in expressing this reduction through the direct and the substructure approaches used in the SSI analysis. In the direct approach, one starts with a free-field analysis to define the input at the base of the finite element model of a soil-structure system. Subsequently, this input is applied at the base of this model and the SSI response is computed. On the other hand, in the substructure approach, the concept of foundation input motion is used. The latter is the response of the rigid foundation in absence of the superstructure to the free-field motion. Since, given a design ground motion, the form of the excitation applied to the soil-structure system is different in the two methods, it is logical to require that the form of the allowable reduction be suitable for both methods. It appears that the recommendation given in Appendix A is more suitable for the direct method (it can be also applied to the substructure method) while the recommendation given in Appendix D is more suitable for the substructure method. Allowable reduction criteria expressed in terms of the foundation input motion could not be easily implemented in the direct method, since the foundation input motion is not computed in the latter method. It is, however, implicitly included in the SSI analysis.

Now, if the percentage of the allowable reduction of the translational component of the foundation input motion could be somehow "equivalent" to the percentage of the allowable reduction in the free-field at the foundation level then the puzzle would be solved. This brings up the following question: Is it more appropriate to place a limit on the free-field motion at the foundation level or on the foundation input motion? One may argue that there is a better handle of the subject when dealing with the foundation input motion. The latter is more representative to what actually is seen by the structure and gives very useful information for appraising the SSI effect. On the other hand, the foundation input motion is related to the free field. Specifically, the former is the response of the massless foundation in absence of the superstructure due to the latter. Consequently, if an exercise of judgment is made for limiting the horizontal component of the foundation input motion, this judgment can also be expressed in terms of reduction of the free-field at the foundation level. The above analogy, however, is not that straightforward.

What is happening in between is that in order to compute the foundation input motion certain assumptions have to be made regarding the nature of the seismic waves in the free-field. In several published studies the foundation input motion has been computed for different foundation configurations and wave types. The general effect is that the free-field is basically filtered for wavelengths which are comparable to the foundation geometry (higher dimensionless frequencies). The resulting foundation input motion has translational components which are generally lower than the free-field as well as rotational (rocking and torsional) components. Nonvertically incident P, SV and Rayleigh waves produce a rocking component while SH waves produce a torsional component. Similar results were obtained in recent, more advanced, treatments of this problem using noncoherent motions. The main problem, however, still remains that we do not yet know enough about the combination of wave trains in a real earthquake environment. In order to guard against potential unconservatism due to insufficient knowledge of the precise character of the seismic waves, it is more reasonable to impose a limit on the amount of reduction with respect to the foundation input motion rather than in the free-field at the foundation level. On the other hand, while free-field motions can be directly measured (recorded data are available at depths below the surface), this is not quite clear for the case of foundation input motions.

Finally, if an allowable reduction with embedment is to be specified with respect to either free-field at foundation level or translational component of the foundation input motion, then some clarification should be made in terms of the soil property variation. Specifically, does the reduction refer to the difference of the surface spectra and the envelope (best estimate, lower bound and upper bound) of the free-field spectra at the foundation level? Similarly, does the reduction refer to the difference of the surface spectra and the envelope (best estimate, lower bound and upper bound) of the translational spectra of the foundation input motion? If it is not the envelope or say some average for that matter, then do we require that the allowable reduction be applied to each case (best estimate, lower bound and upper bound)? Whatever the criteria are, however, the level of uncertainty which is addressed through them should be adequately identified. At the present time this is not quite clear.

Recognizing the uncertainties associated with this subject, a reasonable compromize can be made as follows:

- o Reduction of the translational components of the ground motion with embedment should be permitted in SSI analyses provided that the relevant rotational components are accounted for. This is supported by physical considerations of the problem as well as by recorded data.

- o At this time, it is appropriate to impose a limit on the reduction of the ground motion with embedment. This will guard against the uncertainties discussed previously in this section.

- The reduction should refer to the difference between the surface translational motion and the corresponding motion in the free-field at the foundation level. This form of reduction has primarily two advantages: a) the reduction can be conveniently applied to both the direct as well as the substructure methods of SSI analysis and b) the reduction can be directly measured with field data.

- The amount of reduction should be reasonably taken in the range of 30-40%, with the 30% limit being considered as very conservative.

- The reduction should refer to the envelope (best estimate, lower bound and upper bound cases) of the free-field spectra at the foundation level.

In conclusion, the following criteria are recommended at this time with respect to the variation of ground motion with embedment:

The translational components of the free-field motion at the foundation level should not be less than 60% of the corresponding surface motion. This provision should be: a) allowed only when the associated rotational components are accounted for and b) applied in terms of the envelope of the best estimate, lower and upper bound soil property variation cases.

3.2.6 Limit on Modal Composite Damping

Westinghouse suggested that the composite modal damping used in an SSI analysis, which is based on modal superposition be limited to 20%. This suggestion is acceptable and it is recommended that the proposed SRP Revision 2 incorporate the 20 percent limit in Section 3.7.2 as follows:

P. 3.7.2-18 add after "...complex eigenvectors":

- The use of composite modal damping for computing the response of systems with non-classical modes may lead to unconservative results. The composite modal damping used in conjunction with modal SSI analysis should be limited to 20 percent.

- When the composite modal damping exceeds 20 percent, then generally acceptable methods are a) time domain analysis using complex modes/frequencies (complex eigenvalue problem) b) frequency domain analysis or c) direct integration of uncoupled equation of motion.

3.2.7 Alternate 1 and 2 Requirements

During the December 16, 1988 meeting, the Alternate 1 and 2 approaches of SSI analysis, which are described in Section 3.7.2 of the proposed SRP

Revision 2, were further considered in view of the public comments. These alternates were proposed at the SSI Workshop (Ref. 11). Specifically, the SSI analysis procedures were categorized as follows: Alternate 1 which is associated with enveloping requirements and it is based on broad-banded design ground response spectra and Alternate 2 which is associated with detailed state-of-the-art analysis using site-specific ground motion investigations.

Following the SSI Workshop, however, certain changes have been made in the seismological areas of the SRP. Specifically, Section 2.5.2 of the proposed SRP Revision 2 has embodied the general philosophy of the Alternate 1 and 2 criteria into the definition of the vibratory ground motion. Specifically, it appears that the requirements of Alternate 1 are reflected in Section 2.5.2.6 of the proposed SRP Revision 2 through the broad-banded design response spectra (Item 3, p. 2.5.2-13) while the requirements of Alternate 2 are reflected in the same section by the detailed site-specific ground motion investigations (Item 1, p. 2.5.2-12). Consequently, it is no longer necessary to include this distinction in Section 3.7.2.

Based on these observations, it is recommended that the distinction of Alternate 1 and 2 procedures of SSI analysis be deleted from Section 3.7.2 of the proposed SRP Revision 2. This recognizes that alternative ground motion options are to be included in Section 2.5.2 of the SRP.

3.3 Other Issues

3.3.1 Requirements for Modal Combination

In the public comments (Sargent and Lundy, Westinghouse) it was suggested that the acceptance criteria on modal combination of the proposed SRP Revision 2 should allow for algebraic sum method. Specifically, the proposed SRP Revision 2 refers to Reg. Guide 1.92 which in turn does not permit the use of the algebraic sum method.

This suggestion was found unanimously acceptable by the present review of public comments. Specifically, it is concluded that there is adequate basis in support of the algebraic sum method as an acceptable method to perform modal combination. It is further recommended that a resolution of this issue be made by modifying appropriately Reg. Guide 1.92 to reflect the acceptability of the algebraic sum method. The proposed SRP Revision 2 should be issued with the condition that the Reg. Guide 1.92 be revised accordingly.

3.3.2 Correlation of Damping and Stress Levels

The following paragraph was added in the proposed SRP Revision 2 (p. 3.7.1-12):

> "In addition, a demonstration of the correlation between stress levels and damping values will be required and reviewed for compliance with regulatory position C.3 of Reg. Guide 1.61."

Public comments suggest that more reasonable requirements are provided in item 3.1.2.2 of the ASCE Standard 4-86 (p. 10, Ref. 9) which should be used in place of the above.

This suggestion was considered in the present review of public comments and was found acceptable. Accordingly, it is recommended that the provision 3.1.2.2 of ASCE Standard 4-86 be considered in the proposed SRP Revision 2 as acceptable criteria for demonstrating correlation between stress levels and damping values.

3.3.3 Greater Use of Professional Society Consensus Standards

As a result of the review of the public comments, it is strongly recommended that the proposed SRP Revision 2 should make reference to available standards of professional societies and other organizations.

4.0 RECOMMENDATIONS

The recommendations given in Section 3.0 are summarized here for convenience as follows:

(I): Input Ground Motion Requirements

- It is recommended that the PSD criteria (form of target PSD as well as the 15% requirement to meet the target PSD) of the proposed SRP Revision 2 be replaced with the minimum PSD requirements given in the Appendix B of this report. Furthermore, it should be made clear in the SRP that the design response spectra are the primary acceptance criteria while the PSD requirements are secondary.

- It is recommended that the following items be further considered by the NRC:

  - PSD requirements for other types of generic design spectra.

  - PSD requirements for horizontal/vertical components.

  - PSD requirements for site-specific input spectra.

  - Should PSD representations of the seismic input be also used in other aspects of seismic analysis?

- It is recommended that criteria on the duration of input design time histories be implemented in the SRP as follows:

  - Total duration: 10-25 seconds

  - Strong motion duration: Minimum = 6 seconds
Maximum = 15 seconds

  A provision should be made for acceptance of other durations on a case-by-case basis.

- It is recommended that the current SRP requirement of minimum 5 time histories for multiple time history analysis be reduced at the present time to 4. Further reduction should be done only after additional investigation of this subject.

- It is recommended that the vertical design spectra be taken as 2/3 of the corresponding horizontal over the complete frequency of interest. The 2/3 rule should not be permitted in the following cases:

- o Small epicentral distances (e.g., less than 10-15 km).
- o When the horizontal design spectrum is obtained through the site-specific approach given in item 1 of SRP Section 2.5.2.6.

(II): <u>Soil-Structure Interaction Requirements</u>

- o It is recommended that the definition of rock-like materials per ASCE Standard 4-86 be adopted in the SRP.
- o It is recommended that the ASCE Standard 4-86 provision for fixed-base assumption be accepted in the SRP for fixed-base frequencies of 10 cps or less.
- o It is recommended that the enveloping requirement of results from different SSI methods be deleted from the SRP.
- o It is recommended that the following clarifications/criteria be given in the SRP with regard to soil properly variations:
  - o The shear modulus and the soil damping of hysteretic type used in the SSI analysis should be compatible with the effective shear strains (65% of corresponding peak values) associated with the free-field analysis of the design ground motion.
  - o The low strain best estimate shear modulus should be defined at 10^{-4} percent shear strain. The low and upper bound shear moduli at low strain should be defined as half and twice this value respectively.
  - o The lower bound shear moduli should not be less than those required for an acceptable foundation design.
  - o The upper bound shear moduli should not drop at any shear strain below the value of the best estimate at low strain.
- o It is recommended that the current 5% limit on soil damping of hysteretic type be changed to 15%. Furthermore, the ASCE Standard 4-86 definition of hysteretic damping or other equivalent be specified in the SRP.
- o The translational components of the free-field motion at the foundation level should not be less than 60% of the corresponding surface motion. This provision should be: a) allowed only when the associated rotational components

are accounted for and b) applied in terms of the envelope of the best estimate, lower and upper bound soil property variation cases.

- o It is recommended that when modal superposition is used in the SSI analysis, the modal damping be limited to 20%. If the composite modal damping is higher than 20%, then acceptable methods should be: a) time domain analysis based on solution of complex eigenvalue problem b) frequency domain analysis or c) direct integration.

- o It is recommended that alternate 1 and 2 procedures of SSI analysis be deleted from SRP Section 3.7.2 and their design philosophy be associated with the specification of the vibratory ground motion (Section 2.5.2).

(III): Other Issues

- o It is recommended that the algebraic sum method in modal combination be accepted in the SRP by appropriately revising Reg. Guide 1.92 to that effect.

- o It is recommended that the provisions of the ASCE Standard 4-86 on correlation of damping with stress levels be an acceptable procedure in the SRP.

- o It is recommended that the SRP make reference to available standards of professional societies and other organizations.

It should be realized that these recommendations involve some level of judgment resulting from the fact that the current state-of-the-art does not permit a complete resolution of certain issues. It is to be expected that refinements may be justified in these areas based on future research. Therefore it is recommended that a mechanism be established for reviewing the SRP at some regular intervals (perhaps every five years).

Finally, it is strongly recommended that the following subjects be considered by the NRC for future research: a) development of PSD criteria for other than Reg. Guide 1.60 spectra and b) investigation of spatial variations of free-field motions.

5.0 REFERENCES

1. Proposed Revision 2 to Standard Review Plan, Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3, U.S. Nuclear Regulatory Commission (53 FR 20038; June 1, 1988).

2. NUREG-1233, "Regulatory Analyses for USI A-40, Seismic Design Criteria, Draft Report for Comment," S.K. Shaukat, N.C. Chokshi, N.R. Anderson, U.S. Nuclear Regulatory Commission, April 1988.

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4. Westinghouse Comments on the Proposed Resolution for Unresolved Safety Issue (USI) A-40, "Seismic Design Criteria." Letter from W.J. Johnson to R. Baer, NRC dated July 20, 1988.

5. Stevenson & Associates Comments. Letter to R. Baer, NRC dated July 25, 1988.

6. Duke Power Company Comments: NRC Proposed Resolution for USI A-40 "Seismic Design Criteria." Letter from H.B. Tucker to R. Baer, NRC dated July 25, 1988.

7. General Electric Company: Comments on Proposed Revision 2 to SRP Sections 2.5.2, 3.7.1, and 3.7.2, and Comments on Questions Related to the Lotung SSI Experiment. Letter from R. Mitchell to R. Baer, NRC dated August 1, 1988.

8. Electric Power Research Institute: Comments on Proposed Revision 2 to Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 of NUREG-0800, Standard Review Plan, Draft NUREG-1233 and on the Specific Questions Related to the Lotung Soil-Structure Interaction Experiment. Letter from J.C. Stepp to R. Baer, NRC dated October 4, 1988.

9. ASCE Standard 4-86: Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures, ASCE, September 1986.

10. NUREG/CR-1161: "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," D.W. Coats, May 1980.

11. NUREG/CP-0054: "Proceedings of the Workshop on Soil-Structure Interaction," H.L. Graves, A.J. Philippacopoulos, eds., June 1986.

12. NUREG/CR-3509: "Power Spectral Density Functions Compatible with NRC Regulatory Guide 1.60 Response Spectra," M. Shinozuka, T. Machio, E.F. Samaras, June 1988.

APPENDIX A

COMMENTS ON PROPOSED REVISIONS TO STANDARD REVIEW PLAN SEISMIC PROVISIONS

Prepared for

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by

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1. Introduction

Around May 1988, the U.S. Nuclear Regulatory Commission (NRC) issued Proposed Revision 2 to Sections 2.5.2, 3.7.1, 3.7.2, and 3.7.3 of their Standard Review Plan, NUREG-0800 (1) for public review and comment. Prior to August 30, 1988, comments (2) had been received from five organizations (Sargent & Lundy, Westinghouse, Stevenson & Associates, Duke Power Company, and General Electric). As a contractor to the U.S. Nuclear Regulatory Commission, Brookhaven National Laboratory has been requested to assist the NRC in resolving these public comments. As part of this effort, Brookhaven has formed a panel of consultants in the field of seismic analysis and design of nuclear power plants to review these public comments and to recommend resolutions. I am one member of this panel.

I have carefully reviewed each of the public comments contained in Reference (2). The comments are all of excellent quality and each points to areas of the Proposed Revision 2 (1) where improvements should be made. These comments can be broken down into the following topic areas:

- Earthquake Ground Motion Power Requirements--SRP Section 3.7.1 (Seismic Design Parameters), Subsection II (Acceptance Criteria), Item 1b (Design Time History), pages 3.7.1-10 and 11.

- Time History Strong Motion Duration and Time Envelope Function Requirements--SRP Section 2.5.2 (Vibratory Ground Motion), Subsection II (Acceptance Criteria), page 2.5.2-14.

- <u>Ratio of Vertical to Horizontal Ground Motion Requirements</u>-- SRP Section 3.7.1, Subsection II, Item 1a (Design Response Spectra), page 3.7.1-8.

- <u>Multiple Time-History Requirements</u>--SRP Section 3.7.1, Subsection II, Item 1b, page 3.7.1-11.

- <u>Soil-Structure Interaction Requirements</u>--SRP Section 3.7.2 (Seismic System Analysis), Subsection II (Acceptance Criteria), Item 4 (Soil-Structure Interaction), pages 3.7.2-9 through 12.

- <u>Damping Requirements</u>--SRP Section 3.7.1, Subsection II, Item 2 (Percentage of Critical Damping Values), page 3.7.1-12.

- <u>Modal Combination Requirements</u>--SRP Section 3.7.2, Subsection II, Item 7 (Combination of Modal Responses), page 3.7.2-16.

- <u>Greater Use of Professional Society Consensus Standards</u>--General comment on all sections.

Based upon my review of the public comments (2) plus my own considerations, I make specific recommendations for each of the affected sections of the Proposed Revision 2 (1) in the following sections of this brief report.

2. <u>Earthquake Ground Motion Power Requirements</u>

Prior to the proposed revision, the Standard Review Plan (SRP) had no explicit requirements for the design earthquake ground motion power throughout the frequency range of interest. All that was required was that the design ground motion time history produce a response spectrum which essentially envelopes the design earthquake response spectrum at corresponding damping levels. As an extreme example, a 0.6 Hz steady-state single frequency sinusoidal 0.85g ground motion will totally envelope the R.G. 1.60 Spectrum for a 0.2g earthquake. Theoretically, this 0.85g sinusoidal ground motion could be used to generate floor spectra and for equipment design and qualification because it envelopes the required design response spectrum for a 0.2g SSE. However, all nuclear power plant civil structures (2 Hz and higher frequency) would respond in a cyclic pseudo-static manner to such a low frequency sinusoidal input motion, because this input motion has no power in the frequency range of 2 Hz or higher. Thus, there would be no resonant amplification of this input motion by the civil structures, so that equipment mounted in such structures would only be subjected to this input motion without amplification. Floor spectra generated from such an input would be much less than that generated by broad-frequency content earthquake ground motion at the structure's resonant frequencies and at higher frequencies, even though the input ground motion response spectrum enveloped the design earthquake response spectrum at all frequencies. Such an extreme and obvious example would never be allowed in practice, even though it might be argued that it meets the existing Standard Review Plan requirements. However, to a lesser extent, this same reduction in floor spectra occurs even with broad frequency input time histories when such time histories are significantly deficient in power over a frequency band of about ±20% centered on any of the important structure natural frequencies. Thus, for a 7 Hz structure, floor spectra can be severely underestimated when an input motion deficient in power over the 5.6 to 8.4 Hz range is used as input, even though it has excess power at other frequencies so that its response spectrum envelopes the required earthquake design spectrum at all frequencies. Within my experience, this latter situation has occurred in a few instances within the nuclear industry. Therefore, I fully support proposed revisions to the Standard Review Plan which place broad frequency power requirements on design earthquake ground motion time histories.

There are at least two methods to ensure adequate power throughout the frequency range of interest. One method would be to require the input motion time history to produce a low damped (2% damping) response spectrum that closely matches the design response spectrum over the entire frequency range. In this way, excess power over one frequency range would not be allowed to mask a deficiency in power within another frequency range, since the response spectrum in the frequency range of excess power would greatly exceed the design response spectrum. An input time history that produces an input response spectrum which closely matches the design response spectrum at low damping over the entire frequency range from 0.4 to 33 Hz must contain power throughout this frequency range consistent with that of the design response spectrum. However, how close this match must be over the entire frequency range is not clear. Furthermore, the difficulty of achieving a close match at all frequencies has not been fully investigated. Probably it would be sufficient to require that the input motion produce low damped spectral accelerations which do not average more than 20% above the design response spectrum over any ±20% frequency band width (i.e., 4 to 6 Hz band width for 5 Hz) and do not dip more than 10% below the design response spectrum at any frequency (current requirement). However, this requirement may be difficult to meet.

A second approach is to directly define the minimum power requirements as a function of frequency. I prefer this approach because it directly defines the minimum power requirements within various frequency ranges. This approach has been proposed in the revision to the Standard Review Plan in which the average Power-Spectral-Density (PSD), $S_{O(\omega)}$, over any 0.15 Hz frequency band between 0.2 Hz and 34 Hz be at least 85% of the following target value:

$$S_{O(\omega)} = S_O \frac{1 + 3.836\ (\omega/\omega g)^2}{[1 - (\omega/\omega g)^2]^2 + 3.836\ (\omega/\omega g)^2} \tag{2.1}$$

with S_O = 1,100 in^2/sec^3 for a peak acceleration, A, of 1g, and ωg = 10.66 rad/sec. For other peak accelerations, the factor S_O is scaled proportional to A^2. I support the idea of establishing minimum PSD requirements. However, I have several concerns with regard to Equation (2.1).

First, for earthquake time histories, the reported values for a PSD can vary widely depending upon the exact formulation used to compute the PSD. Each of the following three factors must be defined with regard to Equation (2.1):

1. Either a one-sided or a two-sided PSD can be specified. It should be clearly specified that Equation (2.1) represents a <u>one sided PSD</u>.

2. Even specifying that Equation (2.1) is a one-sided PSD is insufficient. Different relationships between the one-sided PSD and Fourier Amplitude $|F_{(\omega)}|$ exist between common textbooks and within existing practice. The one-sided PSD is specified as either

$$S_{O(\omega)} = \frac{2\ |\ F_{(\omega)}\ |^2}{2\pi\ T_D} \qquad (2.2a)$$

or

$$G_{O(\omega)} = \frac{2\ |\ F_{(\omega)}\ |^2}{T_D} \qquad (2.2b)$$

where T_D is the strong motion duration over which $F_{(\omega)}$ is evaluated. The relationship between these two different definitions of the one-sided PSD is

$$S_{O(\omega)} = \frac{G_{O(\omega)}}{2\pi} \qquad (2.2c)$$

Throughout this brief report, I will use the symbols $S_{O(\omega)}$ and $G_{O(\omega)}$ to distinguish between those two definitions. I don't care which definition is used. However, the Standard Review Plan should clearly specify which relationship between PSD and Fourier Amplitude (Equation (2.2a) or Equation (2.2b)) is being used. The coefficient S_O in Equation (2.1) is based upon the PSD being defined by Equation (2.2a). If Equation (2.2b) is preferred, then $S_O = 1,100\ in^2/sec^3$ should be replaced by $G_O = 6,900\ in^2/sec^3$ in Equation (2.1).

3. For earthquake time histories, some people determine the Fourier Amplitude over the entire duration of the record, while others determine the Fourier Amplitude only over the strong motion duration within which the power is near maximum. Whichever duration is used, the same duration should be used in the denominator of Equations (2.2). If the power is nearly stationary, it is irrelevant which duration is used to determine the Fourier Amplitude, so long as this same duration is used in Equations (2.2). However, for most actual records, the power is only stationary over the duration of strong motion, T_D, during which the power is near maximum. This strong motion duration is discussed further in the next section. Over a longer duration, the average power is less. Equation (2.1) was developed so as to be applicable during the time of maximum power. When a PSD is developed from an input motion time history for comparison with Equation (2.1), the actual PSD should be based on using the duration of near maximum power. Otherwise excess conservatism can be introduced by the comparison.

However, even beyond the need for additional clarification, I have other reservations about Equation (2.1). In my opinion, the Design Ground Motion should be primarily defined by the Design Response Spectrum. The PSD requirement is a secondary requirement which is simply used to prevent a severe deficiency of power over any frequency range. The PSD requirement should not be used to add additional conservatism beyond that contained in the Design Response Spectrum. Any ground motion time history which produces a response spectrum that closely fits the Design Response Spectrum should be able to pass the PSD requirement. However, this situation will not be the case with the PSD requirement given by Equation (2.1). Reference (3) presents results for seven artificial time histories (Nos. 1-3, 6, and 8-10) which have PSD levels similar to those expressed by Equation (2.1). Figure 1 shows a representative example PSD from one of these time histories (jagged line) versus the Equation (2.1) PSD requirement (smooth solid line). Figure 2 shows the response spectrum from this PSD versus the R.G. 1.60 Response Spectrum. In every case, the PSDs fall below Equation (2.1) and appear to average about 90% of the required PSD below about 6 Hz and even less at higher frequencies. Even so, the resultant response spectra are consistently higher than the R.G. 1.60 Response Spectrum. Below 6 Hz, the exceedance appears to average about 20% and is much greater at high frequencies (about 70% at 30 Hz). Thus, the Equation (2.1) PSD requirement will add additional conservatism beyond that contained in the R.G. 1.60 Spectrum, particularly at higher frequencies.

Reference (4) studied the engineering characterization of ground motion. It concluded that the Cumulative PSD as defined by:

$$\text{Cum } G_{O(\omega)} = \int_{O}^{\omega} G_{O(\omega)} \, d\omega \tag{2.3}$$

is an important descriptor of the ground motion. In particular, if one defines f_{10}, f_{50}, and f_{90} as the frequencies below which 10%, 50%, and 90% of the cumulative power occurs, then f_{10}, f_{50}, and f_{90} were found to be very important descriptors of the ground motion. Table 1 reports Cum $G_{O(\omega)}$, f_{10}, f_{50}, and f_{90} for an artificial time history which produced a Response Spectrum which very closely fits the R.G. 1.60 Spectrum plus 6 actual earthquake ground motion records (Olympic, Taft, El Centro No. 12, Pacoima Dam, Hollywood Storage Lot, and El Centro No. 5) which produced both elastic and inelastic response very similar to that produced by the artificial time history when scaled to an effective acceleration, A_{DE}. This effective acceleration, A_{DE}, and actual peak ground acceleration, A, are also given in Table 1. Lastly, a Scaled Cum $G_{O(\omega)}$ appropriate for comparison with a 1g R.G. 1.60 Spectrum is obtained from:

$$\text{Scaled Cum } G_{O(\omega)} = \left(\frac{1.0g}{A_{DE}}\right)^2 (\text{Cum } G_{O(\omega)}) \qquad (2.4)$$

Figure 3 presents plots of the Cum PSD for each of these seven records, as reproduced from Reference (4). All seven of these records have similar characteristics, which accounts for the similarity in elastic and inelastic response produced by these records. These characteristics are:

1. Negligible power above about 12 Hz (see slope of Cumulative PSD curves in Figure 3)

2. Scaled Cumulative PSD between $0.49g^2$ and $0.71g^2$ for an effective peak acceleration of 1.0g

3. f_{10} between 0.55 and 1.20 Hz; f_{50} between 2.15 and 3.30 Hz; f_{90} between 5.50 and 7.90 Hz

Reference (4) also showed that the effective acceleration, A_{DE}, at which the R.G. 1.60 Spectrum needed to be anchored to produce linear and nonlinear responses similar to those from the six actual records could be accurately estimated from:

$$A_{DE1} = K_P \, A_{RMS}$$

where A_{RMS} is the root-mean-square acceleration and K_P is a mean peak factor as defined in Reference (4). For the artificial time history, K_P was 3.04 and ranged from 3.21 for the longest time history (Olympia) to 2.71 for the shortest (El Centro #5), with an average of 2.98 for the six actual records. Thus, a K_P value of 3.0 is a reasonable average for these seven records.

