Revision 1

Technical Rationale for Enhancements to Seismic and Structural Review Guidance

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Introduction

The U.S. Nuclear Regulatory Commission (NRC) technical report NUREG–0800, "Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," Sections 3.7 and 3.8 provide guidance for the seismic analysis and structural design of the containment and other seismic Category I structures and foundations. The NRC issued a major revision to these SRP sections in 2007 (prior to Revision 4) for use in the review of design certification (DC) applications and combined license (COL) applications under Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants." Because of the differences in the licensing process between 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," and 10 CFR Part 52, the seismic analysis and structural design under 10 CFR Part 52 requires different approaches from those used previously. These differences have resulted in some technical issues and challenges to both applicants and reviewers which the 2007 revision to SRP was intended to address.

Nuclear power plant (NPP) structures are designed to withstand both internally and externally initiated hazards. For internal events—such as internal floods and loss-of-coolant accidents (LOCAs)—the design approach for 10 CFR Part 50 and Part 52 facilities is essentially the same. However, for externally initiated events—such as wind, snow, and earthquake—different approaches to structural analyses and designs may be taken for 10 CFR Part 52 applications as opposed to 10 CFR Part 50 facilities, because of the distinct differences in the licensing process. Externally initiated events such as earthquakes are inherently site-dependent, which was easily addressed in single-site applications under 10 CFR Part 50. However, the site-dependent aspects of the external events pose challenges