In turn, the RMS acceleration is related to the cumulative PSD by:

$$A_{RMS}^2 = \text{Cum } S_{O(\omega)}$$

$$A_{RMS}^2 = \frac{\text{Cum } G_{O(\omega)}}{2\pi}$$

depending upon whether the PSD is defined by Equation (2.1a) or (2.1b). For an A_{DE} = 1.0g and K_P = 3.0, the A_{RMS} should be 0.33g and

$$\text{Cum } S_{O(\omega)} = 0.11g^2$$

$$\text{Cum } G_{O(\omega)} = 0.70g^2$$

Thus, for a 1.0g R.G. 1.60 Response Spectrum, the Cum $G_{O(\omega)}$ should not exceed $0.70g^2$. To prevent the PSD requirements from generally controlling and to enable the R.G. 1.60 Response Spectrum to generally control, I recommend that the Cum $G_{O(\omega)}$ be established at about $0.63g^2$, which is midway within the range presented in Table 1 for the seven time histories presented.

Also presented in Table 1 are the Scaled Cumulative PSD, f_{10}, f_{50}, and f_{90} values corresponding to the PSD being defined by Equation (2.1). Since Table 1 is in terms of Cum $G_{O(\omega)}$, a G_O value of 6,900 in^2/sec^3 is used in Equation 2.1 in lieu of the S_O value of 1,100 in^2/sec^3. One should note that the Scaled Cumulative PSD from Equation 2.1, when put on a common basis, is approximately <u>three</u> times as great as for the seven records presented in Table 1. In addition, f_{90} is 17.0 Hz, which is out of line with f_{90} of about 6.6 Hz for the seven records studied. Table 2 compares the cumulative power predicted over various frequency ranges from Equation (2.1) with that given by the artificial R.G. 1.60 time history studied in Reference (4). Below 6.55 Hz, the power given by Equation (2.1) needs to be reduced by a factor of about 2.50, while above 6.55 Hz the cumulative power produced by Equation 2.1 is about 7.0 times too great.

Based upon a review of the Cumulative PSD plots presented in Figure 3 and the power characteristics given in Table 1, I recommend the following revised PSD requirements. From 0.4 Hz to 15 Hz, the average one-sided PSD over any ±20% frequency band width centered on a frequency f (i.e., 4 to 6 Hz band width for f = 5.0 Hz) computed over the strong motion duration should exceed:

<u>0.4 Hz to 15.0 Hz</u>

$$G_{O(f)} \geq 20{,}000 \frac{in^2}{sec^3} (f)^{-2.1} \leq 3{,}500 \frac{in^2}{sec^3} \qquad (2.5)$$

and Cum $G_O \geq 0.63g^2$

for a 1.0g peak ground acceleration with $G_{O(f)}$ scaled proportional to the square of the peak ground acceleration for other ground accelerations. Note that Equation (2.5) is consistent with the one-sided PSD being defined by Equation (2.2b). If it is decided that the one-sided PSD should be defined by Equation (2.2a) as is the case for Equation (2.1), then Equation (2.5) should be converted as defined by Equation (2.2c). Equation (2.5) is much more consistent with the Cumulative PSD results for all seven time histories reported in Table 1 and shown in Figure 3. As shown by Tables 1 and 2, Equation (2.5) produces a Cumulative PSD within each frequency band approximately 85% of that obtained from the artificial R.G. 1.60 time history. It also produces the same f_{10}, f_{50}, and f_{90} frequencies as does the artificial time history. Table 3 compares $S_{O(f)}$ from Equation (2.5) after being converted using Equation (2.2c) with $S_{O(f)}$ from Equation (2.1). My recommendation differs from the proposed SRP in the following ways:

1. No power requirements exist for frequencies below 0.4 Hz. Power below this frequency is immaterial to the seismic performance of nuclear power plant structures and equipment. Furthermore, the requirements of Equation (2.5) become excessively conservative below about 0.4 Hz, since most of the earthquake records show a substantial power dropoff below about this frequency.

2. No power requirements exist for frequencies above 15 Hz. All seven studied records which produce Response Spectra similar to the R.G. 1.60 Spectrum have negligible power above about 12 Hz. Equation (2.5) becomes excessively conservative above about 12 to 15 Hz. The R.G. 1.60 Spectrum can be accurately matched by ground motion records which contain essentially no power above about 15 Hz, and such records are representative of the actual earthquake records upon which the R.G. 1.60 Spectrum is based.

3. The average power over a ±20% frequency band width is compared to Equation (2.5) as opposed to comparing the average power over a 0.15 Hz band width with 85% of the power from Equation (2.1). Actual PSD plots have substantial peaks and valleys. In my opinion, it is the average power over a frequency band which is the important ground motion characteristic and not a very narrow (0.15 Hz), but deep valley. It is very difficult to produce a smooth PSD at frequencies above about 5 Hz (see Figure 1), and a requirement that a narrow 0.15 Hz

wide valley exceed a target PSD will produce excessive conservatism at higher frequencies.

4. The Equation (2.5) one-sided PSD ranges from 46% at 0.4 Hz to 21% at 15 Hz of the Equation (2.1) PSD.

5. The one-sided PSD requirement specified by Equation (2.5) introduces no excess conservatism in the design response spectrum. The R.G. 1.60 artificial time history used in Reference (4) meets the PSD requirements of Equation (2.5) throughout the frequency range of 0.4 to 15 Hz. The six actual records given in Table 1 can be Fourier Amplitude adjusted (retaining their Fourier Phase Spectra) to produce a smooth R.G. 1.60 Response Spectrum while meeting the PSD requirements of Equation (2.5).

The PSD limits defined by Equation (2.5) are appropriate when the required response spectrum is that defined by R.G. 1.60. When a different required response spectrum shape is specified, the PSD limits must be correspondingly adjusted. For instance, with a NUREG/CR-0098 median rock site spectrum shape, the specified PSD limits should be only 60% of those specified by Equation (2.5) over the entire frequency range because of the lesser amplifications with this spectrum shape. Alternately, if a spectrum shape with a substantially enriched high frequency content and lesser lower frequency content were specified such as those currently being considered for the east, the PSD limits should be enriched for the higher frequencies and reduced for lower frequencies.

3. Time History Strong Motion Duration and Envelope Function

In addition to specifying the characteristics of input motion in terms of a required response spectrum plus minimal PSD provisions, some requirements on strong motion duration and/or a time-envelope function should be specified which are consistent with earthquake ground motion records from which the required response spectrum was developed. Reference (4) suggests that the strong motion duration, T_D, of an input motion time history should be defined as the time over which the power is near its maximum. In turn, the power is simply the slope of a cumulative energy plot where cumulative energy $E_{(t_1)}$ at the time, t_1 is given by:

$$E_{(t_1)} = \int_{o}^{t_1} A^2{}_{(t)}\ dt \qquad (3.1)$$

where $A_{(t)}$ is the acceleration at time t. Figure 5 shows a cumulative energy plot from a representative time history. All seven time histories listed in Table 1 produce cumulative energy plots similar to that shown in Figure 5. For most time histories, Reference (4) recommends that the strong motion duration, T_D (duration of near maximum power) can be defined by:

$$T_D = T_{0.75} - T_{0.05} \qquad (3.2)$$

where $T_{0.75}$ and $T_{0.05}$ are the times at which 75% and 5%, respectively, of the cumulative energy are reached. For the time history shown in Figure 5 (El Centro #12), this strong motion duration is 9.6 seconds. For the six actual records listed in Table 1 which produce spectra similar to the R.G. 1.60 response spectrum, T_D ranges from 3.4 to 15.6 seconds. In my opinion, time histories consistent with the R.G. 1.60 response spectrum should have strong motion duration based upon Equation (3.2) of 5.0 to 16.0 seconds.

The use of strong motion duration in excess of 16 seconds can lead to either of the following unrealistic anomalies:

1. The high frequency power can be concentrated near the start of the record with the low frequency power concentrated near the end of the record. In this way the high

and low frequency modes of a 5% or more damped structure will not combine because the high frequency response is damped out before the low frequency response becomes strong. Thus, combined response can be severely unconservatively biased.

2. Alternately, if random phasing is used for all Fourier harmonics, then modes have an increased probability of coming into essentially worst-case phasing (absolute sum combination) at some time as strong motion durations are increased to very long times. Thus, combined responses can be severely overestimated when excessively long strong motion durations are used.

When strong motion durations of 20 seconds or longer are used, combined responses of multi-mode systems can be either severely overestimated or severely underestimated, depending upon how the phasing of different Fourier harmonics is handled. To avoid these problems, the use of artificial time histories with strong motion durations in excess of about 16 seconds should not be allowed.

One method to develop an artificial input time history for use in design is to first choose an actual earthquake time history which produces a response spectrum shape close to the required response spectrum shape (such as the R.G. 1.60 response spectrum shape) and an appropriate strong motion duration. Then the Fourier phase spectrum from this time history is retained and the Fourier amplitudes are adjusted, frequency by frequency, until the resulting response spectrum closely envelopes the required response spectrum. When this method is used, it is unnecessary to define a time-envelope function (Figure 4). I prefer this approach because the resultant artificial time history is assured of being like that produced by an earthquake except that the resulting response spectrum is smooth. Many records have an appropriate strong motion duration, T_D, as defined by Equation (3.1) and spectrum shape so that they may be used as the "seed" record in this approach when the required response spectrum is either defined by R.G. 1.60 or NUREG/CR-0098. Among these are the first five actual records defined in Table 1. The only problem with the sixth record (El Centro #5) is that its strong motion duration is only 3.4 seconds, which might be undesirably short.

Alternately, one might start with a random Fourier phase spectrum. When this is done, it is also necessary to establish a deterministic time-envelope function such as that shown in Figure 4. With this method, one must specify a time of maximum power, t_m, a rise time, t_r, and a decay time, t_d. Reference (4) showed that both peak elastic and inelastic responses are primarily determined by the portion of the time-history record at which the power is near its maximum. Therefore, the strong motion duration, T_D, is only slightly greater than t_m when a Figure 4 envelope function is used. Thus, to achieve a strong motion duration between 5 and 16 seconds, t_m should be specified between 4 and 15 seconds. The rise and decay time durations are relatively unimportant but should typically be taken to be about 1/7 and 5/7 of t_m, respectively. Use of t_m durations longer than about 15 seconds, or total durations longer than about 28 seconds, should not be allowed because such long durations can lead to the previously enumerated anomalies. Also, t_m durations much less than 4 seconds are inconsistent with a R.G. 1.60 response spectrum shape.

4. Ratio of Vertical to Horizontal Ground Motions

I note that all guidance has been removed from SRP Section 3.7.1 (Seismic Design Parameters), Subsection II (Acceptance Criteria), Item 1a (Design Response Spectra) on the recommended relationship between vertical and horizontal response spectra, and that no guidance has been added to Section 2.5.2 (Vibratory Ground Motion) on this subject when site-specific response spectra are not developed. In my opinion, this deletion leaves an obvious deficiency. I concur with Sargent & Lundy and General Electric (2) that in most cases the provisions of Section 2.2.2.2 of the ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures" (5), should be permitted. These provisions state that the vertical spectra should be taken as two-thirds of the horizontal spectra throughout the entire frequency range. In my judgment, such a provision is reasonable except when the design earthquake has an epicentral distance less than about 10 kilometers. In this latter case, the vertical spectra are likely to exceed two-thirds of the horizontal spectra at frequencies of about 8 Hz and greater, and need special consideration.

5. Requirements for Use of Multiple Time Histories

Multiple time history analysis should be strongly encouraged. In my opinion, the requirement that when multiple time history analyses are performed, "as a minimum, five time histories should be used for analysis," as stated on page 3.7.1-11 of Section 3.7.1 of Ref. (1), will discourage the use of multiple time history analyses (for instance, see both the Sargent & Lundy and the Westinghouse comments). I recognize that this requirement was recommended by the Task Action Plan A-40 Seismic Consultants to Lawrence Livermore Laboratory (of which I was a member) back in 1979, as documented in Reference (6). However, since that time considerable discussion on this subject occurred in the ASCE Working Group on Seismic Analysis of Safety Related Nuclear Structures, of which I was Chairman. In 1986, the consensus of this Working Group was published in the ASCE Standard 4-86 (5). I believe that the provisions of Section 2.3.1 of Reference (5) and the corresponding Commentary Section C.2.3.1 on the subject of multiple time history requirements are preferable to the response spectra and minimum number provisions contained on page 3.7.1-11 of Reference (1). The PSD provision on page 3.7.1-11 should be retained. This change will provide greater flexibility, will encourage the use of multiple time histories, and will answer the objections of both Sargent & Lundy and Westinghouse (See Reference 2).

6. Soil-Structure Interaction Requirements

A number of public comments (2) were made to the proposed revisions (1) to Section 3.7.2, Subsection II, Item 4 (Soil-Structure Interaction), pages 3.7.2-9 through 12.

6.1 Fixed-Base Analysis

I concur with Sargent & Lundy (2) in their recommendation that the provisions of Section 3.3.1.1 of the ASCE Standard 4-86 (5) can be used to define when a fixed-base analysis is adequate. The sentence additions to pages 3.7.2-9 and 10 of Reference (1) recommended at the top of page 3 of the Sargent & Lundy letter is an acceptible way to incorporate these provisions.

6.2 Requirements for Two Methods of Analysis Under Alternate 1

Under Alternate 1 for Soil-Structure Interaction, page 3.7.2-9 of SRP 3.7.2 (1) requires that both half-spare and finite boundary methods be used to perform the soil-structure interaction analysis. I concur with the comments of Sargent & Lundy, Westinghouse, and General Electric in Reference (2) that this requirement is no longer necessary, imposes a severe penalty in some cases, and should be deleted.

6.3 Limits on Stiffness Reduction with Increased Strain and Material (Hysteretic) Damping

I strongly believe that there is considerable uncertainty as to how much the shear modulus of soils reduces when subjected to high seismic strains. However, the provision 2 on page 3.7.2-12 of SRP 3.7.2 (1) does not seem to allow for any reduction in shear modulus even for the best-estimate shear modulus at seismic strain levels. If I have interpreted this provision correctly, I believe it will lead to too stiff of soil modeling, since provision 4 requires that lower bound and upper bound soil moduli generally be taken as one-half and twice the best-estimate soil modulus. If the best-estimate shear modulus is taken as the low-strain value, then the lower bound would only be reduced to half of the low-strain value and the upper bound would be twice the best-estimate low-strain value. By this approach, both the lower bound and upper bound shear moduli will be too stiff.

The best-estimate shear modulus under seismic strains should be allowed to be reduced for increased strains in accordance with the best available geotechnical evidence. However, a lower limit should be placed on the upper bound shear modulus to be used in SSI analyses. The upper bound shear modulus at seismic strains should not be allowed to be taken as less than 80% or 90% of the best-estimated low-strain (10^{-3} to 10^{-4}) shear modulus. This restriction adequately guards against uncertainty as to how much the shear modulus of soils reduces at high seismic strains.

The limit of provision 2 on page 3.7.2-12 of SRP 3.7.2 (1), that material (hysteretic) damping is not expected to exceed about 5% of critical, is too restrictive. A 15% limit, as recommended in Reference (6), is more appropriate.

6.4 Vertical Spatial Variation of Ground Motion

Considerable uncertainty exists with regard to the vertical spatial variation of ground motion. When the design control motion is defined at the free ground surface (as it generally should be), and the ground motion at the foundation level for a partially embedded structure is obtained by deconvolution of the free ground surface motion, a limit should be imposed on how much the ground motion is allowed to be reduced with depth. In recognition of the uncertainty in vertical spatial variation of ground motion, the ASCE Standard 4-86 (5) states in Section 3.3.1.2(b) that "the spectral amplitude of the acceleration response spectra in the free field at the foundation depth shall be not less than 60% of the corresponding design response spectra at the finish grade in the free field." I concur with Westinghouse (2), that this limit should be imposed in SRP Section 3.7.2, Subsection II, Item 4 (Soil-Structure Interaction).

7. Damping Requirements

Paragraph 2 on page 3.7.1-12 of SRP Section 3.7.1, Subsection II, Item 2 (Percentage of Critical Damping Values) of Reference (1) defines the correlation between stress levels and damping values. I concur with the Sargent & Lundy comment on the bottom of page 2 of their letter (2), that the requirements for correlation between damping values and stress levels defined in Section 3.1.2.2 of ASCE 4-86 (5) are more reasonable and should be substituted.

8. Modal Combination Requirements

On page 3.7.2-16, Item 7 (Combination of Modal Responses) of SRP Section 3.7.2, Subsection II (1) refers to Reference (7) for the combination of closely spaced modes. I concur with the comment of both Sargent & Lundy and Westinghouse (2) that requirements of Reference (7) for closely spaced modes are overly conservative and that Reference (7) should be revised to allow the algebraic sum of closely spaced modal responses in accordance with Equation (3200-16) of ASCE 4-86 (5) and the recommendations of References (6) and (8).

9. Greater Use of Professional Society Consensus Standards

In my opinion, it is highly desirable to encourage the development of professional consensus standards such as ASCE 4-86 (5) and ASME Appendix N (9). These standards were developed through the voluntary contribution of many hours of effort by professionals in the field and have undergone substantial consensus review. When the Nuclear Regulatory Commission neither acknowledges these standards nor adapts their provisions whenever possible, it is highly discouraging to the development and updating of such standards. I strongly concur with the comments of Dr. John Stevenson (2) in this regard and recommend the incorporation of his Insert A. In addition, I believe the Standard Review Plan should reference those standards and adapt their provisions wherever possible. Otherwise, these voluntary efforts will simply disappear.

References

(1) Proposed Revision 2 to Standard Review Plan, Sections 2.5.2, 3.7.1-3.7.3, NUREG-0800, U.S. Nuclear Regulatory Commission, May 1988.

(2) Public Comments on Proposed Revision 2 to Sections 2.5.2, 3.7.1-3.7.3 of Standard Review Plan, July 1988.

(3) Shinozuka, M., Mochio, T., and Samaras, E. F., "Power Spectral Density Functions Compatible With NRC Regulatory Guide 1.60 Response Spectra," NUREG/CR-3509, U.S. Nuclear Regulatory Commission, March 1984.

(4) Kennedy, R. P., et al., "Engineering Characterization of Ground Motion--Task I," NUREG/CR-3805 Vol. 1, U.S. Nuclear Regulatory Commission, February 1984.

(5) "Seismic Analysis of Safety-Related Nuclear Structures," ASCE Standard 4-86, September 1986.

(6) Coats, D. W., "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," NUREG/CR-1161, U.S. Nuclear Regulatory Commission, December 1979.

(7) Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis."

(8) "Report of the U.S. Nuclear Regulatory Commission Piping Review committee," Vo. 4, NUREG-1061, September 1984.

(9) ASME Boiler and Pressure Vessel Code, Section III, Appendix N, "Dynamic Analysis Methods," 1986.

TABLE 1

POWER CHARACTERISTICS OF R.G. 1.60 TYPE GROUND MOTION (FROM REFERENCE 4)

Record	Peak Acceleration A (g)	Effective Acceleration A_{DE} (g)	Cumulative $G_{O(\omega)}$ $(g)^2$	Scaled Cumulative $G_{O(\omega)}$ $(g)^2$	f_{10} (Hz)	f_{50} (Hz)	f_{90} (Hz)
Artificial	0.20	0.20	2.70×10^{-2}	0.675	0.60	2.15	6.55
Olympia	0.281	0.219	2.35×10^{-2}	0.490	1.20	3.05	6.10
Taft	0.180	0.149	1.58×10^{-2}	0.712	1.10	2.70	5.50
El Centro #12	0.142	0.128	1.12×10^{-2}	0.684	0.55	3.05	7.50
Pacoima Dam	1.170	0.856	0.445	0.607	0.75	2.60	6.70
Hollywood Storage	0.211	0.233	3.41×10^{-2}	0.628	0.75	3.30	7.90
El Centro #5	0.530	0.471	0.138	0.622	0.80	2.75	6.75
Eqn (2.1)	1.00	1.00	2.010	2.010	0.62	2.93	17.0
Eqn (2.5)	1.00	1.00	0.578	0.572	0.59	2.16	6.57

TABLE 2

COMPARISON OF CUMULATIVE POWER OVER VARIOUS FREQUENCY RANGES

Record	Cumulative Power G_O (g^2) for 1.0 Peak Acceleration			
	Below 0.6 Hz	0.6 Hz to 2.15 Hz	2.15 Hz to 6.55 Hz	Above 6.55 Hz
Artificial R.G. 1.60	0.068	0.270	0.270	0.068
Equation (2.1)	0.194	0.583	0.721	0.501
Ratio $\frac{\text{Artificial}}{\text{Eqn (2.1)}}$	0.35	0.46	0.37	0.14
Equation (2.5)	0.059*	0.229	0.232	0.058**
Ratio $\frac{\text{Artificial}}{\text{Eqn (2.5)}}$	1.14	1.18	1.16	1.16

* Conservatively assumes that G_O from 0 to 0.4 Hz averages half of required G_O from 0.4 to 0.6 Hz.

** Conservatively assumes no power above 15 Hz.

TABLE 3

COMPARISON OF RECOMMENDED
PSD REQUIREMENTS

Frequency (Hz)	$S_O(f)$ Eqn (2.5) (in^2/sec^3)	$S_O(f)$ Eqn (2.1) (in^2/sec^3)	$\frac{\text{Eqn (2.5) PSD}}{\text{Eqn (2.1) PSD}}$
0.4	557	1,208	0.46
1.7	557	1,386	0.40
3.0	317	865	0.37
6.5	63	259	0.24
8.4	36	162	0.22
10.0	25	116	0.22
15.0	11	53	0.21
20.0	0	30	0

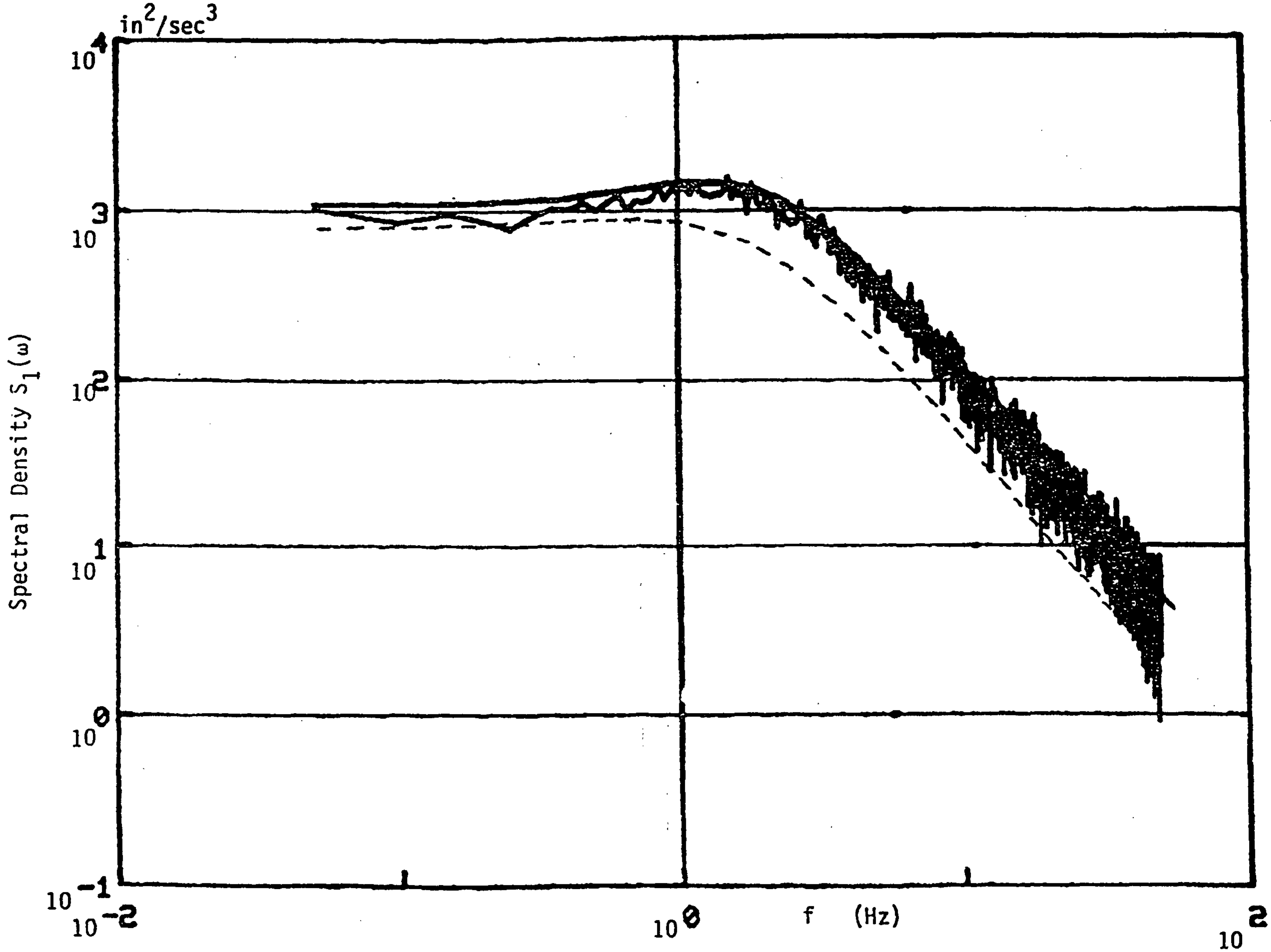


Fig. 1 Spectral Density Functions (Initial, Actual and Lower Bound); Parameter Set No. 2 (From Reference 3)

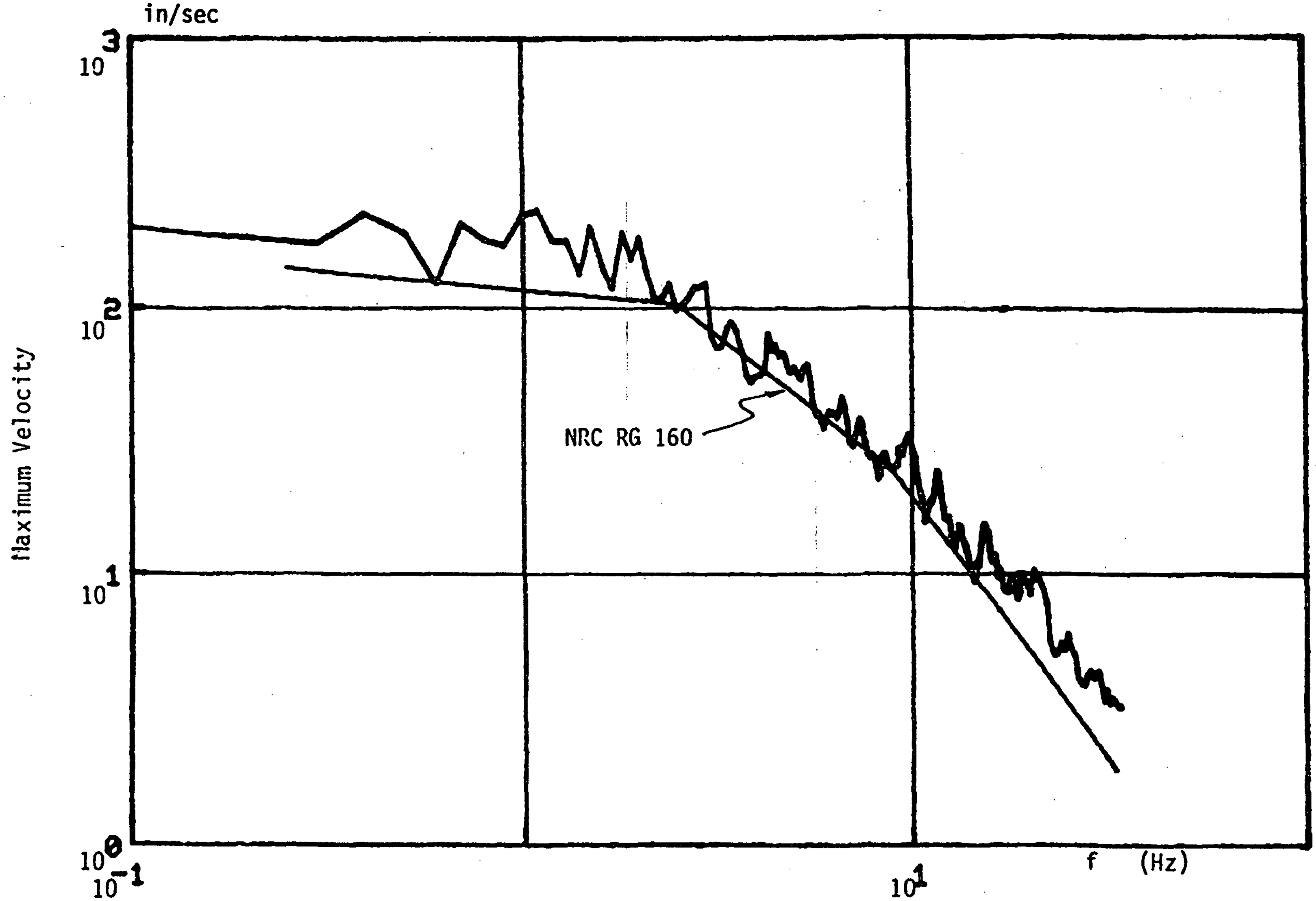
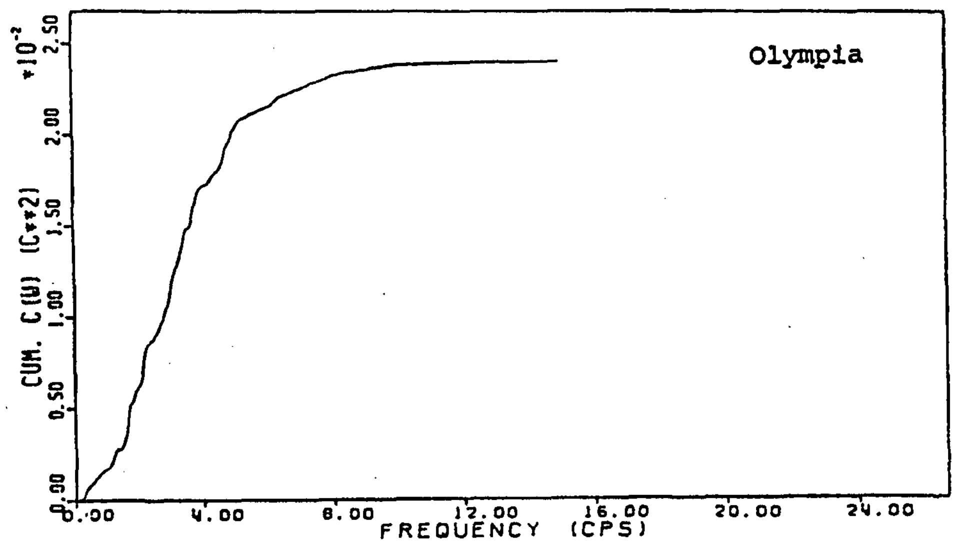
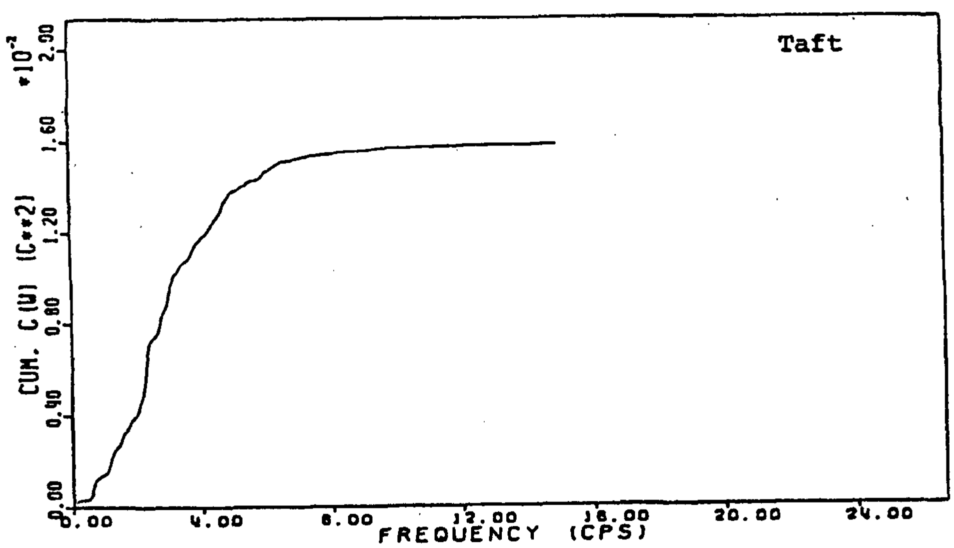


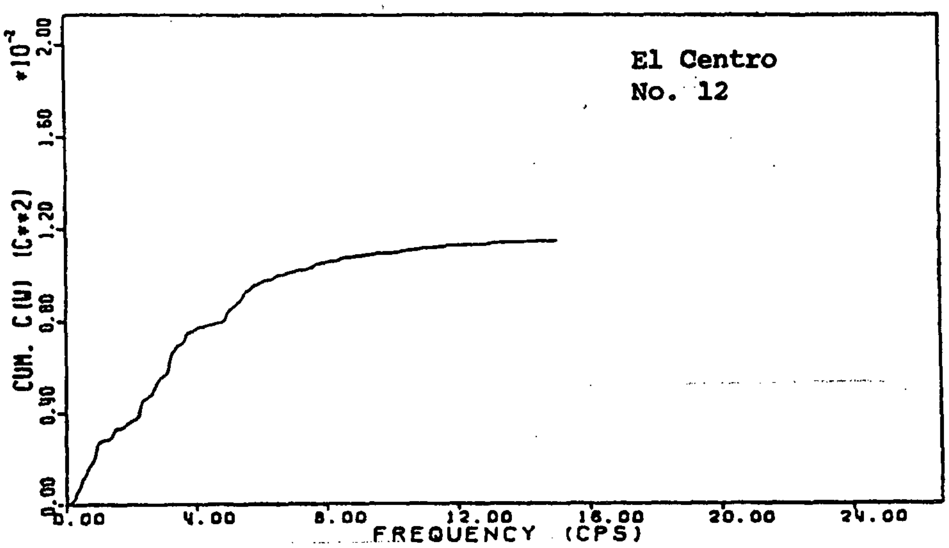
Fig. 2 Velocity Spectrum; Parameter Set No. 2
(From Reference 3)

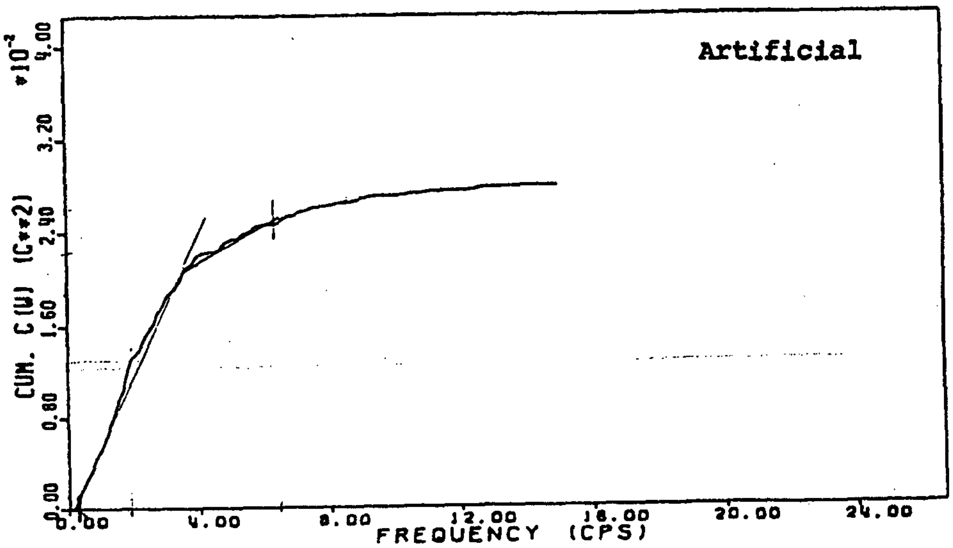




(a) $T_D' > 9$ sec.

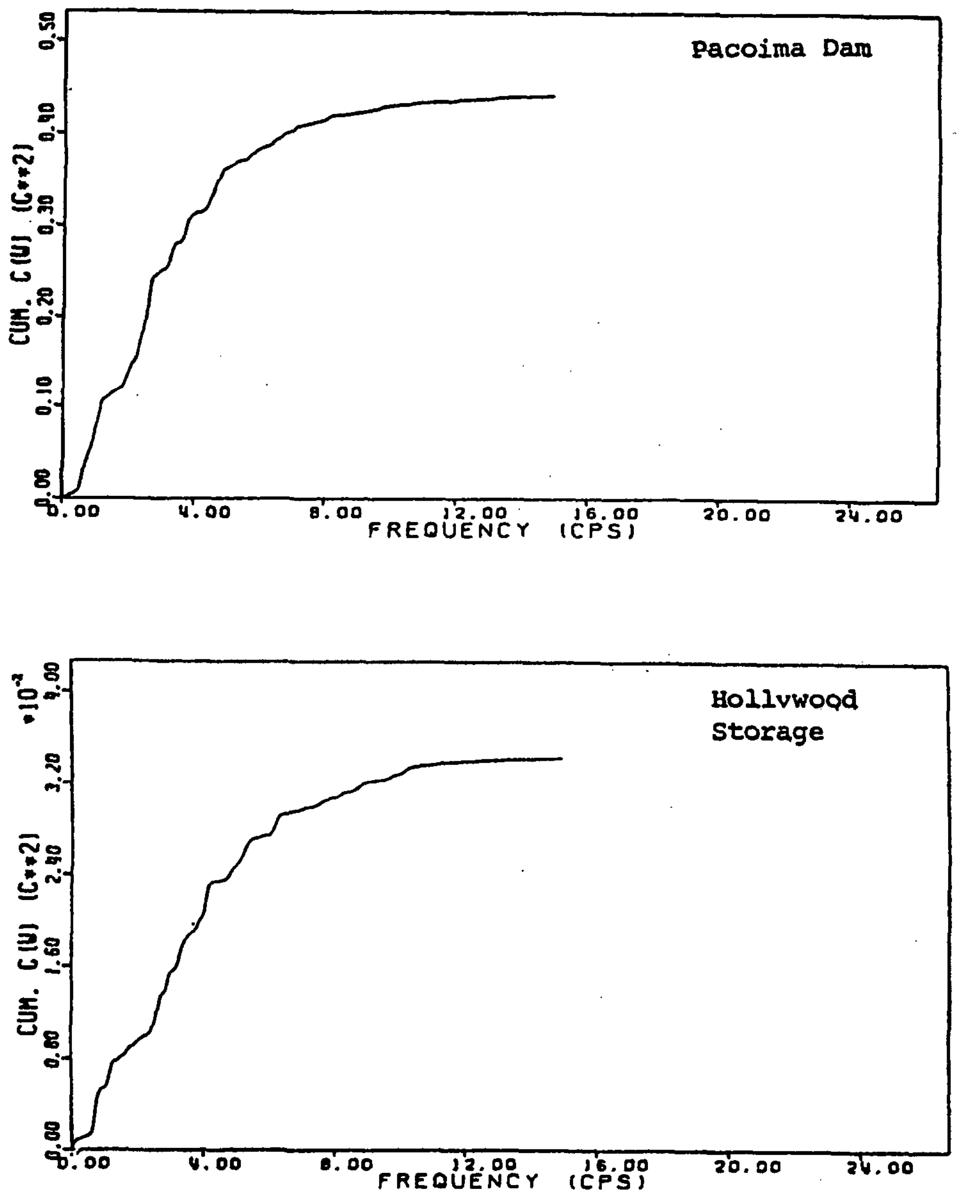
FIGURE 3. CUMULATIVE SPECTRAL DENSITY FUNCTIONS OF EFFECTIVE ACCELEROGRAM SEGMENT DEFINED BY $T_D' = T_m - T_{.05}$ (Reproduced from Ref. 4)





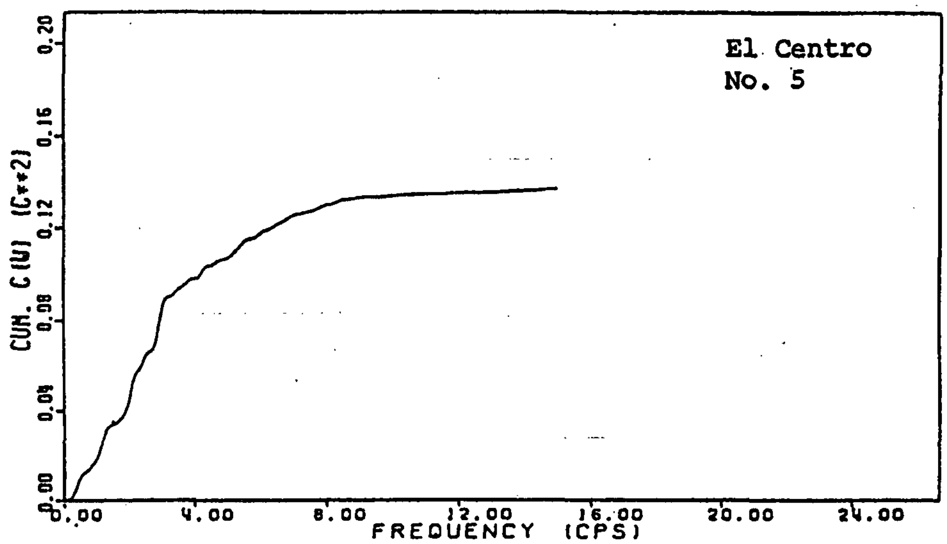
(a) $T_D' > 9$ sec.
(Continued)

FIGURE 3 (Cont.). CUMULATIVE SPECTRAL DENSITY FUNCTIONS OF EFFECTIVE ACCELEROGRAM SEGMENT DEFINED BY $T_D' = T_m - T_{.05}$ (Reproduced from Ref. 4)



(b) 2.5 sec $\leq T'_D \leq$ 9 sec.

FIGURE 3 (Cont.). CUMULATIVE SPECTRAL DENSITY FUNCTIONS OF EFFECTIVE ACCELEROGRAM SEGMENT DEFINED BY $T'_D = T_m - T_{.05}$ (Reproduced from Ref. 4)



(b) 2.5 sec $\leq T'_D \leq$ 9 sec.
(Continued)

FIGURE 3 (Cont.). CUMULATIVE SPECTRAL DENSITY FUNCTIONS OF EFFECTIVE ACCELEROGRAM SEGMENT DEFINED BY $T_D = T_m - T_{.05}$ (Reproduced from Ref. 4)

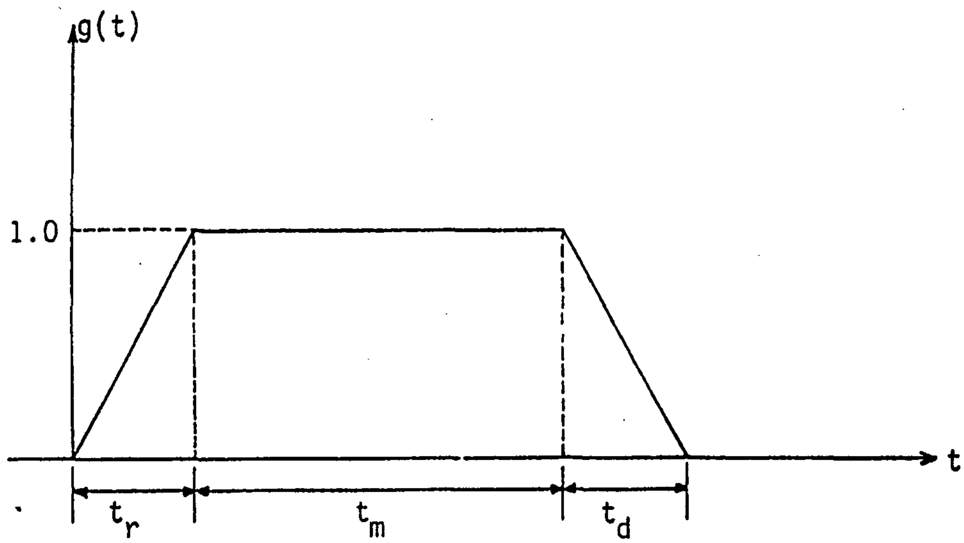
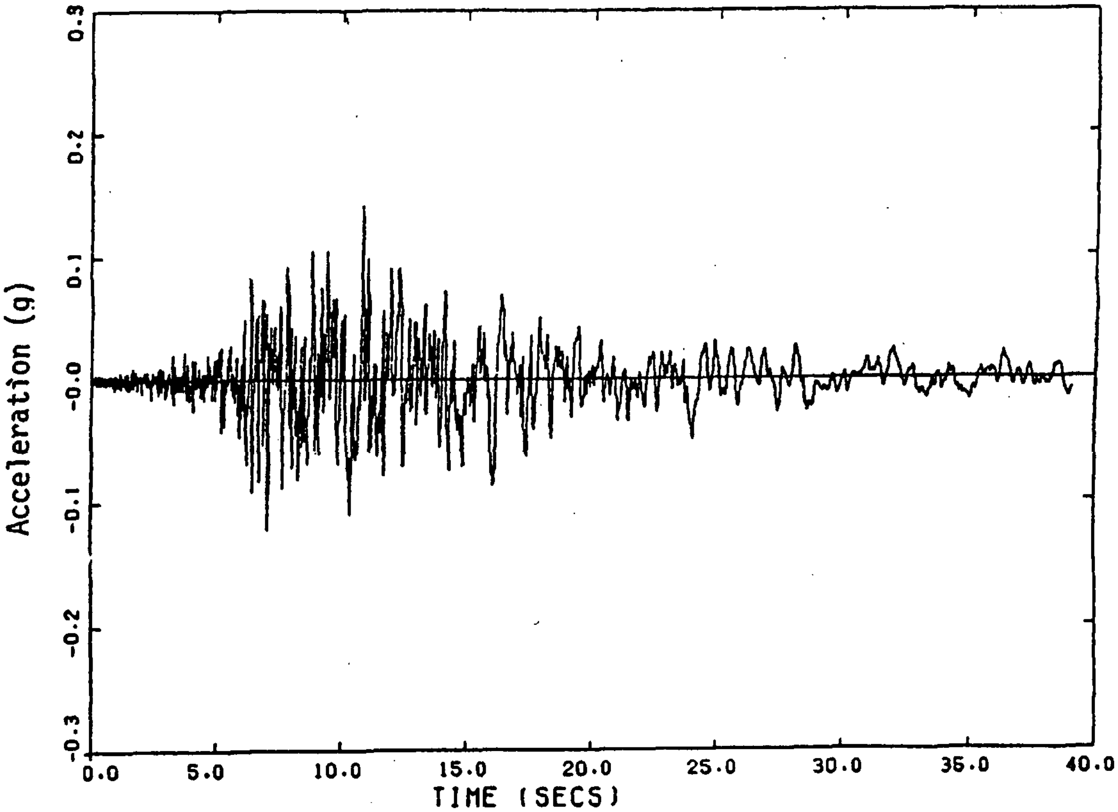


Figure 4. Deterministic Envelope Function



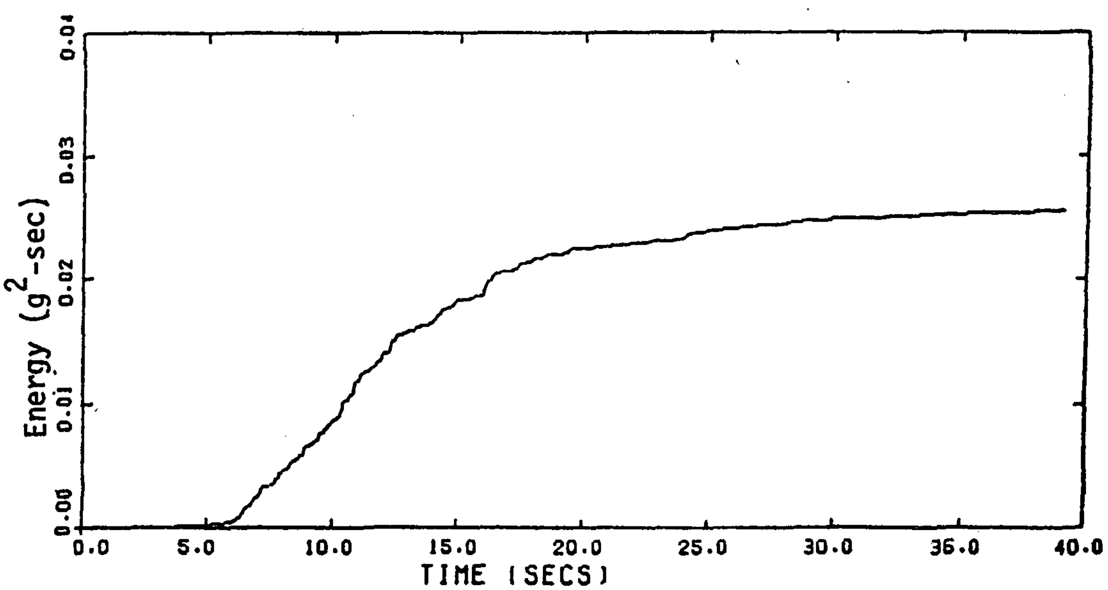


FIGURE 5. ACCELEROGRAM AND CORRESPONDING CUMULATIVE ENERGY FOR THE EL CENTRO, ARRAY NO. 12, IMPERIAL VALLEY 1979 (140) RECORD (From Ref. 4)

RPK Robert P. Kennedy

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February 18, 1989

Dr. A. J. Philippacopoulos
Brookhaven National Laboratories
Bldg. 129
Upton, NY 11973

Re: Comments on Proposed Revisions to Standard Review Plan Seismic Provisions

Dear Mike:

Enclosed are the original copies for incorporation into your report of my report on the subject material and the report by Professor Shinozuka and myself on PSD functions compatible with R.G. 1.60.

Very truly yours,

Bob

Robert P. Kennedy

cc. Prof. Shinozuka

APPENDIX B

RECOMMENDED MINIMUM POWER SPECTRAL DENSITY FUNCTIONS COMPATIBLE WITH NRC REGULATORY GUIDE 1.60 RESPONSE SPECTRUM

Prepared for

Brookhaven National Laboratory

by

R. P. KENNEDY
R.P.K. STRUCTURAL MECHANICS CONSULTING

M. SHINOZUKA
PRINCETON UNIVERSITY

JANUARY 1989

1. Introduction

Around May 1988, the U.S. Nuclear Regulatory Commission (NRC) issued Proposed Revision 2 to Sections 2.5.2, 3.7.1, 3.7.2, and 3.7.3 of their Standard Review Plan, NUREG-0800 (1) for public review and comment. One of the proposed revisions was the introduction of the following Power Spectral Density (PSD) requirement to Section 3.7.1:

> In addition to the response spectra enveloping requirement, the use of single time history will also be justified by demonstrating sufficient energy at the frequencies of interest through the generation of PSD function which is greater than some specified values throughout the frequency range of significance, from 0.24 Hz to 34 Hz. For the cases where the design response spectra correspond to those of RG 1.60 spectra, the underlying stationary process of the artificial time history (representing horizontal component of the earthquake) must possess a power spectral density which is, generally, not less than the following target spectral density $S_O(\omega)$ Kanai-Tajimi form throughout the frequency range between 0.2 Hz and 34 Hz. Reference (2) contains details of the basis for the staff recommendation. The spectral values should be computed at frequency intervals no greater than 0.05 Hz. The smoothing of the PSD function is acceptable, if it is performed by means of the moving average method involving three successive frequency points (ω_{i-1}, ω_i and ω_{i+1}) with the average values plotted at ω_i. Further, the computed PSD at no frequency should drop below 15 percent of the target value.

$$S_O(\omega) = S_O \frac{1 + 4\xi_g^2 (\omega/\omega_g)^2}{[1 - (\omega/\omega_g)^2]^2 + 4\xi_g^2 (\omega/\omega_g)^2} \qquad \text{(PSD 1)} \quad (1)$$

> with S_O = 1,100 in^2/secs^3 (this value corresponds to a peak acceleration of 1g), ω_g = 10.66 rad/sec and ξ_g = 0.9793.

Such an artificial time history, having satisfied both the response spectrum and power spectral requirements, may be used as a representative seismic input for design purposes after being properly scaled (Reference 2). The above target PSD function is one acceptable form to demonstrate sufficient energy content in the frequency range of interest. Other forms may be used, if justified. For the cases where design response spectra do not correspond to RG 1.60 spectra, the target PSD function corresponding to the design response spectra and the demonstration of adequate energy in the frequency range of the interest are reviewed on a case-by-case basis.

At the outset, it should be clearly noted that Equation (1) represents a one-sided PSD which is related to the Fourier Amplitude $| F_{(\omega)} |$ by:

$$S_{O(\omega)} = \frac{2 \ | F_{(\omega)} |^2}{2\pi \ T_D} \qquad (2)$$

where T_D is the strong motion duration over which $F_{(\omega)}$ is evaluated. This duration T_D represents the duration of near maximum and nearly stationary power of an acceleration time history record. For an artificial time history with a deterministic time envelope function such as that shown in Figure 1:

$$T_D = t_m \qquad (3)$$

For an actual earthquake time history, T_D represents the duration over which the slope (power) of a cumulative energy plot is nearly constant and near maximum where cumulative energy $E_{(t_1)}$ at the time t_1 is given by:

$$E_{(t_1)} = \int_{o}^{t_1} A^2(t) \ dt \qquad (4)$$

where A(t) is the acceleration at time t. Figure 2 shows a cumulative energy plot from a representative time history. For the record shown in Figure 2, power is nearly constant and near maximum from about 6.4 seconds to 16 seconds for a duration T_D of about 9.4 seconds.

Alternative and more sophisticated definitions exist within the literature for the PSD and for the duration T_D over which it is to be evaluated. Throughout this brief report the definitions presented in the previous paragraph are used.

Reference (3) recommends that the Design Ground Motion should be primarily defined by the Design Response Spectrum. The PSD requirement is a secondary requirement which is simply used to prevent a severe deficiency of power over any frequency range. The PSD requirement should not be used to add additional conservatism beyond that contained in the Design Response Spectrum. Most ground motion time histories which produce a response spectrum that closely fits the Design Response Spectrum should be able to pass the PSD requirement.

The PSD defined by Equation (1) was initially recommended in Reference (2) as being compatible with the RG 1.60 Response Spectrum. However, in Reference (2) it was recommended for use in generating artificial time histories which produced response spectra which <u>conservatively enveloped</u> the RG 1.60 Response Spectrum. As such, it was never intended by its authors to represent a <u>minimum</u> PSD requirement. Equation (1) is not compatible with the goal recommended by Reference (3) that the Design Ground Motion should be primarily defined by the Design Response Spectrum and that the PSD requirement should be a secondary requirement used to prevent a severe deficiency of power over any frequency range. In fact, the PSD requirement of Equation (1) will introduce additional conservatism beyond that contained in the RG 1.60 Response Spectrum at all frequencies, but particularly so above about 10 Hz, as was clearly illustrated by the results presented in Ref. (2).

Cumulative PSD plots and other power characteristics of a number of actual earthquake ground motion records have been presented in Reference (4). It concluded that the Cumulative PSD as defined by:

$$\text{Cum } S_{O(\omega)} = \int_{0}^{\omega} S_{O(\omega)} \, d\omega \tag{5}$$

is an important descriptor of the ground motion. In particular, if one defines f_{10}, f_{50}, and f_{90} as the frequencies below which 10%, 50%, and 90% of the cumulative power occurs, then f_{10}, f_{50}, and f_{90} were found to be very important descriptors of the ground motion. Table 1 reports f_{10}, f_{50}, and f_{90} for an artificial time history that produced a response spectrum which very closely fits the RG 1.60 Spectrum plus 6 actual earthquake ground motion records (Olympic, Taft, El Centro No. 12, Pacoima Dam, Hollywood Storage Lot, and El Centro No. 5) which produced both elastic and inelastic response very similar to that produced by the artificial time history.

Based upon a review of the Cumulative PSD plots and the power characteristics given in Reference (4), Reference (3) recommended that the minimum PSD requirements compatible with the RG 1.60 Response Spectrum should be as follows. From 0.4 Hz to 15 Hz, for a 1.0g peak ground acceleration, the average one-sided PSD over any ±20% frequency band width centered on a frequency f (i.e., 4 to 6 Hz band width for f = 5.0 Hz) computed over the strong motion duration should exceed:

0.4 Hz to 2.3 Hz

$$S_{O(\omega)} \geq 557 \frac{in^2}{sec^3}$$

(PSD 2)

2.3 Hz to 15 Hz (6)

$$S_{O(\omega)} \geq 3183 \frac{in^2}{sec^3} (f)^{-2.1}$$

where $f = \frac{\omega}{2\pi}$.

Table 2 compares the minimum PSD requirements from Equation (6) with the conservative envelope requirements from Equation (1). The two requirements differ by a ratio of 2.2 at 0.4 Hz to 4.8 at 15 Hz.

Because of this large difference, Mr. Nilesh Chokshi and Mr. Klalid Shaukat of the NRC Staff requested that we present a mutually agreeable minimum Power Spectral Density (PSD) Function compatible with the NRC Regulatory Guide 1.60 Response Spectrum. A time history based upon this minimum PSD requirement should produce a response spectrum which lies close to, but generally below, the RG 1.60 Response Spectrum.

2. Development of Minimum PSD Requirement

The process followed in developing a recommended minimum Power Spectral Density (PSD) requirement compatible with the RG 1.60 Response Spectrum was as follows:

1. Starting with a candidate PSD function, a deterministic time envelope function (Figure 1) and a randomly selected set of phase relationships generate an artificial time history.

2. From this artificial time history, produce the 2% damped response spectrum and compare with the 2% damped RG 1.60 Response Spectrum.

3. Repeat this process until the resultant response spectrum lies close to, but generally below, the RG 1.60 Response Spectrum for frequencies between about 0.4 Hz and 20 Hz. The response spectrum below 0.4 Hz is of little interest for stiff nuclear power plant structures and so a match below this frequency was not considered to be of interest. The response spectrum above about 20 Hz for the RG 1.60 Response Spectrum shape is primarily controlled by the peak acceleration of the resultant time history. In turn, this peak acceleration is insensitive to the shape of the PSD function. Artificially high peak accelerations can be removed from the resultant time history by either "clipping" or "fractional folding," as described in Reference (2), with little or no effect on the smoothed PSD function averaged over any ±20% frequency band, as will be shown. Thus, response spectrum fitting above about 20 Hz was not a prime consideration in selecting the minimum PSD requirement.

Two time-envelope functions of the type shown in Figure 1 were used for this brief study. They were:

Time	Function A	Function B
t_r (sec.)	5.0	1.4
t_m (sec.)	10.24	10.24
t_d (sec.)	5.0	7.0

Function A has a symmetric rise and decay time, while Function B has an asymmetric rapid rise time and much slower decay time, similar to many actual earthquake ground motion records. Both have the same maximum power duration, t_m, of 10.24 seconds, which is sufficiently long so that the ground motion can be treated as stationary, at least within the frequency range of interest (0.4 Hz to 40 Hz). It will be shown that time histories generated using Envelope Functions A and B both produce essentially the same response spectra so that t_r and t_d are of little significance. Low (2%) damped response spectra will increase slightly with increasing maximum power duration t_m and will decrease slightly with decreasing t_m. However, so long as t_m exceeds about 4 seconds, these differences will be small at frequencies in excess of about 1 Hz.

Given a candidate PSD function, $S_{O(\omega)}$, and a time-envelope function, $g_{(t)}$, as shown in Figure 1, an artificial ground acceleration time history, $\ddot{z}_{O(t)}$, can be generated from:

$$\ddot{z}_O(t) = g(t)\,\ddot{\zeta}_O(t) \tag{7}$$

$$\ddot{\zeta}_O(t) = \sqrt{2}\sum_{k=1}^{N}\sqrt{S_O(\omega_k)\,\Delta\omega}\,\cos\,(\omega_k t + \phi_k) \tag{8}$$

with

$$\omega_k = k\Delta\omega, \qquad \Delta\omega = \omega_u/N \tag{9}$$

and ϕ_k representing a sequence of independent realizations of the random variable $\mathbf{\Phi}$ uniformly distributed between 0 and 2π. The quantity ω_u in Equation (9) is the largest natural frequency value considered in this study;

$$\omega_u = N\Delta\omega = 1{,}000 \text{ rad/sec} \qquad (N = 1{,}630) \tag{10}$$

Figure 3 compares the 2% damped pseudo-relative velocity (PSRV) response spectrum generated from an artificial time history corresponding to the PSD function defined by Equation (1) (PSD 1) and time Envelope Function A with the 2% damped RG 1.60 Response Spectrum anchored at 1.0g. Figure 4 makes a similar comparison for the PSD function defined by Equation (6) (PSD 2). Note that PSD 1 produces a response spectrum that exceeds the RG 1.60 Response Spectrum by approximately a factor of 1.3 from 1.5 Hz to 10 Hz with greater exceedance at both lower and higher frequencies. Therefore establishing PSD 1 as a minimum requirement would produce greater conservatism than is embedded within the RG 1.60 Response Spectrum. On the other hand, PSD 2 produces a response spectrum which averages only about 70% of the RG 1.60 Response Spectrum between about 2.5 Hz and 12 Hz while being a bit high at frequencies below about 0.8 Hz.

Using the results obtained for PSD 1 and PSD 2, one can quickly narrow in on a recommended PSD which will produce a response spectrum close to the RG 1.60 Response Spectrum at all frequencies between about 0.4 Hz and 20 Hz. From about 2.5 Hz to about 9.0 Hz, the minimum required PSD should lie about 25% to 30% of the difference between PSD 1 and PSD 2 above PSD 2. At about 1.2 Hz, the minimum required PSD should approach PSD 2 and be less than PSD 2 below this frequency. Similarly, at about 15 Hz, the required PSD should approach PSD 2 and should drop off very rapidly at higher frequencies. Based upon these observations and several trials, the following minimum PSD requirement was developed:

Less Than 2.5 Hz

$$S_{O(\omega)} = 650 \text{ inch}^2/\text{sec}^3 \ (f/2.5 \text{ Hz})^{0.2}$$

2.5 Hz to 9.0 Hz

$$S_{O(\omega)} = 650 \text{ inch}^2/\text{sec}^3 \ (2.5 \text{ Hz}/f)^{1.8}$$

9.0 Hz to 16.0 Hz (PSD 3)

$$S_{O(\omega)} = 64.8 \text{ inch}^2/\text{sec}^3 \ (9.0 \text{ Hz}/f)^{3} \qquad (11)$$

Greater Than 16 Hz

$$S_{O(\omega)} = 11.5 \text{ inch}^2/\text{sec}^3 \ (16.0 \text{ Hz}/f)^{8}$$

where $f = \omega/2\pi$.

The PSD requirement defined for PSD 3 by Equation (11) is shown in Figure 5 while the relative cumulative power for PSD 3 is shown in Figures 6 and 7. The f_{10}, f_{50}, and f_{90} frequencies are about 0.7 Hz, 2.6 Hz, and 8.1 Hz, respectively, as noted in Table 1. The f_{10} and f_{50} frequencies are consistent with those obtained for the broad frequency content ground motion records also listed in Table 1, while f_{90} is only slightly higher than the highest f_{90} listed for the actual records in Table 1. If anything, PSD 3 may be slightly too broad in its high frequency content. However, a slight error in this direction is prudent for stiff nuclear plant structures.

Figure 8 presents the 2% damped PSRV response spectrum obtained from a time history based on PSD 3 and time Envelope Function A and compares this response spectrum with a 2% damped RG 1.60 Response Spectrum anchored at 1.0g. Figure 9 presents the same results for PSD 3 coupled with the time Envelope Function B. Note that the response spectra in Figures 8 and 9 are essentially identical, indicating the lack of importance of the specified rise time, t_r, and decay time, t_d. Note the excellent fit of the PSRV response spectrum generated from the PSD 3 requirements to the RG 1.60 Response Spectrum at all frequencies between about 0.25 Hz and about 23 Hz. With the exception of a couple of narrow spikes and a couple of narrow valleys, the PSRV response spectrum generated for the time history based on PSD 3 lies between 80% and 110% of the RG 1.60 Response Spectrum from 0.25 Hz to 23 Hz.

Figures 10 and 11 present the time histories obtained using PSD 3 and Envelope Functions A and B, respectively. Note the single high acceleration spike to approximately 520 inch/sec^2, which is approximately 35% greater than the desired 1.0g (386 in/sec^2). When a smooth PSD function and random phasing are specified, it is common to get at least one high frequency acceleration spike which exceeds the target peak ground acceleration (in this case, 1.0g). It is this high acceleration spike which causes the PSRV response spectra in Figures 8 and 9 to exceed the 1.0g RG 1.60 Response Spectrum at frequencies above about 23 Hz. The simplest solution is to either "clip" or "fractionally fold" the high peak acceleration at the target peak ground acceleration (1.0g) as recommended in Reference (2). When either "clipping" or "fractional folding" is done, the resulting PSD will only slightly be changed. However, the resulting response spectrum will closely match the RG 1.60 Response Spectrum at all frequencies.

Figure 12 shows the resulting PSD obtained when the time history shown in Figure 10 based on PSD 3 has the one peak which exceeds 1.0g clipped at 1.0g. Figures 13 and 14 show the 2% damped PSRV response spectrum obtained when the time histories are clipped at 1.0g. Note that the exceedances of the RG 1.60 Response Spectrum above 23 Hz have now disappeared.

3. Recommended Power Spectral Density Requirement

For an RG 1.60 Response Spectrum anchored to 1.0g, the following minimum PSD requirement should be specified in the Standard Review Plan. For other peak accelerations, this PSD requirement should be scaled by the square of the peak acceleration. From 0.3 Hz to 24 Hz, the average one-sided PSD defined by Equation (2) over a ±20% frequency band width centered on any frequency f (i.e., 4 to 6 Hz band width for f = 5.0 Hz) computed over the strong motion duration should exceed 80% of PSD 3 as defined by Equation (11). The power above 24 Hz for PSD 3 is so low as to be inconsequential so that checks above 24 Hz are unnecessary. Similarly, power below 0.3 Hz has no influence on stiff nuclear plant facilities so that checks below 0.3 Hz are unnecessary. This minimum check is set at 80% of PSD 3 so as to be sufficiently high to prevent a deficiency of power over any broad frequency band, but sufficiently low that this requirement introduces no additional conservatism over that already embodied in the RG 1.60 Response Spectrum. A time history can meet this minimum PSD requirement and still produce a response spectrum that lies below the RG 1.60 Response Spectrum at all frequencies.

To produce a response spectrum that accurately fits the 2% damped, 1.0g, RG 1.60 Response Spectrum at all frequencies above 0.25 Hz, we recommend the use of PSD 3 as defined by Equation (11) with the resulting time history being clipped at ±1.0g.

To produce a response spectrum that conservatively envelopes the 1.0g RG 1.60 Response Spectrum at 2% damping and greater, we recommend the use of a PSD set at 130% of PSD 3 defined by Equation (11) with the resulting time history being clipped at ±1.0g. Following this recommendation will result in a response spectrum 14.0% greater than that shown in Figure 13 at frequencies less than about 23 Hz and equal to that shown at frequencies greater than about 33 Hz.

References

(1) Proposed Revision 2 to Standard Review Plan, Sections 2.5.2, 3.7.1-3.7.3, NUREG-0800, U.S. Nuclear Regulatory Commission, May 1988.

(2) Shinozuka, M., Mochio, T., and Samaras, E. F., "Power Spectral Density Functions Compatible With NRC Regulatory Guide 1.60 Response Spectra," NUREG/CR-3509, U.S. Nuclear Regulatory Commission, March 1984.

(3) Kennedy, R. P., "Comments on Proposed Revisions to Standard Review Plan Seismic Provisions," Brookhaven National Laboratory, January 1989.

(4) Kennedy, R. P., et al., "Engineering Characterization of Ground Motion--Task I," NUREG/CR-3805 Vol. 1, U.S. Nuclear Regulatory Commission, February 1984.

TABLE 1

FREQUENCY CHARACTERISTICS OF RG 1.60 TYPE
GROUND MOTION (FROM REFERENCE 4)

Record	Frequencies		
	f_{10} (Hz)	f_{50} (Hz)	f_{90} (Hz)
Artificial	0.60	2.15	6.55
Olympia	1.20	3.05	6.10
Taft	1.10	2.70	5.50
El Centro #12	0.55	3.05	7.50
Pacoima Dam	0.75	2.60	6.70
Hollywood Storage	0.75	3.30	7.90
El Centro #5	0.80	2.75	6.75
PSD 1	0.62	2.93	17.0
PSD 2	0.59	2.16	6.57
PSD 3	0.69	2.64	8.13

TABLE 2

COMPARISON OF PSD REQUIREMENTS

Frequency (Hz)	$S_O(\omega)$ (in^2/sec^3) PSD 1	$S_O(\omega)$ (in^2/sec^3) PSD 2	$S_O(\omega)$ (in^2/sec^3) PSD 3
0.4	1,208	557	451
1.7	1,386	557	602
3.0	865	317	468
6.5	259	63	116
8.4	162	36	73
10.0	116	25	47
15.0	53	11	14
20.0	30	0	2

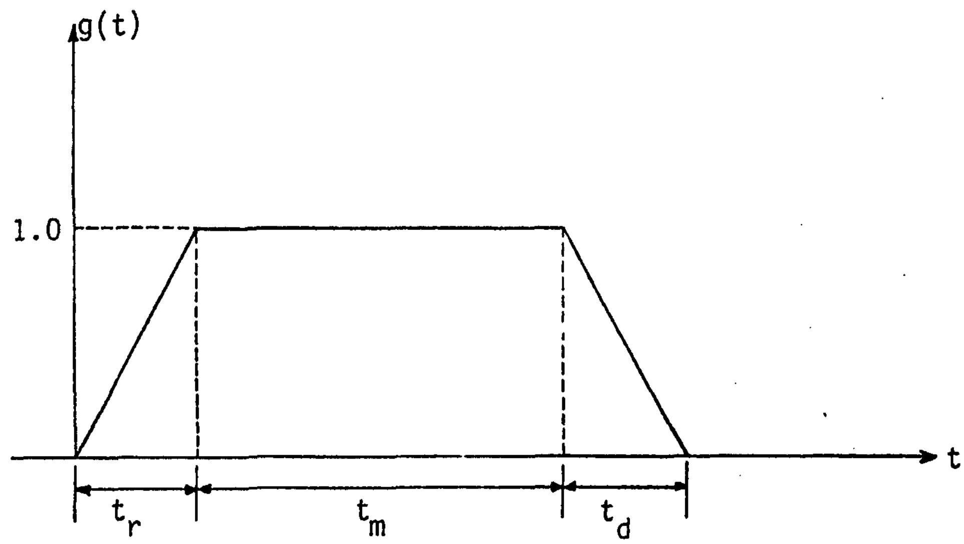
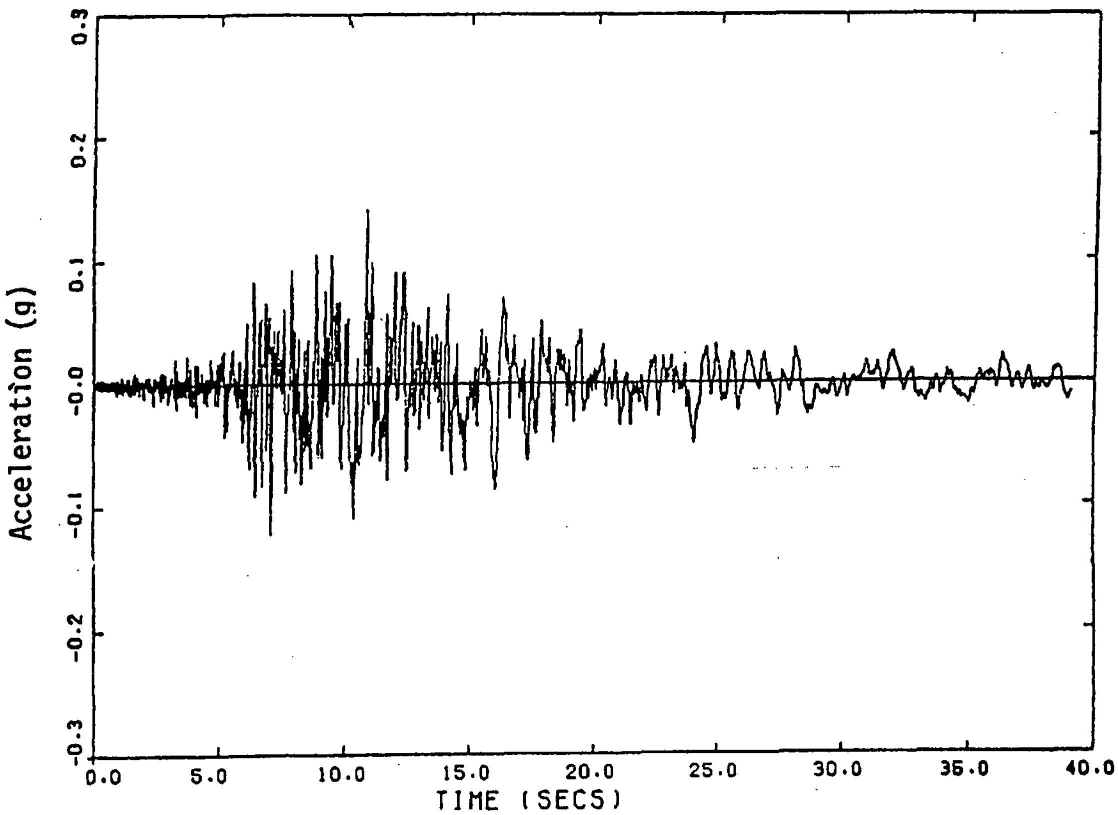


Figure 1. Deterministic Envelope Function



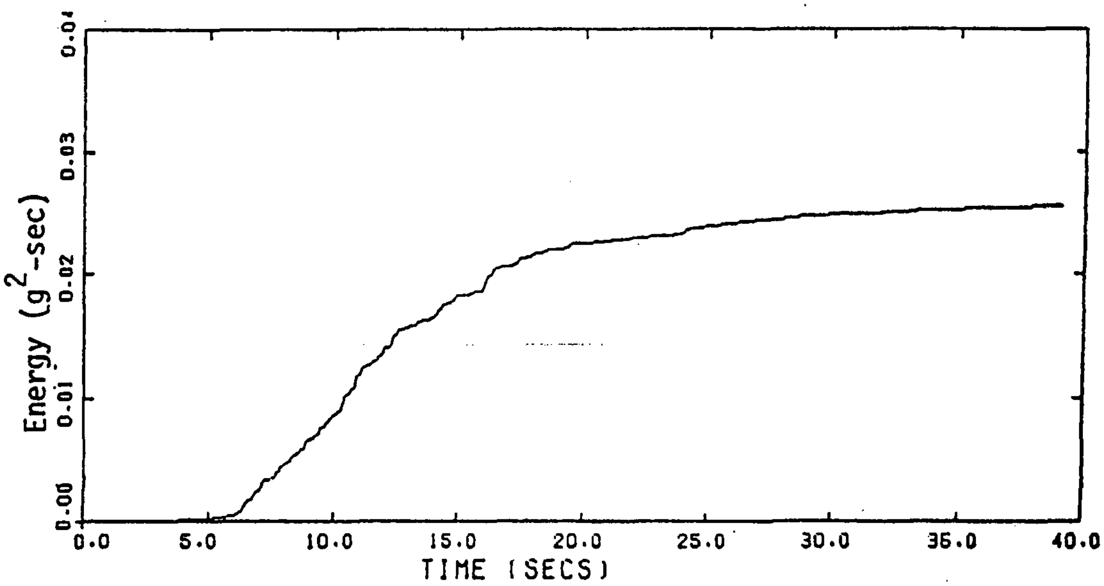


Figure 2. ACCELEROGRAM AND CORRESPONDING CUMULATIVE ENERGY FOR THE EL CENTRO, ARRAY NO. 12, IMPERIAL VALLEY 1979 (140) RECORD (From Ref. 4)

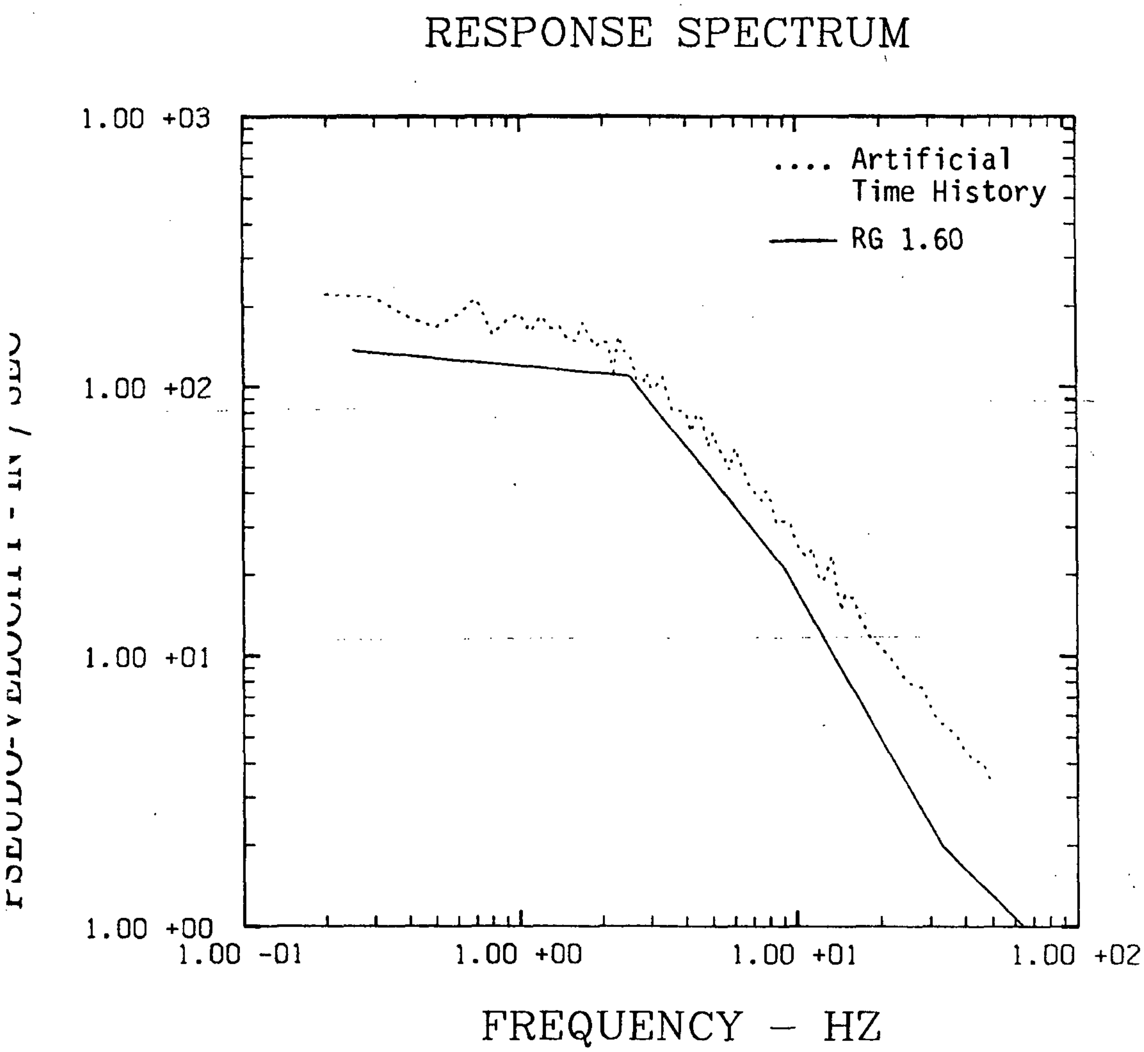


Figure 3. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM ASSOCIATED WITH PSD1 AND ENVELOPE FUNCTION A COMPARED TO RG 1.60

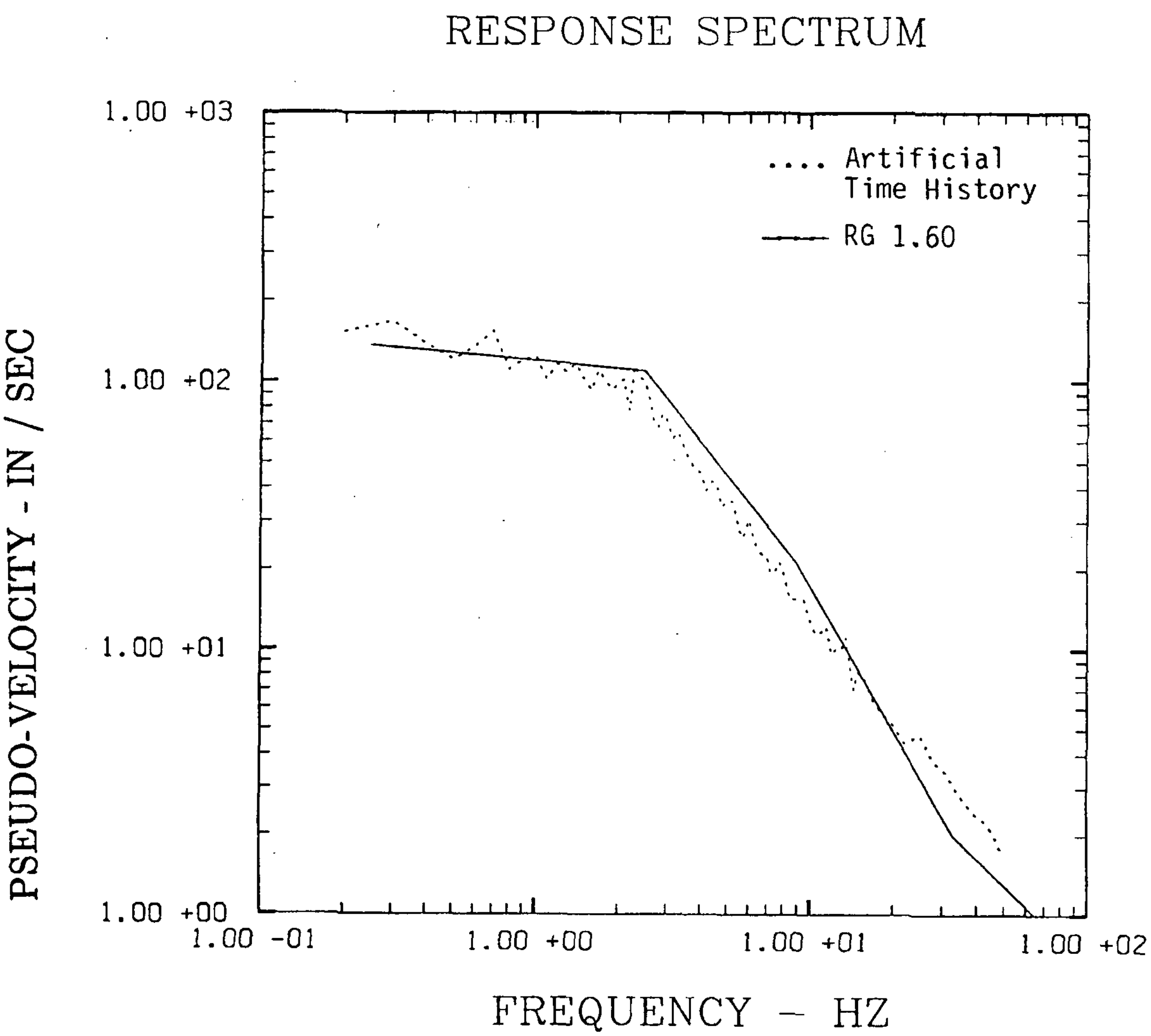


Figure 4. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM ASSOCIATED WITH PSD2 AND ENVELOPE FUNCTION A COMPARED TO RG 1.60

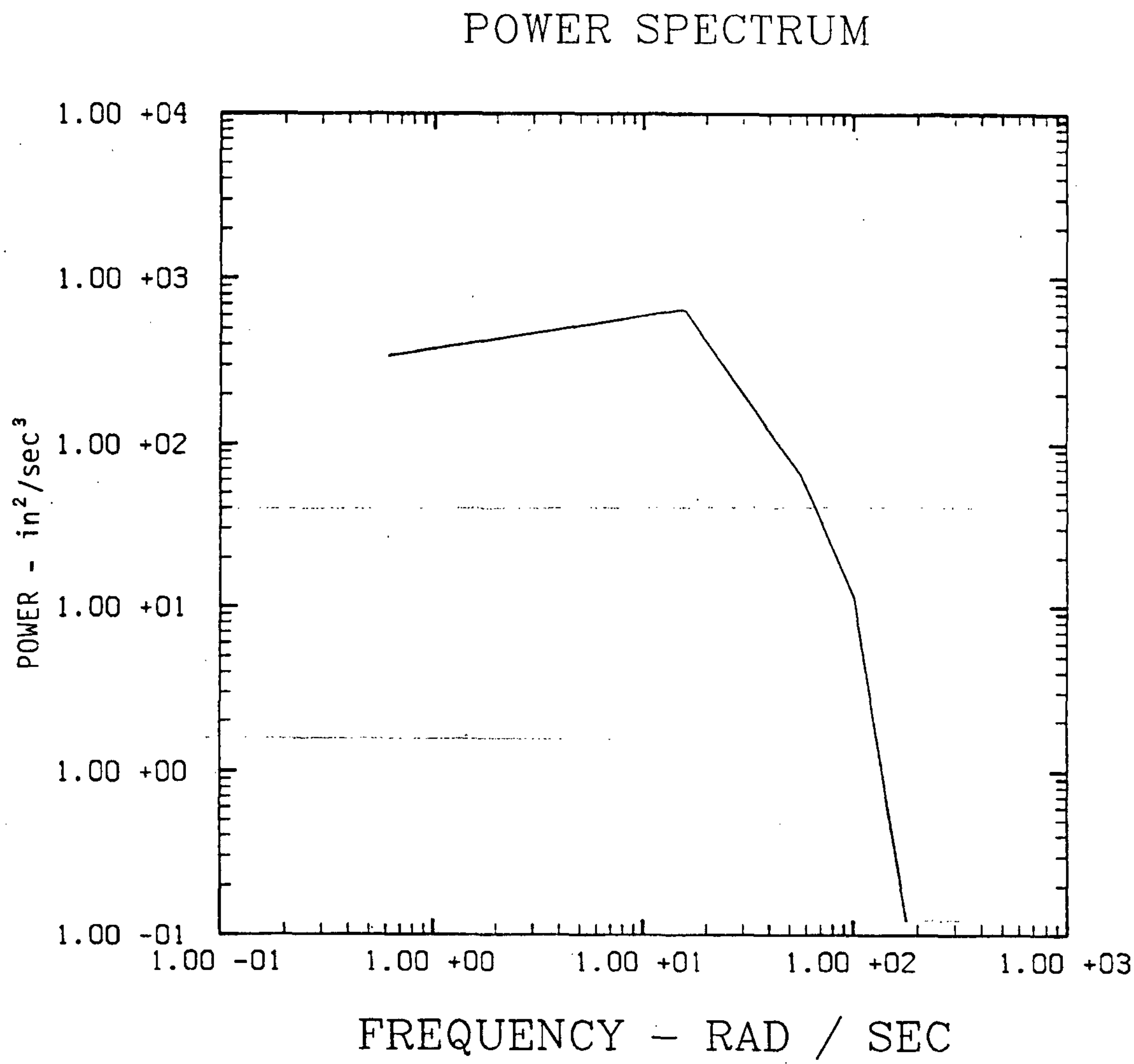


Figure 5. RECOMMENDED MINIMUM POWER SPECTRAL DENSITY REQUIREMENT (PSD3)

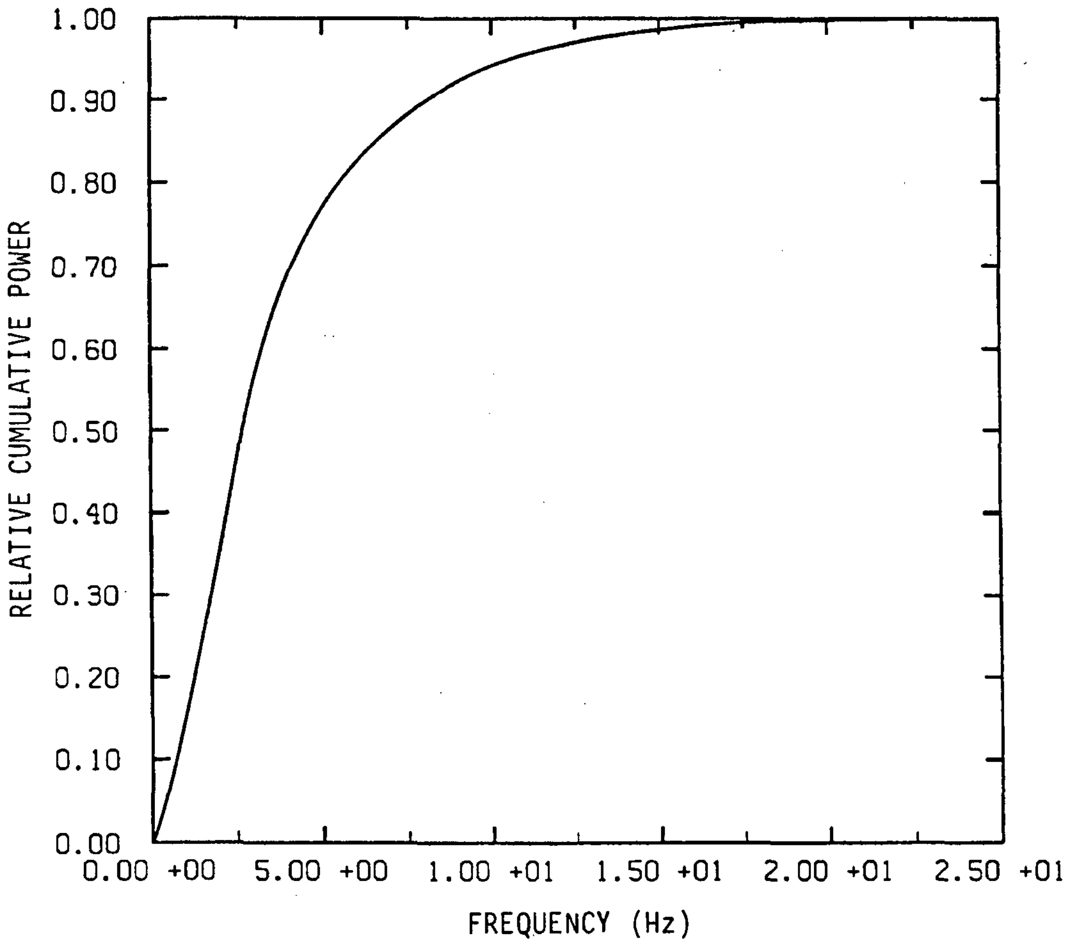


Figure 6. RELATIVE CUMULATIVE POWER FOR PSD3

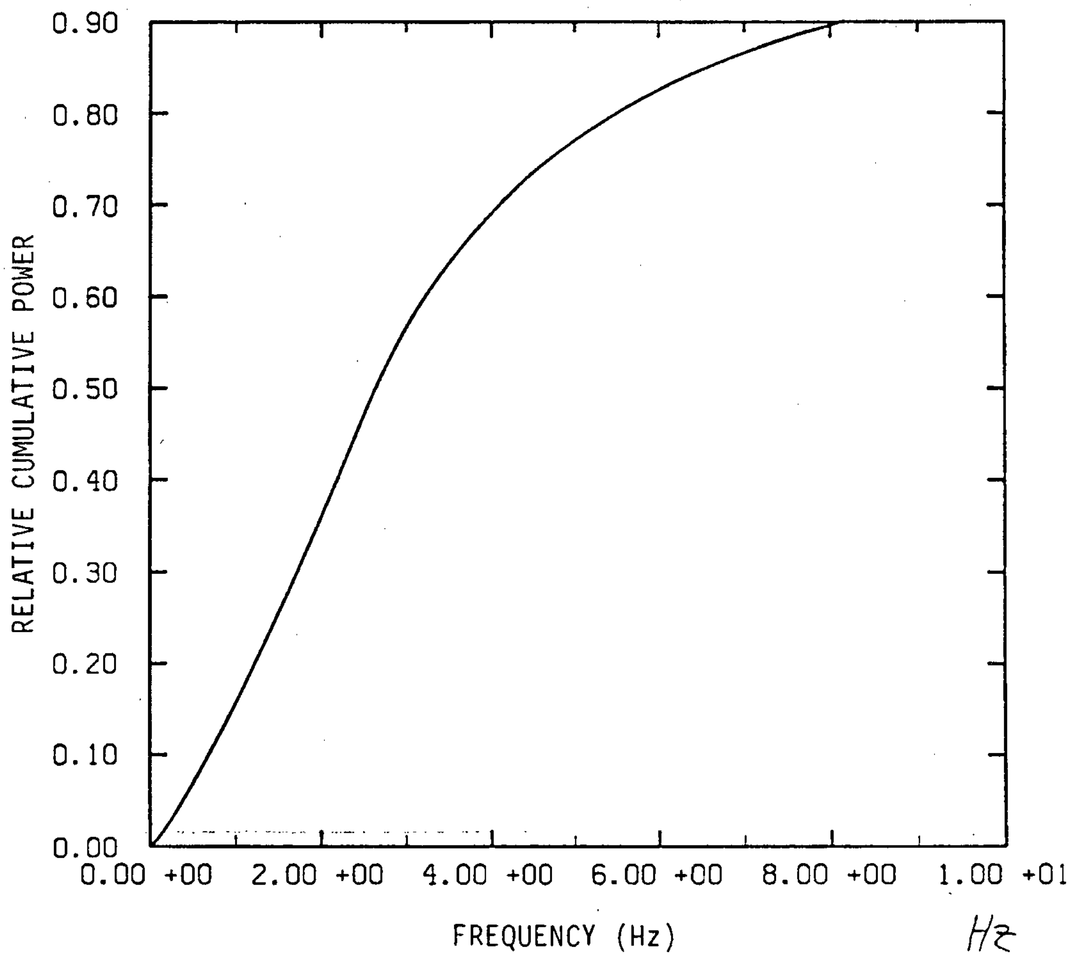


Figure 7. ALTERNATE PLOTTING OF RELATIVE CUMULATIVE POWER FOR PSD3

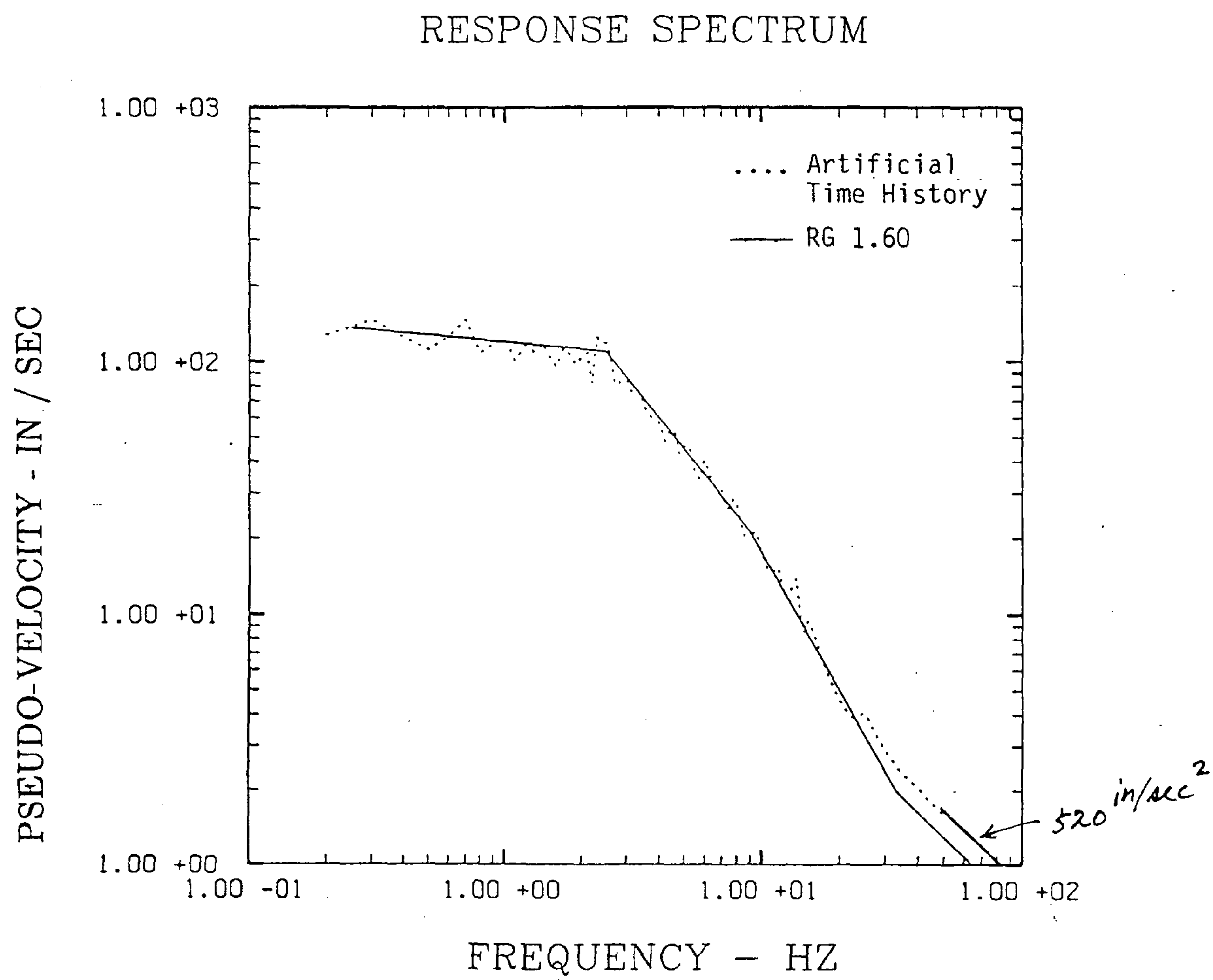


Figure 8. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM ASSOCIATED WITH PSD3 AND ENVELOPE FUNCTION A COMPARED TO RG 1.60

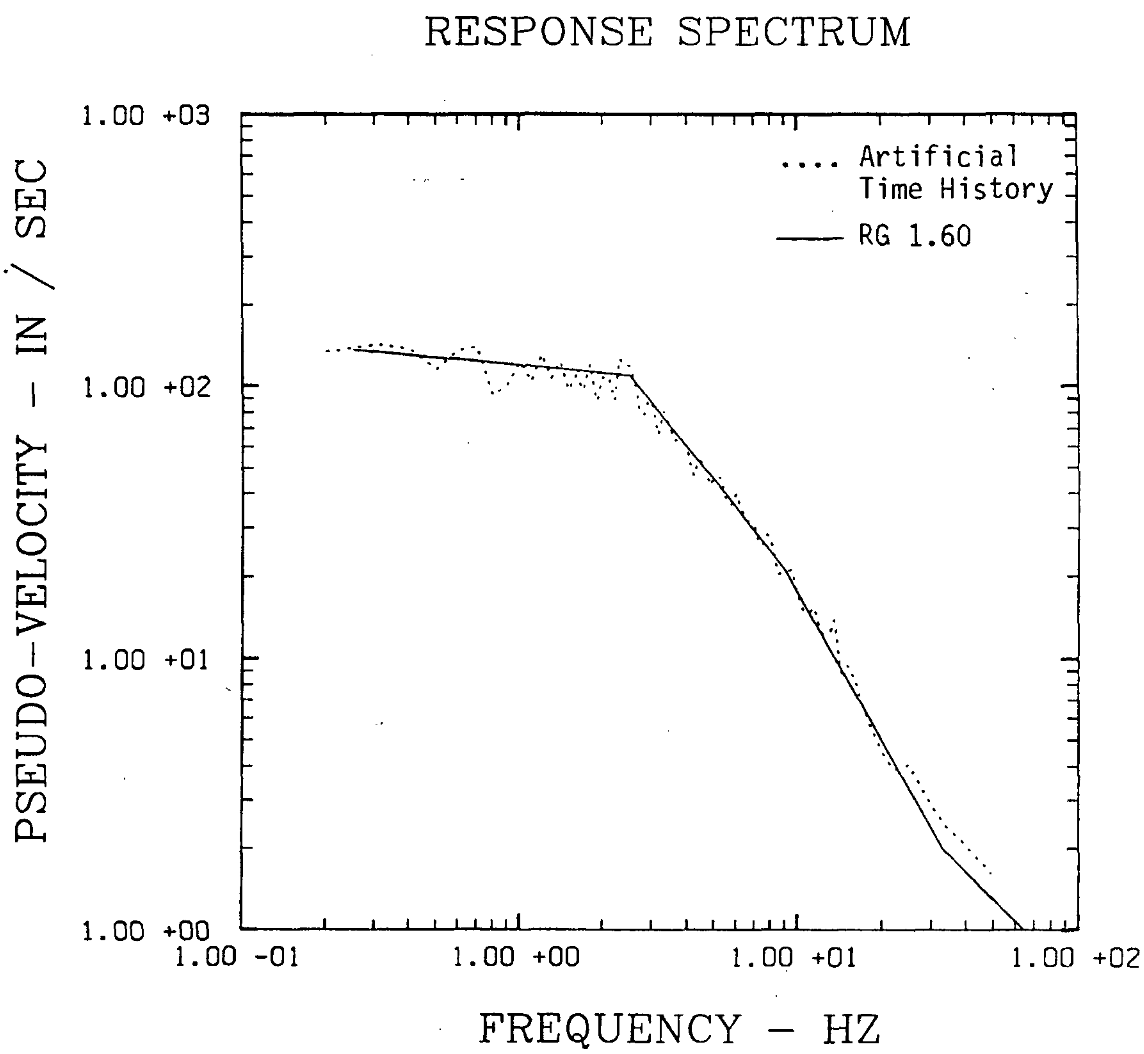


Figure 9. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM ASSOCIATED WITH PSD3 AND ENVELOPE FUNCTION B COMPARED TO RG 1.60

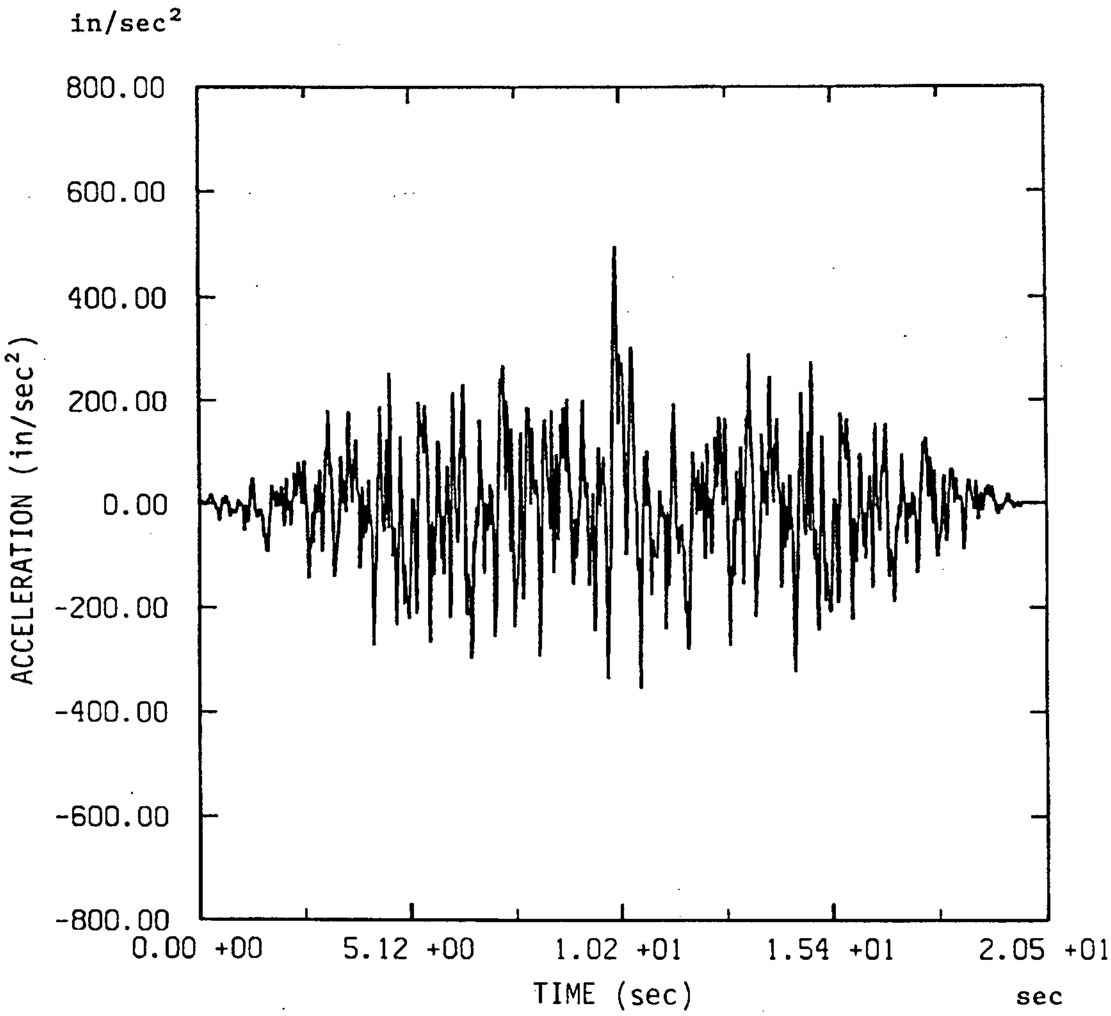


Figure 10. TIME HISTORY OBTAINED FROM PSD3 AND ENVELOPE FUNCTION A

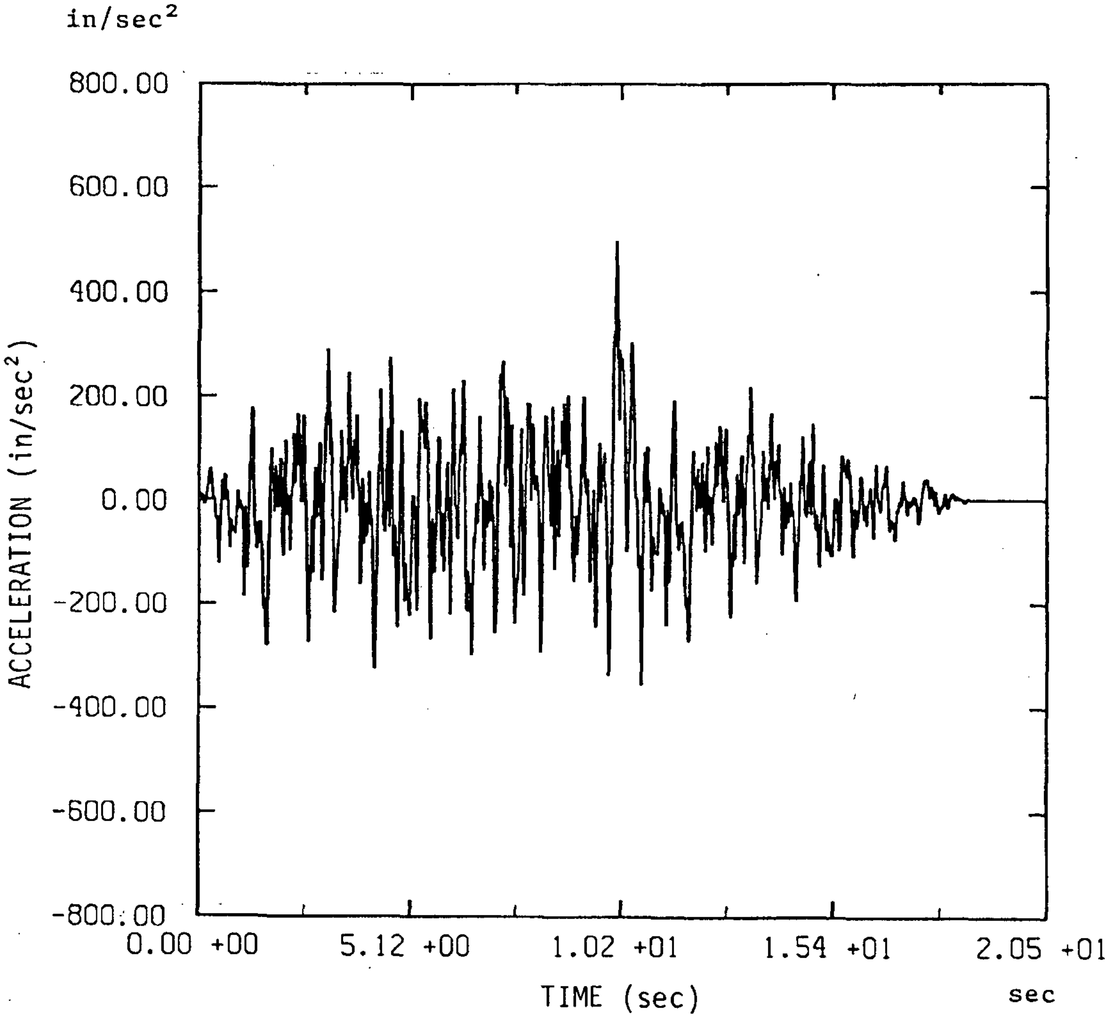


Figure 11. TIME HISTORY OBTAINED FROM PSD3 AND ENVELOPE FUNCTION B

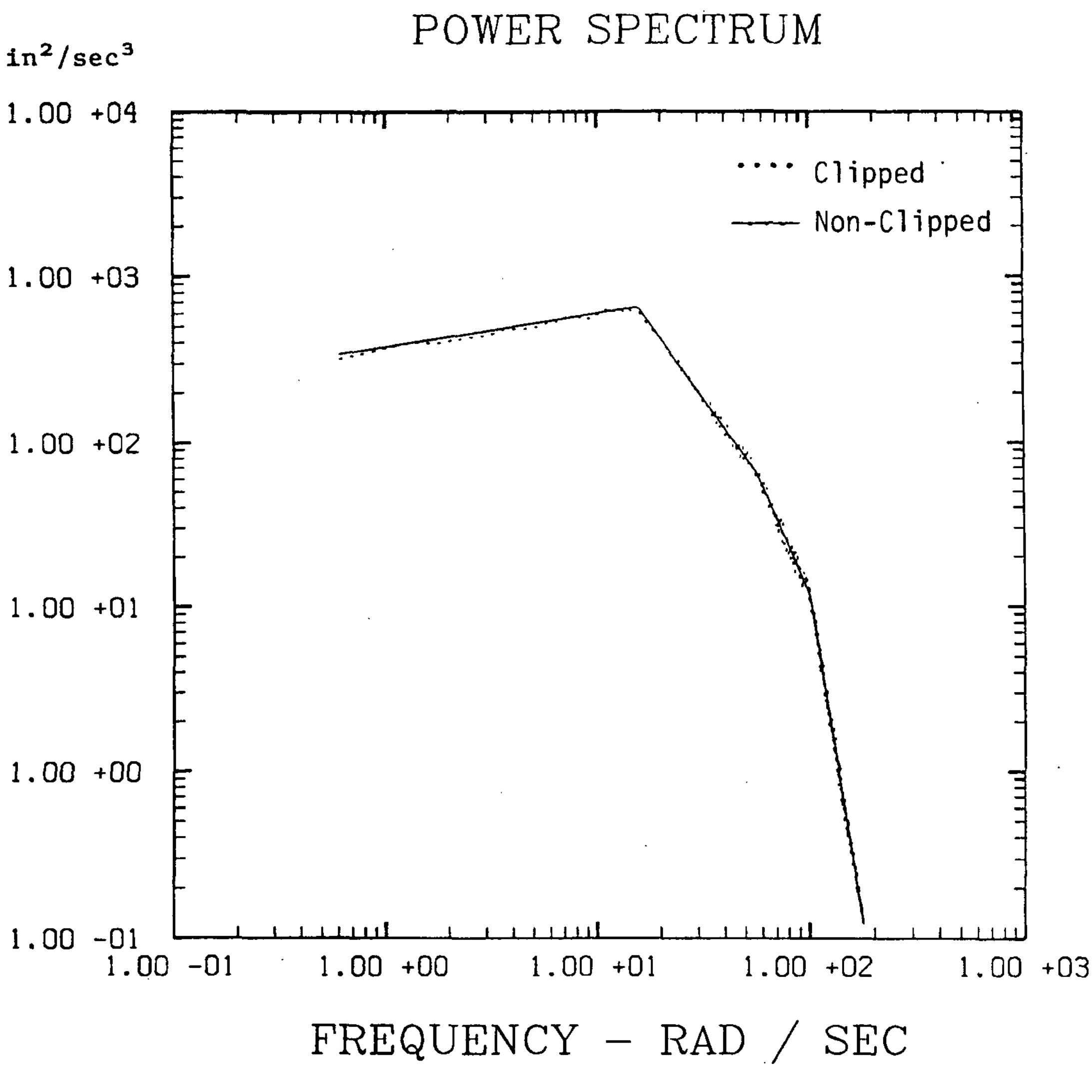


Figure 12. PSD OBTAINED FROM CLIPPED VERSUS NON-CLIPPED TIME HISTORIES

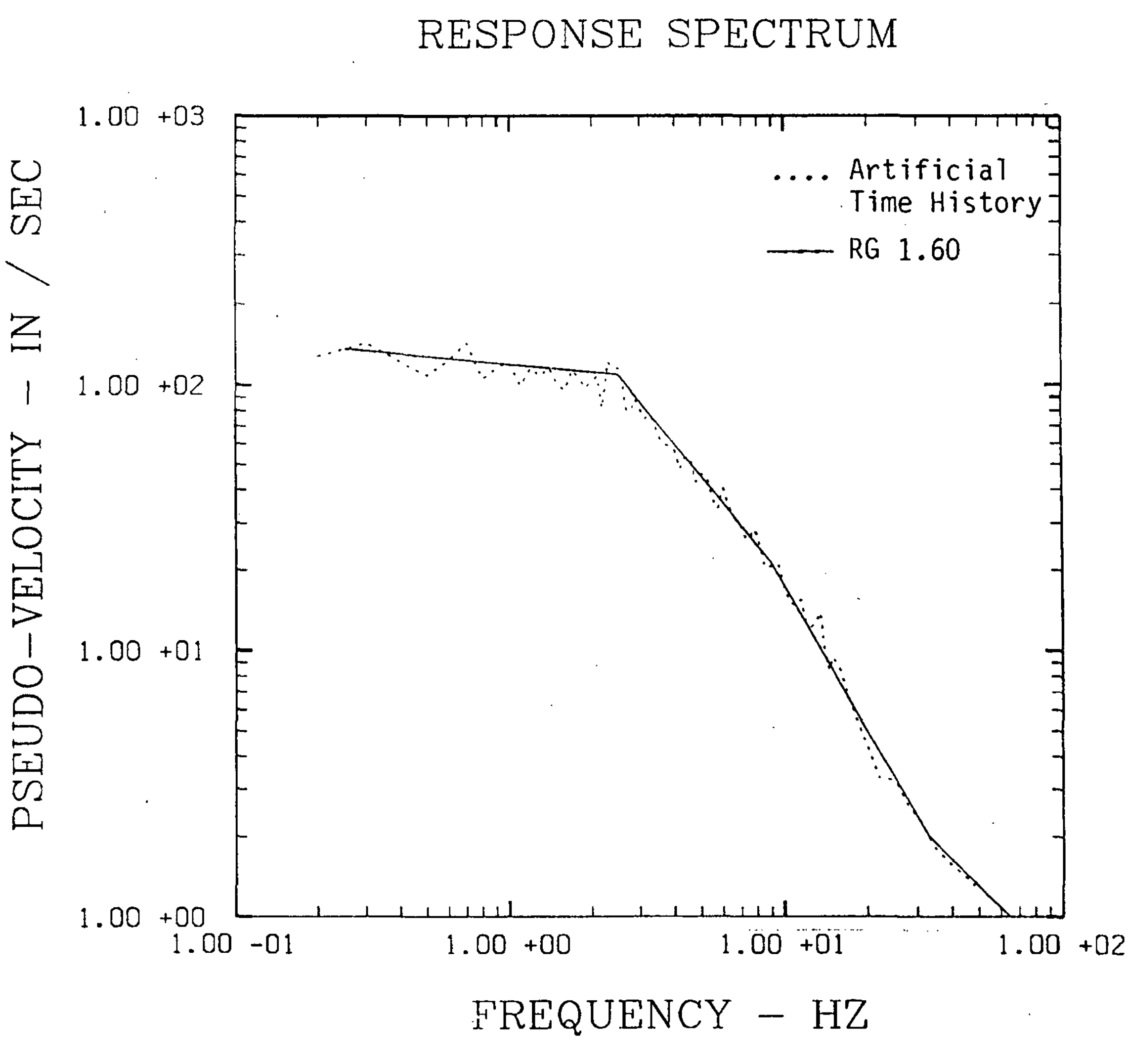


Figure 13. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM OBTAINED FROM PSD3 AND ENVELOPE FUNCTION A WITH TIME HISTORY CLIPPED AT 1.0g

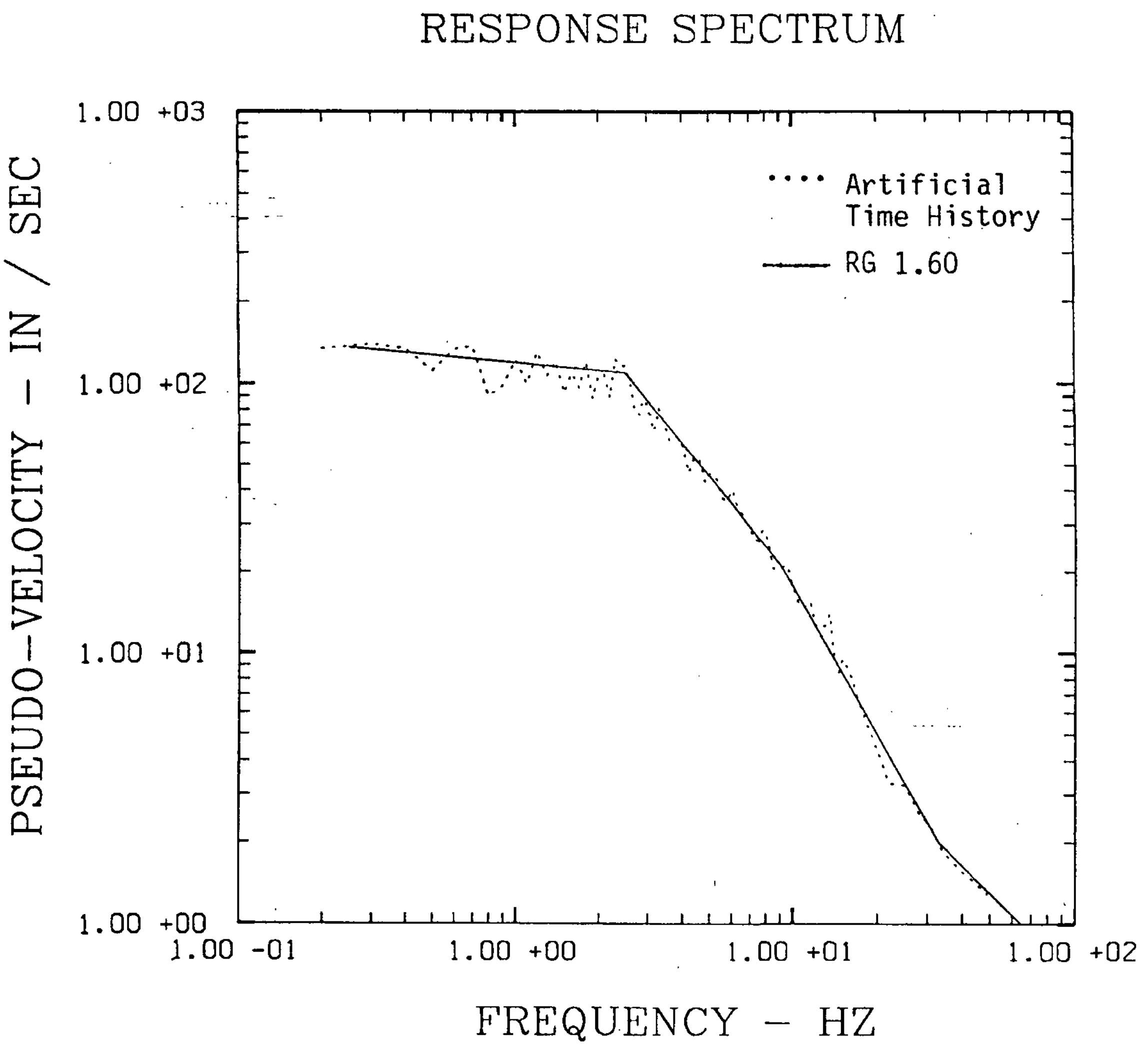


Figure 14. 2% DAMPED PSEUDO RELATIVE VELOCITY RESPONSE SPECTRUM OBTAINED FROM PSD3 AND ENVELOPE FUNCTION B WITH TIME HISTORY CLIPPED AT 1.0g

RPK Robert P. Kennedy

Structural Mechanics Consulting, Inc.

18971 Villa Terrace, Yorba Linda, CA 92686 • (714) 777-2163

February 18, 1989

Dr. A. J. Philippacopoulos
Brookhaven National Laboratories
Bldg. 129
Upton, NY 11973

Re: Comments on Proposed Revisions to Standard Review Plan Seismic Provisions

Dear Mike:

Enclosed are the original copies for incorporation into your report of my report on the subject material and the report by Professor Shinozuka and myself on PSD functions compatible with R.G. 1.60.

Very truly yours,

Bob

Robert P. Kennedy

cc. Prof. Shinozuka

88C1516
0503G

APPENDIX C

COMMENTS ON PROPOSED REVISIONS TO STANDARD REVIEW PLAN SEISMIC PROVISIONS

PREPARED FOR

Brookhaven National Laboratory
Building 129
Upton, NY 11973
Attn: A. J. Philippacopoulos

March 29, 1989

by

J.D. Stevenson
Stevenson and Associates
9217 Midwest Avenue
Cleveland, OH 44125
(216) 587-3805

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1.0 INTRODUCTION

Around May 1988, the U.S. Nuclear Regulatory Commission (NRC) issued Proposed Revision 2 to sections 2.5.2, 3.7.1, 3.7.2, and 3.7.3 of their Standard Review Plan, NUREG-0800 for public review and comment. Prior to August 30, 1988, comments had been received from five organizations (Sargent & Lundy, Westinghouse, Stevenson and Associates, Duke Power Company, and General Electric). As a contractor to the U.S. Nuclear Regulatory Commission, Brookhaven National Laboratory has been requested to assist the NRC in resolving these public comments. As part of this effort, Brookhaven has formed a panel of consultants in the field of seismic analysis and design of nuclear power plants to review these public comments and to recommend resolutions. The comments contained in this report are the result of my serving as a member of that panel.

This report in Section 2.0 is meant to document the changes to the proposed text of SRP Section 3.7.3 based on the detailed review performed by Stevenson and Associates (J.D. Stevenson) of the public comments relative to proposed changes to the Standard Review Plan Section 3.7.3. In Section 3.0 is contained J.D. Stevenson's general review comments concerning public comments to proposed changes to the Standard Review Plan Sections 2.2.5, 3.7.1 and 3.7.2. In several cases specific sections of the ASCE Standard 4-86 are recommended for incorporation into the revised SRP by reference. In Attachment 1 to this report J.D. Stevenson has identified a number of areas that in his opinion should be the subject a continuing effort on the part of the NRC to improve and rationalize the SRP sections devoted to seismic design and analysis. In Attachment 2 to this report are contained comments relative to the use of a PSD function in the generation of Design Basis Response Spectra.

2.0 REVIEW OF PUBLIC COMMENTS TO PROPOSED CHANGES TO THE STANDARD REVIEW PLAN SECTION 3.7.3

2.1 Suggested Modification to Change to SRP 3.7.3.II.12, Buried Piping Conduit and Tunnels, Proposed by Sargent and Lundy and J.D. Stevenson

1. Reason for Proposed Change

Sargent and Lundy has proposed the addition of another reference to item 12 Subsection II to Section 3.7.3 concerning acceptance criteria for buried piping conduit and tunnels.

Stevenson and Associates has proposed the use of industry standards by reference where possible in the proposed changes to the S.R.P.

2. Discussion

The reference proposed by S&L is contained in the list of references of the commentary (Ref. 3.5-4) to ASCE Standard ASCE 4-86, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary of Standard for Seismic Analysis of Safety Related Nuclear Structures," ASCE September 1986.

ASCE Standard 4-86, Section 3.5.2 presents in detail design procedures and acceptance criteria to be used in seismic design and analysis of Category I Buried Piping, Conduits and Tunnels. It has been stated NRC policy to use existing industry standards by reference in Regulatory Guides and Standard Review Plans where the industry standard is acceptable in total. Where there is any disagreement on the part of the NRC Staff as to the content of such standards, these disagreements should be identified and published in the appropriate Regulatory Guide or Standard Review Plan. In my review of the text of the proposed change to SRP Section 3.7.3 II 12, to incorporate the ASCE 4-86 Standard by reference I see no conflict between the industry position and the current NRC Staff position.

3. Recommendation

It is recommended that a change be made to the currently proposed SRP Section 3.7.3.II.12 as follows:

"12. Category I Buried Piping, Conduits, and Tunnels

For Category I buried piping, conduits, tunnels, and auxiliary systems, the following items should be considered in the analysis:

(a) Two types of ground-shaking-induced loadings must be considered for design.

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

(ii) Lateral earth pressures and ground water effects acting on structures.

(b) The effects of static resistance of the surrounding soil on piping deformations or displacements, differential movements of piping anchors, bent geometry and curvature changes, etc., should be adequately considered. Procedures utilizing the principles of the theory of structures on elastic foundations are acceptable.

(c) When applicable, the effects due to local soil settlements, soil arching, etc., should also be considered in the analysis.

(d) Actual methods used for determining the design parameters, methods of analysis and acceptance criteria associated with seismically induced transient relative deformations are reviewed and accepted on a case-by-case basis. Additional information, for guidance purposes only, can be found in Analysis Standards and Commentary Sections 3.5.2 of Ref. 7."

All other text in the currently proposed SRP Section 3.7.3.II.12 text is to be deleted.

2.2 <u>Suggested Changes to SRP Section 3.7.3.II.1 Seismic Analysis Methods in Response to J.D. Stevenson General Comment Concerning Use of Industry Standards</u>

1. <u>Statement of Proposed Change</u>

Stevenson and Associates has proposed use of references to industry standards where available and appropriate instead of detailed "how to" text in the SRP.

2. <u>Discussion</u>

The ASME Boiler and Pressure Vessel Code Section III explicitly permits the use of plastic, limit or inelastic analysis (e.g. NB 3213.21, NB 3213.22, NB 3228, NB 3653.6, NF 3340, A-9000, F-1321.4, F-1321.5, F-1321.6, F-1321.7, F-1322.1, F-1340). Since subsystem components constructed to the requirements of the ASME BPVC Section III can be designed using other than linear elastic analysis, this permitted exception to elastic analysis should be so stated in the SRP.

3. <u>Recommendation</u>

It is recommended that a change be made to the proposed SRP Section 3.7.3.II.1 as follows:

"1. <u>Seismic Analysis Methods</u>

The acceptance criteria provided in SRP Section 3.7.2 Subsection II.1 are applicable."

change to:

"1. <u>Seismic Analysis Methods</u>

The acceptance criteria provided in SRP Section 3.7.2 Subsection II.1 are applicable. In the design and analysis of subsystem components, non-linear analysis is acceptable consistent with the provisions of applicable Codes and Standards (e.g. Ref. 8) subject to review on a case-by-case basis."

2.3 Suggested Changes to SRP Section 3.7.3.II.3, Procedures Used for Analytical Modeling in Response to J.D. Stevenson General Comment Concerning Use of Industry Standards

1. Statement of Proposed Change

Stevenson and Associates has proposed use of references to industry standards where available and appropriate instead of detailed "how to" text in the SRP.

2. Discussion

Section 3.1.7 of the ASCE Standard 4-86 contains specific dynamic coupling criteria which is more detailed than the proposed SRP text.

3. Recommendation

It is recommended that the proposed changed SRP text in Section 3.7.2.II.3.b which is referenced in Section 3.7.3.II.3 reference the ASCE Standard as follows:

current text:

"b. Decoupling Criteria for Subsystems

It can be shown, in general, that frequencies of systems and subsystems have negligible effect on the error due to decoupling. It can be shown that the mass ratio, R_m, and the frequency ration, R_f, govern the results where R_m and R_f are defined as:

$$R_m = \frac{\text{Total mass of the supported subsystem}}{\text{Total mass of the supporting system}}$$

$$R_f = \frac{\text{Fundamental frequency of the supported subsystem}}{\text{Dominant frequency of the support motion}}$$

The following criteria are acceptable:

(i) If $R_m < 0.01$, decoupling can be done for any R_f.

(ii) If $0.01 \leq R_m \leq 0.1$, decoupling can be done if $0.8 \geq R_f \geq 1.25$

(iii) If $R_m > 0.1$, an approximate model of the subsystem should be included in the primary system model.

If the subsystem is rigid compared to the supporting system, and also is rigidly connected to the supporting system, it is sufficient to include only the mass of the subsystem at the support point in the primary system model. On the other hand, in case of a subsystem supported by very flexible connections, e.g., pipe supported by hanger, the subsystem need not be included in the primary model. In most cases the equipment and components, which come under the definition of subsystems, are analyzed (or tested) as a decoupled system from the primary structure and the seismic input for the former is obtained by the analysis of the latter. One important exception to this procedures is the reactor coolant system, which is considered a subsystem but is usually analyzed using a coupled model of the reactor coolant system and primary structure."

change to:

"b. <u>Decoupling Criteria for Subsystems</u>

If the subsystem is rigid compared to the supporting system, and also is rigidly connected to the supporting system, it is sufficient to include only the mass of the subsystem at the support point in the primary system model. On the other hand, in case of a subsystem supported by very flexible connections, e.g., pipe supported by hanger, the subsystem need not be included in the primary model. In most cases the equipment and components, which come under the definition of subsystems, are analyzed (or tested) as a decoupled system from the primary structure and the seismic input for the former is obtained by the analysis of the latter. One important exception to this procedures is the reactor coolant system, which is considered a subsystem but is usually analyzed using a coupled model of the reactor coolant system and primary structure.

To determine whether or not dynamic coupling of systems and subsystems is significant, hence, must be considered in analytical modeling, the criteria contained in Section 3.1.7 of Ref. 7 is acceptable."

2.4 <u>Suggested Change to SRP 3.7.3.II.14, Methods for Seismic Analysis of Above Ground Tanks, In Response to Stevenson and Associates Comment Concerning the Use of Industry Standards</u>

1. <u>Reason for Proposed Change</u>

Stevenson and Associates has proposed use of references to industry standards where available and appropriate instead of detailed "how to" text in the SRP.

2. <u>Discussion</u>

The ASCE Standard 4-86 in Sections 3.5.4 of the Analysis Standards and Commentary has specific design procedures, analysis method and acceptance criteria applicable to seismic analysis of vertical above ground tanks. The ASCE Standards 4-86 provides significantly more detail and guidance than does the current SRP text.

3. <u>Recommendations</u>

Delete the current proposed text of SRP Section 3.7.3.II.14 and replace with the following:

"14. <u>Methods for Seismic Analysis of Above Ground Tanks</u>

Most aboveground fluid-containing vertical tanks do not warrant sophisticated, finite element, fluid-structure interaction analyses for seismic loading. However, the commonly used alternative of analyzing such tanks by the "Housner-method" (Ref. 4) may be inadequate in some cases. The major problem is that direct application of this method is consistent with the assumption that the combined fluid-tank system in the horizontal impulsive mode is sufficiently rigid to justify the assumption of a rigid tank. For the case of flat bottomed tanks mounted directly on their base, or tanks with very stiff skirt supports, the assumption leads to the usage of a spectral acceleration equal to the zero-period base acceleration. Recent evaluation techniques (Refs. 5 and 6) have shown that for typical tank designs the frequency for this fundamental horizontal impulsive mode of the tank shell and contained fluid is generally between 2 and 20 Hz. Within this regime, the spectral acceleration is typically far greater than the zero-period acceleration. Thus, the assumption of a rigid tank could lead to inadequate design loadings. The SSI effects are also very horizontal and vertical motions.

The acceptance criteria, modeling and analytical procedures contained in Analyses Standards and Commentary of Sections 3.5.4 of Ref. 7 are acceptable."

2.5 <u>Add References to SRP Sections 3.7.2 and 3.7.3 to Accommodate Recommended Changes Contained in Sections 2.1 - 2.4.</u>

The following reference should be added to SRP Section 3.7.2.VI, References on pages 3.7.2 - 23.

7. <u>ASCE Standard, ASCE 4-86 "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," American Society of Civil Engineers, September 1986.</u>

The following references should be added to SRP Section 3.7.3.VI, References on pages 3.7.3 - 12.

7. <u>ASCE Standard, ASCE 4-86 "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," American Society of Civil Engineers, September 1986.</u>

8. <u>ASME BPVC Section III, "Rules for Construction of Nuclear Power Plant Components," American Society of Mechanical Engineers Boiler and Pressure Vessel Code, July 1986.</u>

3.0 GENERAL REVIEW OF PUBLIC COMMENTS TO PROPOSED CHANGES TO THE STANDARD REVIEW PLAN SECTIONS 2.2.5, 3.7.1 and 3.7.2.

I have reviewed the comments from the organizations listed in Section 1.0 of this report as well as the draft comments on Proposed Revisions to Standard Review Plan Seismic Provisions prepared for Brookhaven National Laboratory by R.P. Kennedy dated December 1988. I concur with Dr. Kennedy's recommendations with some additional clarification as suggested in this Section.

3.1 Time History Strong Motion Duration and Envelop Function

I concur with Dr. Kennedy's comments regarding time history strong motion duration. However, I would recommend a specific time history strong motion duration criteria as currently contained in industry standards be stated as being acceptable to the NRC. Kennedy has recommended a time history strong motion duration of between 5.0 and 16.0 seconds to be compatible with the R.G. 1.60 Spectra. Subparagraph N1212.2, Duration of Time History of Appendix N to Section III of the ASME Boiler and Pressure Vessel Code recommends a minimum strong motion duration of 6.0 seconds. Based on Kennedy's recommendation and that contained in the ASME Code, it is recommended that the SRP Section 3.7.1.II.1.b. be modified to include a time history strong motion duration, t_D within the range of $6.0 \leq t_D \leq 16.0$ seconds.

3.2 Damping Requirements

Dr. Kennedy has recommended use of ASCE Standard 4-86 Section 3.1.2.2 proposed by Sargent and Lundy and Kennedy to define damping requirements as a function of stress level. I concur with the recommended use of Stress Level 2 damping valves to generate floor response spectra be limited to cases where concrete stress is greater than 50 percent of ultimate strength of concrete and also greater than 50 percent of yield stress in the steel.

ATTACHMENT 1

Recommendations for Future Revisions of Sections
of the Standard Review Plan Dealing with
Seismic Design and Evaluation of Nuclear Power Plants

There have been a number of NRC staff recommendations, consultant reports and NRC research reports which have made recommendations concerning changes to NRC seismic design requirements covered by SRP Sections 2.2.5, 3.7.1, 3.7.2 and 3.7.3. Many if not most of these recommendations are not contained in the current proposed changes to the SRP Sections 2.2.5, 3.7.1, 3.7.2 and 3.7.3.

In my opinion a number of technical areas covered by SRP Sections 2.2.5 and 3.7 still require NRC Staff review to develop more consistent, rational and realistic seismic design and evaluation requirements for structural systems and subsystems.

It must be understood that "conservative" design for inertia seismic loads which is the focus of current NRC seismic design and evaluation requirements covered in SRP Sections 2.2.5 and 3.7 does not necessarily lead to "conservative" overall design.

In general optimum design of elevated temperature, high energy structural subsystems tries to minimize the amount of restraint in such systems in order to minimize stress induced in the system by restraint of free end displacement caused by thermal expansion, support motions and water and steam hammmer and sudden valve operation effects. Conservative design for seismic inertia effects tends to increase restraint hence overall operating stress levels in such systems.

In addition conservatively high seismic loads on structural systems (buildings) require use of structural joints designed to transfer large loads. This discourages use of ductile joint details because of the resultant congestion (e.g. ACI 318 Appendix A). Earthquake response experience shows ductile joint detailing to be very effective and necessary to resist significant structural damage in strong motion earthquakes.

In Table 1 is presented a list of technical areas suggested actions and associated references which should receive continued NRC Staff review to improve the seismic and overall design basis of nuclear power plant systems and subsystems.

Table 1 - Summary of Technical Areas Related to Seismic Design Requiring Further NRC Design Criteria Development

	Area		Action	Reference
1.	Modify (Increase) Seismic Damping Values Used in Design of Subsystems (Piping)	a)	Increase Pipe Damping Values to ASME Code Case N-411	1, 2, 3
		b)	Minimize Caveats Associated with Use of ASME CC N-411	
		c)	Increase Damping for Heavily Insulated Pipe	
2.	Decouple OBE from SSE and Eliminate the OBE as a Design Requirement for Low Seismicity Sites	a)	Change or Clarify Wording of 10CFR 100 Appendix A to Permit Decoupling of OBE from SSE	1, 4
		b)	Eliminate OBE as a Design Basis for Low Seismicity Sites SSE PGA $\leq$ 0.15g	
3.	Use of a Median or Uniform Hazard Spectra Rather than Variable Mean Plus One Standard Deviation Design Response Spectra	a)	R.G. 1.60 Contains a Variable Design Margins as a Function of Frequency with a Median Value Defined at the High Frequency Limit (33Hz) and Mean Plus One Standard Deviation Defined in the Amplified Frequency Range 2-10 Hz	5
		b)	Item 1 under SRP 2.5.2.6 Requires Generation of Mean Plus Standard Deviation (84 Percentile) Spectra. Item 5 under SRP 2.5.2.6 Requires Generation of Uniform Hazard Spectra at Various Probability Levels. NRC Should Permit Use of a UHS instead of 84th Percentile Spectra at a Probability Level Acceptable to NRC.	
4.	Permit Limited Amounts of Inelastic Response of Systems and Subsystems	a)	Consultants have Recommended Allowing Limited Amounts of Non-linear Response Behavior (Global Ductility > 1.0) in seismic design of Systems as a Function of Importance to Safety	5, 6
		b)	Add Additional Constraints Based on Ductility Capabilities of Systems	

Table 1 - Summary of Technical Areas Related to Seismic Design Requiring Further NRC Design Criteria Development (Continued)

	Area		Action	Reference
		c)	Provide Explicit Global Ductility Limits for Systems and Subsystems as a Function of Importance to Safety and Ductility Capabilities.	
5.	Permit Balanced Seismic Design such that Seismic Capacities of Subsystems are Not Required to be Significantly Greater than the Structural System that Houses or Supports Them.	a)	Institute a Design Margin Review to Compare Seismic Capabilities of Subsystems to the System Housing or Supporting Them.	
6.	Reconcile Results of Recent Seismic Tests of Subsystem (Piping Systems) to Insure Rational Seismic Design Margins are Being Required.	a)	Consider Changes in Ductility and Damping Parameters to Assure Rational Seismic Design Margins (e.g. 1.5-2.0 against failure for the SSE) Are Being Maintained.	
7.	Use of Bounding or Threshold Damage Seismic Spectra to Design Safety Related and Evaluate Class 2 (2 over 1 Issue) to Assure They Do Not Fail and Endanger Class 1 Components in Their Proximity	a)	Recent Comprehensive Experience Data on the Behavior of Structural Systems and Subsystems in Strong Motion Earthquake and in Tests Indicate That There Are Threshold Spectral Values before Damage Results. Use of These Threshold Damage Spectra Together with Layout and Detailing Caveats Should Be Permitted in Design of Certain Types and Classes of Systems and Subsystems.	7
		b)	Threshold Damage Spectra Procedures Should Be Allowed in the Evaluation Class 2 Subsystems to Insure They Do Not Fail Under Seismic Loads.	
8.	Redefinition of High Frequency Induced Seismic Inertia Stresses as Secondary	a)	Permit Limited Application of ASME Code Cases N451 and N462 and to Components Other Than Piping	8,9
		b)	Seismic Induced Loads above About 10 Hz Tend to Be Displacement Limited Hence Develop Secondary Stresses.	
9.	Permit Use of Vibration Acceptance Criteria in Terms of Velocity or Displacement to Be Applied to Seismic Design Adequacy	a)	Permit the Application of ANSI/ASME OM3-1982 Criteria Limits for Vibration Be Extended to Include High Stress Low Cycle Conditions Associated with Earthquake Response	10

TABLE 1 REFERENCES

(1) Seismic Design Task Group "Report of the U.S. Nuclear Regulatory Commission Piping Review Committee - Summary Piping Review Committee Conclusions and Recommendations," NUREG-1061 Vol. 5 U.S. Nuclear Regulatory Commission, April 1985.

(2) PVRC Committee, "Technical Position on Damping Values for Piping - Interim Summary Report," WRC Bulletin 300, Welding Research Council, December 1984.

(3) Bitner, J.L. et. al. "Technical Position on Damping Values for Insulated Pipe - Summary Report," WRC Bulletin 316, Welding Research Council, July 1986.

(4) Seismic Design Task Group "Report of the U.S. Nuclear Regulatory Commission Piping Review Committee - Evaluation of Seismic Design -A Review of Seismic Design Requirements for Nuclear Power Plant Piping," NUREG-1061 Vol. 2 U.S. Nuclear Regulatory Commission, April 1985.

(5) Newmark, N.M, and Hall, W.J. "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," NUREG/CR 0098, U.S. Nuclear Regulatory Commission, May 1978.

(6) Coats, D.W., "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," NUREG/CR 1161 Lawrence Livermore Laboratory, May 1980.

(7) Senior Seismic Review and Advisory Panel (SSRAP) "Use of Seismic Experience and Test Data to Show Ruggedness of Equipment in Nuclear Power Plants," (Draft) Seismic Qualification Utility Group and USNRC, August 1988.

(8) ASME Boiler and Pressure Vessel Code Case N-451 "Alternate Rules for Analysis of Piping Under Seismic Loading, Class 1, 1987.

(9) ASME Boiler and Pressure Vessel Code Case N-462, "Alternate Rules for Analysis of Piping Under Seismic Loading, Class 2 and 3," 1983.

(10) ANSI/ASME OM3-1982, "Requirements for Preoperational and Initial Start-up Vibration Testing of Nuclear Power Plant Piping Systems," ASME, 1982.

ATTACHMENT 2

Comments Concerning the Application of PSD Functions to the Generation of Design Basis Response Spectra

Comment 1 - High Frequency Power of the Target PSD Is Too High

The Kanai-Tajimi Power Spectral Density (PSD) function form has a shape identical to the response of a single resonance system due to a white noise input. This is true in general at a specific site.

Nuclear Regulatory Guide 1.60 response spectra, on the other hand, are enveloped from an ensemble of response spectra at various sites. The enveloped response spectrum has a much broader energy content than any single site. Trying to fit a single Kanai-Tajimi form to the PSD consistent with NRC 1.60 spectra, event though it fits well at the low frequency end where most of the power lies, results in the use of high damping value.

The PSD at the high frequency end, in this case greater than about 10 Hz, decays must slower than typical single site PSDs due to the large damping value. A more sophisticated function form or some attenuation function should be applied to the high frequency power.

Comment 2 - Comparison of PSD

To compare the PSD of a time history to the target PSD, the Procedure recommended in the proposed revision, calculating at frequency spacing of 0.05 Hz and perform a three point moving average, is very difficult, if not impossible, to achieve. Due to the statistical error in the PSD estimate, the PSD will still be very spiky after the moving average.

From random vibration theory, the standard deviation of the raw PSD estimate is approximately equal to the mean value. After the three point moving, the ratio of standard deviation to mean, or the normalized random error, will be reduced to about 0.6, which is still too high to compare with the smooth target curve.

A more reasonable approach, which is also consistent with the previous section in the Standard Review Plan, is to compare the area under the calculated PSD and the target PSD at the same frequency intervals as the response spectrum comparison, whether it is from Table 3.7.1-1 or based on 10% spacing ratios. The acceptance criteria can be set up the same way, that "no more than five points of the spectra obtained from the time history should fall below, and no more than 10% below the target PSD."

The comparison of areas under the PSD, which becomes the Power Spectrum (PS), is well established in the industry to compare the effect of noise and vibrations.

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February 13, 1989

Dr. A. J. Philippacopoulos
Brookhaven National Laboratory
Building 129
Upton, NY 11973

Dear Mike:

Per our conversation on 8 February 1989, please find attached hereto my report containing my comments on proposed revisions to the Standard Review Plan seismic provisions. Specific recommended changes to the text of the proposed changes to the SRP are contained in Sections 2.0 and 3.0 of my report. I have also described in Attachment 1 to my report technical areas where I believe still require further NRC regulatory definition. In Attachment 2 are comments concerning the use of power spectral density functions in the generation of design response spectra discussed in SRP Section 3.7.1.II.1.b.

Please advise if you require any clarification of the material sent.

Sincerely,

John D. Stevenson

John D. Stevenson
President

JDS:ss

Enclosures

APPENDIX D

COMMENTS ON PROPOSED REVISIONS TO NRC STANDARD REVIEW PLAN

by

A. S. Veletsos

Prepared for

Brookhaven National Laboratory

Upton, Long Island, New York

January 1989

INTRODUCTION

The objectives of this report are:

1. To respond to the public comments concerning the U.S. Nuclear Regulatory Commission's Proposed Revision 2 for Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3 of their Standard Review Plan (SRP) on Seismic Design Criteria (Refs. 1 and 2); and
2. To comment on selected sections of the proposed revisions for which clarifications and minor adjustment are deemed to be desirable.

The subject matters addressed, along with the relevant sections of the SRP, are identified in the headings of the following sections. The comments are presented in the order of the sections to which they refer rather than the order of their importance.

SECTION 2.5.2 VIBRATORY GROUND MOTION

Definition of SSE and OBE, p. 2.5.2-1

I concur with the view expressed by Sargent and Lundy to the effect that the definitions for the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE) presented on p. 2.5.2-1 are not clear. On the other hand, I do not subscribe to the view that the requirements for the SSE should be liberalized.

The following changes in wording may alleviate the reservations expressed:

> "The Safe Shutdown Earthquake is the maximum credible earthquake which would induce the most severe ground motion at the plant site. This earthquake is determined from evaluations of the maximum earthquake potential for the site, giving due regard to the regional and local geology, seismisity, and characteristics of the subsurface materials involved. Safety related structures, systems and components are designed to remain functional under this earthquake."

> "The Operating Basis Earthquake defines the class of earthquakes which can reasonably be expected to affect the plant during its operating life. Those elements of the power plant which are

necessary for its continuous operation without undue risk to the health or safety of the public are designed to remain functional under the ground motions induced by this event."

In Sect. 2.5.2.7 of the SRP, the return period for the OBE is indicated to be "of the order of hundreds of years," whereas Sect. 3.7.3.II.B.2 requires that at least "five operating basis earthquakes" be assumed during the plant life. Are the two requirements consistent? Also, is there any correlation between the number of earthquakes referred to above and the minimum number of ground motion histories specified for purposes of dynamic analysis? I would think not.

Maximum Earthquake Potential, pp. 2.5.2-6 to 7

In recognition of the fact that the most severe ground motion for systems with different natural frequencies may be induced by different earthquakes, the last paragraph on p. 2.5.2-7 has been revised to refer to several earthquakes rather than a single one. For the same reason, the end of the first paragraph of Sect. 2.5.2.4 should be modified to read "...when the earthquake <u>or earthquakes</u> which would produce the maximum...<u>have</u> been determined."

Safe Shutdown Earthquake, pp. 2.5.2-12 to 14

1. The ground motion for the design earthquake in the SRP is specified indirectly in terms of a response spectrum rather than directly in terms of ground motion histories. In the most sophisticated of the recommended procedures, the design response spectrum is determined from analyses of a collection of appropriate ground motion records for the site. However, no guidance is given as to the minimum number of records required in this approach. It is recommended that this number be specified, or that, as a minimum, a statement be included to the effect that the number of ground motion records considered should be sufficiently large such that the resulting spectrum is reasonably broad banded and properly reflects the uncertainties of the problem.

2. It is not clear if the use of a reasonably large collection of appropriate earthquake ground motion records, as contrasted to the use of

a design spectrum, constitutes an acceptable basis for design. If this is indeed acceptable, then what should the minimum number of records be? If the required number is no smaller than that needed in the design spectrum approach, then clearly this is not a distinct option.

3. Item 2 at the top of p. 2.5.2-13 refers to the case in which the available set of ground motion records is not sufficiently large to determine the site-specific design spectrum. The requirements of this option require clarification. Incidentally, the recommended adjustments should provide for the effects of magnitude and epicentral distance in addition to those of fault mechanism, propagation path and local site conditions.

SECTION 3.7.1 SEISMIC DESIGN PARAMETERS

Use of Single and Multiple Time Histories, pp. 3.7.1-3 and 4

The objections to the use of multiple ground motion histories expressed in the public comments appear to have stemmed, in part, from a lack of clarity in the SRP of precisely what is intended in this regard.

Following discussions of this matter with Dr. Nilesh Choksi of NRC, I believe that the intent of the proposed provisions can more appropriately be stated as follows:

- If a single artificial, real or modified real ground motion history is employed, its response spectrum must match or exceed the design spectrum over the entire range of frequencies and damping values which are of interest. In addition, the ground motion history must satisfy the power spectral density (PSD) requirement examined in a later section of this report. The spectrum matching or enveloping requirements are identified on p. 3.7.1-10 of the SRP.

- If a collection of artificial, real or modified real ground motion histories is used, the response spectra for the individual records need not separately match the design spectrum, but the spectrum for the ensemble of records corresponding to the mean plus one standard deviation (MSD) level of non-exceedance must match it. The response values considered for design in this option must be those associated with the MSD level of non-exceedance. Alternatively, one may initially adjust the intensities of the ground motion records so that the mean of their response spectra matches the design spectrum, and work with the mean values of the resulting responses. In either case, the match should hold over the entire range of frequencies and damping values of interest.

Inasmuch as the power content at different frequencies for the collection of real or modified real time histories can be expected to be representative of those deemed to be appropriate for the site, it is my view that the PSD requirement need not be imposed when multiple histories are used. Expressed differently, implementation of the PSD provision is recommended only for single artificial, real or modified real histories and for multiple artificial histories.

- Only multiple real or modified real ground motion histories are appropriate for inelastic and other nonlinear analyses. In this connection, the word 'appropriate' in the third line from the bottom on p. 3.7.1-4 of the SRP should be changed to 'required'.

In general, I regard the use of multiple ground motion histories to be preferable to the use of a single history, and the use of real histories to be preferable to that of artificial ones. Consequently, I strongly favor options which would encourage the use of multiple real or modified real histories. The proposed relaxation of the PSD requirement, along with the clarification of the requirements on target response spectra which has been presented, should provide a reasonably strong incentive for the more extended use of such input motions, and should dilute the objections to the use of such motions expressed in the public comments.

With regard to the minimum number of ground motion histories that should be employed in the implementation of the multiple history option, I consider the proposed number of five to be quite reasonable. However, if in the opinion of the other members of the Review Panel this number is still likely to discourage the use of this option, I would concur to having the number reduced to four, but would deem a further reduction to be inadvisable. In particular, I consider the multiple history option of the ASCE Standard 4-86 (Ref. 3) to be inappropriate, as it effectively permits the use of as few as two ground motion histories. The recommended minimum number of records should also govern all nonlinear response analyses.

Relationship Between Vertical and Horizontal Ground Motions and the Associated Response Spectra, p. 3.7.1-8

According to Item 1 on p. 2.5.2-12 of the proposed SRP, the design response spectrum for the vertical component of ground shaking should be determined from appropriate ground motion histories in a manner analogous to that used in the development of the corresponding spectrum for horizontal shaking. However, the relationship between vertical and horizontal design response spectra, previously specified on p. 3.7.1-8, has been deleted, and no acceptance criterion is specified in this regard in the revised SRP. I concur with the public comments to the effect that this deletion is undesirable.

In the deleted section, the vertical component of the design acceleration is taken as 2/3 of the horizontal component, and the design spectrum for vertical motion is taken as 2/3 of the spectrum for horizontal motion for all frequencies of interest. I consider this relationship to be generally reasonable, and recommend that its use be permitted for those cases in which the horizontal design spectrum is determined by the procedures specified in Items 2 and 3 on p. 2.5.2-13. However, the appropriateness of this rule must be justified for relatively small epicentral distances. When the design spectrum for horizontal motion is determined by the approach outlined in Item 1, then the spectrum for vertical motion should be determined, as presently proposed, by statistical analysis of relevant ground motion records.

PSD Requirement, pp. 3.7.1-10 to 12

The intent of the proposed PSD requirement is to ensure that the ground motion histories employed in the analysis have adequate power in the frequency ranges of interest. The need for such requirements has clearly been described by Dr. Kennedy (Ref. 4) and need not be reemphasized here. The questions requiring evaluation are whether the recommended provisions represent the most desirable means of attaining the desired objective, and whether they are sufficiently rational and well founded for adoption at this time.

As indicated by Dr. Kennedy, the desired objective could be achieved by imposing stricter requirements on the response spectrum that the ground motion histories must satisfy. In particular, the response spectra of the ground motions for small amounts of damping (of the order of 2 percent of the critical value) may be required to match closely and at closely spaced frequency intervals the corresponding design spectrum. Such a requirement would not be particularly difficult to implement if one were to start with real ground motion histories for which the relevant response spectra are reasonable approximations of the target spectrum. Furthermore, inasmuch as this requirement is consistent with the use of the response spectrum concept as a design basis, it is preferable, in my view, to an approach based on a fundamentally different (the PSD) concept.

There is, of course, nothing wrong with the PSD approach provided it is calibrated to yield practically the same results as those obtained from the response spectrum approach. However, this calibration does not appear to have been implemented to date, and I am not convinced that it can be implemented readily in design applications. Under the circumstances, I question the advisability of adopting the proposed PSD requirement at this time. This view is in agreement with that expressed by Westinghouse on this matter (Ref. 2).

The following facts are noted in further support of this view:

- The PSD function specified on p. 3.7.1-11 is meant to be compatible with response spectra of the type presented in R.G. 1.60, but is clearly not compatible with all other site-specific response spectra permitted in the SRP.

- The discussions of the Review Panel in the December 1988 meeting raised serious doubts about the appropriateness of the coefficient S_0 in the proposed PSD function, as well as about the shape of this function at high frequencies. These uncertainties may be resolved, however, as a result of studies now in progress.

- The operations involved in the determination of the PSD function corresponding to a specified response spectrum are generally delicate, and there are many opportunities for getting the wrong interrelationship between the two functions.

The requirement near the top of p. 3.7.1-11 to the effect that the computed PSD does not fall at any frequency by more than 15 percent below the target function is considered unrealistic by General Electric Company (Ref. 2). While I tend to agree with this assessment, I feel that this issue requires further study. Incidentally, in view of the almost erratic nature of the PSD functions for real earthquakes, it may be preferable to select the coefficient S_0 in the target PSD so that it may be related to the mean of the computed PSD rather than to its lowest 15 percent level.

As previously indicated, I feel that the PSD requirement need not be imposed when the analysis is based on multiple real time histories, even if it is adopted for other cases.

SECTION 3.7.2 SEISMIC SYSTEM ANALYSIS

Soil-Structure Interaction Methodology, pp. 3.7.2-8 to 14

Two different acceptance criteria are specified for soil-structure interaction (SSI) analyses, depending essentially on how the design ground motion is prescribed. Alternate 1 is required for those cases in which the design ground motion is defined either by a broad-banded response spectrum of the type presented in R.G. 1.60, or by some other standardized spectrum determined from estimates of the maximum ground acceleration, velocity and displacement for the site and the application of appropriate amplification factors (as indicated in Item 3, p. 2.5.2-13 of the SRP). Alternate 2 is determined from detailed, site-specific investigations, essentially in the manner specified in Item 1, p. 2.5.2-12.

In Alternate 1, one is required to use both the direct and substructuring methods of analysis, and to envelope the results obtained by the two methods. There is no requirement for detailed parametric or sensitivity studies for this case. By contrast, in Alternate 2, one is allowed to use any state-of-the-art method of analysis and, through detailed parametric studies, is required to assess the sensitivity of the computed responses.

The difference in the requirements for the state-of-the-art analyses referred to in Alternate 2 and those referred to in Alternate 1 is not clear. Neither is the rationale for requiring detailed parametric studies for Alternate 2 but not for Alternate 1. Finally, the requirement for enveloping the results of the direct and substructuring methods of analysis specified for Alternate 1 is not justified in my view.

When properly implemented, both the direct and substructuring methods of analysis, or any other rational approach for that matter, will yield essentially the same results. When improperly implemented, the individual solutions may, of course, be significantly in error, and their envelope may be no better than their component solutions.

The greatest uncertainties in SSI analyses in my view relate to the idealization of the structure-foundation system and its supporting medium, rather than to the method used to analyze the idealized system. In recognition of this fact, the following recommendations are made:

- Delete reference to the two alternates, recognizing that the design ground motion may, as indicated in Section 2.5.2.6, be specified either by a standardized response spectrum or by a site-specific spectrum.
- Permit use of either the direct or substructuring method of analysis, without any enveloping requirement.
- Ensure that the structure-foundation-soil system is properly modeled, and that detailed parametric studies are made to assess the sensitivity of the calculated responses to the numerous uncertainties involved and to bound the solutions. Special reference need be made in this regard to the merits of simple techniques with which the effects of the primary parameters may be evaluated readily and cost-effectively in design.

- Ensure that the analysis of the idealized system is implemented properly by enforcing the relevant provisions of the SRP. In this connection, I do not concur with Sargent & Lundy in their recommendation that item b on p. 3.7.2-11 be deleted. On the contrary, I feel that this item should be presented as item a.

In general, I concur with the position expressed by General Electric (Ref. 2) to the effect that "as long as the major uncertainties associated with SSI effects are properly considered in the analysis, any state-of-the-art approach shall be acceptable". I further concur with the view expressed on page 14 of Ref. 5 that "in view of the large uncertainties, it is not clear that complex, expensive calculations are justified or necessary to develop a soundly engineered design".

Acceptability of Fixed-Base Analysis, p. 3.7.2-10

The SSI effects depend on the relative stiffnesses of the structure and the supporting medium involved rather than on the absolute stiffness of the latter. Accordingly, I believe that the acceptability of the fixed-base analysis should not be expressed solely in terms of the shear wave velocity of the supporting medium, as recommended by Sargent & Lundy (Ref. 2), although their recommendation is likely to yield satisfactory results for many practical cases. However, I do agree with the view that, if reference is made in the SRP to rock and rock-like materials, these terms must be defined.

It is my recommendation that the last paragraph in Item ii on p. 3.7.2-10 be modified as follows:

> "For structures supported on rock or rock-like materials, a fixed-base assumption may be acceptable. Such materials are defined by a shear wave velocity of 3,500 ft/sec or greater at a shear strain of 10^{-3} percent or smaller [when considering preloaded soil conditions due to the structure (?)]. A comparison of the fundamental natural frequencies of the fixed-base and interacting structures can be used to justify the fixed-base assumption."

It might also be desirable to specify the maximum change in frequencies that would be acceptable in this option. A reduction limited to 5 percent of the fixed-base natural frequency value appears to be reasonable.

In the December 1988 meeting of the Review Panel, Dr. Kennedy suggested that the fixed-base analysis be considered to be acceptable when the shear wave velocity of the supporting medium is 3,500 ft/sec and the fundamental fixed-base natural frequency of the system is 10 cps or less. This provision would be equally satisfactory in my view, but I wish to stress that there is no special difficulty in evaluating the fundamental natural frequency of an interacting system when its corresponding fixed-base frequency is known (see, for example, Ref. 6).

Limits for Soil Parameters, p. 3.7.2-12

I believe that the best-estimate values for the shear modulus of the soil should be those corresponding to the strain levels associated with the design earthquake. These strains may be determined from analyses of the seismic wave propagation under free-field conditions, or by some other appropriately substantiated approach. However, the best-estimate values should probably be no less than a prescribed percentage, say 40 percent, of those corresponding to strain values of the order of 10^{-3} percent or less. The specified variations in soil properties should be measured with respect to the best-estimate values.

With regard to the maximum acceptable value of soil material damping, I believe that the limit of 5 percent of critical specified in the SRP is too low, and recommend that it be increased to 15 percent of the critical value. It should be recalled that this percentage is only one-half as large as the value of the $\tan\delta$ factor frequently used in SSI studies.

The recommendations of this section are consistent with those presented on p. 15 of Ref. 5.

Variation of Ground Motion with Depth, p. 3.7.2-14

Because of the multitude of uncertainties involved in the evaluation of the variation of the ground motion with depth, I believe that there should be a limit on the magnitude of the maximum reduction that may be permitted due to embedment. I do not subscribe to the view that such a limit is unnecessary in view of the requirement of varying the soil properties over specified ranges. The latter requirement does not provide for the uncertainties relating to the nature and composition of the seismic waves and their modes of propagation, or the manner in which the nonlinear action of the soil is approximated.

The value of the maximum reduction from the surface motion that may be permitted has been a subject of considerable controversy over the years (see, for example, p. 20 of Ref. 5), and no unanimity of opinion is expected among the membership of the Review Panel. The proposed reduction of 40 percent, which is the same as that permitted in the ASCE Standard (Ref. 3) is too high in my view, and should preferably be limited to a value of no more than 25 or 30 percent.

Such a reduction should refer to the horizontal component of foundation input motion (i.e., the motion that the massless foundation would experience at the level of embedment compared to that at the surface), and should be permitted only when account is taken of the associated rocking and torsional modes of vibration. If the rotational components of motion are ignored, no reduction should be permitted in the horizontal component.

Damping and Modal Combination Requirements, pp. 3.7.1-12 and 16

I concur with the views expressed by Dr. Kennedy on these issues (see Sections 7 and 8 of Ref. 4).

Appendix A, p. 3.7.2-24

1. The notation in this Appendix is highly confusing, and I feel that it should be revised. Considering that the quantities F_i and K_i are dimensionless and do not represent forces or stiffnesses, I recommend that they be replaced by d_i and e_i. I further suggest that the symbols m and M be changed to n and N, respectively, to avoid possible confusion with the mass of the system, and that the participation factor for the nth mode be denoted by c_n. With these revisions, the three equations of the section become:

$$d_i = \sum_{n=1}^{N} c_n \phi_{n,i}$$

in which n = the order of the mode under consideration,

$$e_i = d_i - \delta_{ij}$$

and

$$P_i = ZPA \times M_i \times e_i$$

2. The following expression should be given for the participation factor:

$$c_n = \frac{\{\phi_n\}\{1\}}{\{\phi_n\}^T[m]\{\phi_n\}}$$

in which $\{\phi_n\}$ = the nth natural mode of the system. It may be recalled that these factors refer to displacements and do not involve the circular natural frequencies of the system as multipliers.

Greater Use of Professional Society Consensus Standards

While I strongly concur with Dr. J. Stevenson's recommendation (Ref. 2) of making reference to relevant standards of professional societies and other organizations, I believe that such reference should be limited only to those sections of the standards with which NRC finds itself in agreement, and there should be no impression created of a blanket approval for these documents. I would also be concerned about creating the impression that the proposed SRP is not reasonably up-to-date. In this regard, I question the advisability of incorporating Dr. Stevenson's Insert A in its proposed form.

SECTION 3.7.3 SEISMIC SUBSYSTEM ANALYSIS

Analysis of Above Ground Tanks, pp. 3.7.3-2 and 7 to 9

The following revisions are recommended:

1. On p. 3.7.3-2, change the sentence under Item 12 to read:

"For Category ..... methods which consider the effects of hydrodynamic forces, tank flexibility, soil-structure and other pertinent factors are reviewed."

Basically, I suggest referring to the SSI effects at the end, because they are generally the least important of the factors enumerated and because there is no guidance given in the SRP for their consideration.

2. On p. 3.7.3-7 change the last three sentences of Item 14 to read:

   "For ~~the-ease-of~~ flat bottomed ..... acceleration. Recent studies (Refs.) have shown ..... contained fluid is such that the spectral acceleration may be significantly greater ....."

3. On p. 3.7.3-8, change the last sentence in the first paragraph to

   "The SSI effect may also be very important for ....."

   It may be recalled that the SSI effects are more likely to reduce rather than increase the response.

4. On p. 3.7.3-8 Item b, change the first two sentences to the following:

   "The fundamental natural frequency for the horizontal impulsive mode of vibration of the tank-fluid system must be evaluated giving due consideration to the flexibility of the supporting medium. It is unacceptable to assume a rigid tank unless the assumption can be justified. The horizontal impulsive-mode spectral acceleration S_{a1} is then determined using this frequency and the appropriate damping for the tank-liquid system. Alternatively, the maximum spectral acceleration corresponding to the relevant damping may be used."

Note that no reference is made in this proposal to uplifting. While it is true that uplifting will tend to increase the effective period of the system, this change represents only one aspect of such action, and the magnitude of the change cannot adequately be quantified at this stage. Should it be deemed advisable to refer to uplifting, the first sentence of the proposed section in Item 4 above may be modified to conclude as follows:

" ..... giving due consideration to the flexibility of the supporting medium and to any uplifting tendencies for the tank."

5. Revise Item c on p. 3.7.3-8 to permit consideration of the additional system damping associated with soil-structure interaction, subject, of course, to properly substantiated analyses.

6. At the top of p. 3.7.3-9, delete the last sentence at the end of the first paragraph.

7. Revise Item i on p. 3.7.3-9 to read:

   "The tank foundation ..... seismic forces imposed on it. The forces include ..... as well as the axial tank shell forces resulting from M_0 (caution: not M_b).

8. While Ref. 6 on p. 3.7.3-12 might be retained for its historical interest, Ref. 5 on p. 3.7.3-11 should be replaced by the following more recent and more readily accessible references:

   - A. S. Veletsos and J. Y. Yang, "Earthquake Response of Liquid Storage Tanks," Advances in Civil Engineering Through Engineering Mechanics, Proceedings of the Engineering Mechanics Division Specialty Conference, ASCE, Raleigh, North Carolina, 1977, pp. 1-24

   - M. A. Haroun and G. W. Housner, "Seismic Design of Liquid Storage Tanks," Journal of the Technical Councils, ASCE, Vol. 107, No. TC1, 1981, pp. 191-207

   - A. S. Veletsos, "Seismic Response and Design of Liquid Storage Tanks," Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, Technical Council on Lifeline Earthquake Engineering, ASCE, 1984, pp. 255-370 and 443-461

   Consideration may also be given to referring to the following recent contribution on SSI effects:

   - A. S. Veletsos and Y. Tang, "Soil-Structure Interaction Effects for Laterally Excited Liquid-Storage Tanks," to appear as an EPRI Technical Report, Palo Alto, California, 1989.

No reference is made in the SRP to the effects of the vertical component of ground shaking. This omission should be rectified by the addition of the following statements:

> "The maximum hoop forces in the tank wall must be evaluated with due regard for the contribution of the vertical component of ground shaking. The beneficial effects of soil-structure interaction may be considered in this evaluation."

Following is a list of references on these topics:

- M. A. Haroun and M. A. Tayel, "Axisymmetrical Vibrations of Tanks--Numerical," Journal of Engineering Mechanics Division, ASCE, Vol. 111, No. 3, 1985, pp. 329-345.
- A. S. Veletsos and Y. Tang, "Dynamics of Vertically Excited Liquid Storage Tanks," Journal of Structural Engineering, ASCE, Vol. 112, No. 6, 1986, pp. 1228-1246.
- A. S. Veletsos and Y. Tang, "Interaction Effects in Vertically Excited Steel Tanks," Dynamic Response of Structures, G. C. Hart and R. B. Nelson, Editors, ASCE, 1986, pp. 636-643.

REFERENCES

1. Proposed Revision 2 to Standard Review Plan, Sections 2.5.2, 3.7.1-3.7.3, NUREG-0800, U. S. Nuclear Regulatory Commission, May 1988.

2. Public Comments on Proposed Revision to Sections 2.5.2, 3.7.1-3.7.3 of Standard Review Plan, July 1988.

3. "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE Standard 4-86, September 1986.

4. Kennedy, R. P., "Comments on Proposed Revisions to Standard Review Plan Seismic Provisions," Prepared for Brookhaven National Laboratory, Preliminary Draft, December 1988.

5. Coats, D. W., "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," NUREG/CR-1161, U. S. Nuclear Regulatory Commission, December 1979, Reprinted April 1988.

6. Veletsos, A. S. "Dynamics of Structure-Foundation Systems," <u>Structural and Geotechnical Mechanics</u>, W. J. Hall, editor, Prentice-Hall, Inc., Englewood Cliffs, N. J., 1977, pp. 333-361.

7. Shaukat, S. K., Chokshi, N. C. and Anderson, N. R., "Regulatory Analysis for USI A-40, Seismic Design Criteria, Draft Report for Comment," NUREG-1233, U. S. Nuclear Regulatory Commission, April 1988.

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February 20, 1989

Dr. A. J. Phillippacopoulos
Brookhaven National Laboratory
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Building 129
Upton, Long Island, New York 11973

Dear Mike:

This concerns the originals of my report to you on the USI A-40 Project. Please replace the cover sheet and pages 7, 9 and 10 of the originals which accompanied my letter to you of January 30 with the corresponding pages enclosed herewith. After reviewing the material that you sent me recently, I have decided to make no other changes at this time.

Sincerely,

A. S. Veletsos

ASV:rm

Enclosures

APPENDIX E

COMMENTS ON PROPOSED REVISIONS TO SEISMIC SPECIFICATIONS OF THE US NRC STANDARD REVIEW PLAN

by

C. J. Costantino

prepared for
Brookhaven National Laboratory

January, 1989

1. INTRODUCTION

Recently, the U.S. Nuclear Regulatory Commission (NRC) issued a proposed revision (Revision 2) to the Standard Review Plan (NUREG - 0800) for public comments. These revisions are associated with Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3, which present requirements for the seismic design of nuclear power plants. Comments to these proposed revisions were received from six organizations active in the nuclear industry. In August 1988, a Consulting Panel was formed under the direction of Brookhaven National Laboratory to assist the NRC in resolving the issues brought up by these public comments. As a member of this panel of consultants, I have prepared this report which describes my evaluation of these comments as well as a summary of my position on many of the issues associated with the proposed revisions to the SRP.

The comments that follow can be organized into three primary areas of activity typically associated with the seismic response analyses performed by the industry, namely:

(a) definition of the seismic input motions used in the seismic response analyses of nuclear facilities;

(b) requirements for seismic response analyses to be performed which suitably incorporate soil/structure interaction effects;

(c) details of the structural response analyses performed to assess both primary structural and subsystem dynamic response.

A description of my comments on the above items are presented in the following paragraphs.

2. PSD REQUIREMENTS FOR SEISMIC INPUT MOTIONS

The proposed Revision 2 to Section 3.7.1 of the SRP has added a requirement to judge acceptability of artificial accelerograms to be used in seismic response and SSI analyses. This new criterion requires that the Power Spectral Density (PSD) of the acceleration time history satisfy certain target PSD criteria. Prior to this revision, the requirements on input accelerograms concerned enveloping from above the broad-banded Regulatory Guide 1.60 (R. G. 1.60) ground response spectra. Some arguments in support of the newly added PSD requirements make use of extreme examples of relatively simple input motions which formally envelop the R.G. 1.60 criteria but which may yield deficient responses of subsystems at frequencies of interest in reactor systems.

However, there are two points to be made regarding this argument. Firstly, the extreme examples make use of sinusoidal input motions which do not look like typical accellerograms and therefore would not be accepted in the course of conventional reviews associated with licensing applications. Secondly, at low equipment damping ratios (2% or less), there is no significant difference between the spectrum approach and the PSD criteria (once the definition of the PSD is completely specified). Both are expressions of the Fourier components of the input motions and both strive to enforce adequate representation of the input motion over the entire frequency band of interest. For all practical purposes, they lead to the same conclusions as far as safety of nuclear structures is concerned.

Therefore, I recommend that a PSD criteria <u>not</u> be required in the revised SRP, provided that the Applicant satisfies two conditions, namely:

1. that the design time history satisfies the enveloping criteria for response spectra associated with equipment damping of 2% or less, whether the response spectra used in the analyses are of the broad-banded generic type (such as those of R.G. 1.60) or site specific;
2. that the enveloping criteria be defined as follows:
   a) no more than five points of the calculated spectrum fall below, and no more than 10% below the target spectrum,

(b) the calculated spectrum does not exceed the target spectrum by more than 50% at any frequency,

(c) the calculated spectrum lies at or above the target spectrum at all calculated structural frequencies of interest, and

(d) the calculated spectrum satisfies the specific frequency requirements of the current SRP.

If these requirements are included in the SRP, the need for an added PSD requirement is, in my opinion, not required to demonstrate adequacy of any artificial time history to be used in a seismic response calculation. The structural frequencies of interest mentioned above are to include all frequencies of both the primary and secondary components of the system, and include the effects of SSI on these frequencies.

If, however, the analyst chooses to select a target design response spectrum at higher levels of damping (greater than 2%) from which artificial time histories are to be generated, then a corresponding target PSD criteria should be required to show that the input accelerogram contains adequate power at all frequencies of interest. For the broad-banded spectra specified by R.G. 1.60, I recommend that the procedures which have been developed by M. Shinozuka and R. Kennedy (Ref 1) as part of this Panel's activity be used as a specification of the target PSD, which is suitably compatible with the target design response spectrum. To eliminate any ambiguity in the calculations, the specific definitions of the PSD, its method of calculation and the generation of the corresponding time history should be specified in the SRP.

I do not agree with the suggestion that a Cumulative Power Spectral Density function be used in place of the convential PSD. Since the Cumulative PSD is the integral of the PSD, gaps in power at specific frequencies tend to be masked and seem to me to violate the original intent of the PSD criteria which has been added to the Revision 2 of the SRP. In addition, computation of cumulative PSD's from actual digitized records should be held suspect at the higher frequencies of interest (above 15 hz) since the digitization process itself may have eliminated adequate measures of the power at these frequencies (Refs. 3 and 4).

Guidelines for developing corresponding target PSD requirements for other types of design ground response spectra to be used in the seismic evaluations, either broad-banded or site specific, should be described in the SRP. It is important that sufficient effort be undertaken to develop PSD target functions compatible with the target response spectra to allow for a meaningful comparison to both criteria. Suitable smoothing processes as used in Ref. 2 should be included in the descriptions.

For any case where both target PSD and ground response spectra criteria are specified to generate a design input motion, I recommend that the following procedure be employed to judge the adequacy of the generated time history. First, the computed ground response spectra should satisfy the four specific criteria listed above for the definition of enveloping criteria. Secondly, the computed PSD of the artificial time history should on the average envelop the target PSD over the entire frequency range of interest from 0.4 hz to 33 hz and should not be less than 85% of the target at all the structural frequencies of interest (as previously defined). In applying this last criterion, the comparison should be made using average values computed over a frequency band of ± 15% at each structural frequency.

3. DURATION OF ARTIFICIAL TIME HISTORY

I agree with the comments presented at the various panel sessions that a specific recommendation should be made in the SRP concerning ground motion duration requirements. For linear structural response analyses, the total duration of the accelerogram should be long enough such that adequate representation of the Fourier components (or PSD) at low frequency be included in the time history. To adequately match spectra at 0.4 hz requires total time duration of the ground motion of between 15 and 25 seconds. The suggestion made that a corresponding duration criteria be specified in terms of strong motion duration, based on computation of cumulative energy in the pulse as a function of time, is equivalent. However the duration is specified, it should be adequately tied to the definition of the PSD computation which is dependent on the definition of duration.

The upper bound on potential duration is more questionable. For nonlinear analyses, which may be associated with liquefaction and/or yielding structural response, it seems to me that more care should be taken in defining adequate duration. Firstly, duration should be incorporated in the seismicity study as described in SRP Section 2.5.2 from which the anticipated acceleration levels and earthquake magnitudes are determined. In the calculation of the nonlinear response, a primary topic of interest should be the sensitivity of the specific response to the (strong motion) duration. Specification of exceedingly long pulse durations can lead to overly conservative results. However, if the characteristics of the nonlinear response changes significantly for total durations slightly longer than say 25 seconds, engineering judgement must be incorporated to protect the system from such occurrences. Although I agree that the maximum total duration (rise, stationary, and decay portions) of 25 seconds is reasonable, I recommend that the revised SRP should make provision for such evaluations on a case by case basis.

4. VERTICAL SPECIFICATION OF GROUND MOTION

It is my opinion that the SRP should be clear on the specification of compatible vertical time histories which should be used in conjunction with horizontal motion definitions, whether using R.G. 1.60 criteria as well as site specific horizontal motions. In Ref. 5, it is recommended that a simple scaling of the horizontal spectra (by a factor of 2/3) across the entire frequency band be allowed for the definition of the vertical spectra. This procedure has the obvious advantage of simply scaling the horizontal time history to obtain the compatible vertical history. Such a recommendation may be appropriate for sites located well away from the fault. However, whenever the primary causative fault lies within 10 to 15 km of the site, such a simple scaling would not be appropriate, especially for higher frequencies. For site independent analyses as defined in R.G. 1.60, amplification functions for horizontal and vertical design spectra are not the same at all frequencies.

I recommend that the revised SRP contain a clear specification for the definition of vertical motions for all cases, whether they be for site independent R.G. 1.60 or for site specific evaluations. This definition should be specified in the seismicity studies associated with Section 2.5.2. For most cases, this will lead to a separate development of vertical time histories which must be made in conjunction with the development of horizontal motions. In addition, potential estimates of variability of time phasing between the arrival of vertical and horizontal strong motions should be incorporated in the description of acceptable analyses. For evaluation of linear responses, this phasing is probably not too significant. However, for nonlinear effects at the higher acceleration levels, the phasing could have significant influence on the magnitudes of computed response.

5. NUMBER OF INPUT MOTIONS

If the specification of the input motions discussed above are satisfied, that is, the pulse is chosen to closely match both the target response spectrum and the target PSD, as described above, then the requirement to use multiple time histories in the structural response analyses is not necessary. The primary purpose of the use of multiple time histories in response studies is to ensure that all frequencies of interest are adequately excited. If any one record was deficient at any one frequency, the possibility was that the other records used would not have gaps at the same frequency. With the use of the new criteria for specification of the input motion, the potential for such gaps in energy content is no longer of concern for practical applications.

If more emphasis is placed on closely matching the target spectra when developing input criteria motions, the need for multiple histories reduces. The only variability that would be incorporated in the response calculations with multiple time histories, all of which satisfy the new criteria, would be in the definition of the phasing of the Fourier components of the records. If it is shown that over the frequency range of from 0.4 to 33 hz the phase angles of the components are uniformly distributed over the interval 0 to 2Π, it is my opinion that the potential for clustering of the response is minimal and the need for multiple records is eliminated. In developing time histories which satisfy the new criteria, initial records obtained from actual seismic records can be used to "seed" the computation. The artificial records so developed would then satisfy the above requirement.

I therefore recommend that the SRP include the following options in the seismic response evaluations:

a. If the analyst chooses to use multiple time histories, the envelope spectra produced from all the time histories should satisfy the target response spectra enveloping criteria, and the average of the PSD's of the individual records should also satisfy the target PSD criteria described above. I agree with the previous recommendation that a minimum of five such records be considered.

b. If the analyst chooses to use a single time history to perform his seismic evaluation, then the response spectrum and PSD calculated from this single record should satisfy the criteria described above.

6. <u>SOIL-STRUCTURE INTERACTION</u>

Various modifications have been suggested in the public comments for the revised SRP which are the result of the advances that have been made in recent years in SSI analysis, both as to computational ability as well as to our understanding of the basic phenomena. However, significant uncertainty in specific response of both the soil and structure will always exist so that we must temper our understanding with realistic judgements which, in turn, will lead to "suitable" safety in the design. The following subsections summarize my comments in these various areas associated with the SSI analysis.

A. Alternate Methods of Analysis

In the Summer of 1986, at the workshop held on SSI in Washington, a relatively broad concensus of the computational community arrived at a definition of two separate alternatives for the analyses that may be performed to determine seismic response, one associated with a non-site-specific study using the broad-banded R.G. 1.60 (or equivalent) spectra definition, and the second associated with detailed site-specific evaluations of site seismicity. The basic intent of the approach was to allow the analyst the choice of (a) using broad-banded criteria, or (b) expending more time and effort to reduce the degree of uncertainty in input specification. In this second alternative, the gain achieved is the potential for a more narrow-banded spectra to define input motions.

However, the current proposed revision to the SRP associates this option in alternatives with the specific SSI analysis used in the response calculation. In my opinion, the alternative input option should be placed in Section 2.5.2, and be associated with the description of the applicable input spectra and/or motion histories to be used in the calculations. Section 3.7.2 is intended to describe acceptable methods of SSI analyses, which is a specific technical discussion uncoupled from the specification of input motions.

B. <u>Soil - Structure Interaction Analyses</u>

The primary emphasis of the SRP should be to ensure that proper methods of SSI analysis be utilized which adequately account for the various phenomena involved, such as, radiation and hysteretic damping effects, frequency dependent impedance effects, depth of burial consequences, etc. Various methods of analysis, whether called lumped parameter or half space or finite boundary methods or the three-step approach or substructuring, etc., all can be acceptable provided they are properly applied. In the past, this was not always the case, which in turn led to the conservative enveloping criteria now in effect. All the methods of analysis require detailed evaluation of range of acceptability, all are relatively complex to apply, and all can lead to correct results. However, if not properly applied or evaluated, they can lead to grossly erroneous results. I agree with the comments made by A. Veletsos time and again that more emphasis should be placed on simplified studies to allow for prediction of the range of potential influences of various aspects of the phenomena, as well as describing the bounds on the results that will be anticipated from the complex analyses.

If a proper SSI analysis is performed, suitably accounting for the effects important in a particular problem, no specific concern should be raised as to specification of the criteria motion. In general, this motion is specified at the ground surface (or at some rock outcrop, or rock interface). However described, the analysis performed should be compatible with the specification, and all phenomena associated with the interaction process accounted for and accepted. There is no need to limit any reductions obtained for the process, except as the need requires to account for those aspects of the problem not known or treated adequately.

Thus, if a complete SSI analysis is performed, properly accountingn for all effects due to kinematic and inertial interaction for an embedded structure, with the criteria ground motion specified at the ground surface or hypothetical outcrop, there is in my opinion no need to limit the degree of reduction in the foundation level inputs. This assumes, however, that suitable variability in soil properties, wave specification, etc, is considered. If, however, the SSI analysis is deficient, as say by first performing a vertical motion variation calculation (a la SHAKE) and using this reduced motion as input to the foundation level, then I would favor a limit to the allowable reduction since the complete SSI effect is not properly included in the analysis. I would then suggest that this reduction be limited to 40% of the criteria input spectrum.

C. Compatible Soil Properties For SSI Analyses

A variety of issues can be discussed under this general topic heading. In the SRP, it is not clear how the definition of the "best estimate" soil properties should be incorporated in the analyses. It is my opinion that the pseudo linear approach, assuming upward propagating shear waves, should be used to characterize both the shear modulus and damping variation of the soil column compatible with available experimental soil data. The degradation of soil modulus and increase of soil damping with strain should include both the results from site test data as well as the mass of data accumulated over the years. The "best estimate" SSI analyses should then be performed with these strain compatible soil properties, adequately accounting for the effects of soil layering, depth of burial, etc. Liquefaction, uplift and potential sidewall separation are obviously evaluated from other detailed nonlinear studies.

To account for variability in soil properties in the analyses, I would recommend that the range of properties used in the SSI study be varied from 1/2 to 2 times these "best estimate" values, unless the analyst can show that a reduced degree of variability is appropriate. It has been my experience over the years in testing of soils that such a range of variability is not uncommon in foundation studies. The results at Lotung, Taiwan, even though sampling and testing was carefully controlled, in my opinion demonstrate the validity of this argument.

For all analyses performed for the upper and lower bound soil variation cases, the shear moduli and hysteretic damping ratio used in the SSI analyses should both be compatible with the peak strains calculated from the free-field analysis for the given seismic input accelerogram. This, in turn, can be expected to lead to calculations with high shear modulus and low damping ratio and vice versa. Specifically, I recommend that the upper bound, best-estimate and lower bound cases be defined as follows. The low strain shear modulus (G_{max}), for each soil, should be determined for the best-estimate case based on the results of the field geophysical testing program. The upper bound shear modulus at low strain can then be defined as twice this best-estimate value while the lower bound shear modulus can be defined as one-half this value, provided that this range of variability suitably encompasses the scatter typically found in the field program. Then, average shear modulus degradation (G/G_{max} vs peak shear strain) and hysteretic damping ratio (D vs peak shear strain) curves, as defined in Ref. 5, can be determined from the laboratory testing program, together with typical data available for similar soils. These curves can then be used in the iterative pseudo-linear analyses to determine shear moduli and hysteretic damping ratios compatible with the peak shear strains computed in the free-field for the input seismic criteria motions for all soil layers for each of the three cases of interest. These properties can then be used directly in the SSI computational model.

I would recommend that the final shear moduli results be limited by the following criteria. First, the lower bound shear moduli should not be less than the moduli required for an acceptable foundation design, that is, lead to static settlements much greater than considered acceptable for normal foundation design. Secondly, the upper bound shear moduli should not be less than the best estimate shear moduli defined at low strain (G_{max} defined at 10^{-4} percent peak shear strain) for all soils.

Finally, the limit stated in Section 3.7.2 that hysteretic soil damping should not exceed 5% appears to be too conservative. I would recommend that this value be set at 15%, as suggested in the public comments.

7. REFERENCES

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4. D.W. Coates, "Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria", NUREG/CR-1161, Lawrence Livermore Laboratory, May 1980
5. ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures" Sept 1986.

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February 23, 1989

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Re: Report on Proposed Revisions to the US NRC Standard Review Plan

Dear Mike:

Please find enclosed my final report on the subject modifications to the Standard Review Plan. If you have any questions, please do not hesitate to contact me.

Sincerely yours

Carl

Carl J. Costantino

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13. ABSTRACT (200 words or less)

In June 1988 the Nuclear Regulatory Commission (NRC) issued for public comment the proposed Revision 2 of the Standard Review Plan (SRP) Sections 2.5.2, 3.7.1, 3.7.2 and 3.7.3. Comments were received from six organizations. Brookhaven National Laboratory (BNL) was requested by NRC to provide expert consultation in the seismic and soil-structure interaction areas for the review and resolution of these comments. For this purpose, a panel of consultants was established to assist BNL with the review and evaluation of the public comments. This review was carried out during the period of October 1988 through January 1989. Many of the suggestions given in the public comments were found to be significant and a number of modifications to appropriate SRP sections are recommended. Other public comments were found to have no impact on the proposed Revision 2 of the SRP. Major changes are recommended to the SRP sections dealing with (a) Power Spectral Density (PSD) and ground motion requirements and (b) soil-structure interaction requirements. This report contains specific recommendations to NRC for resolution of the public comments made on the proposed Revision 2 of the SRP.

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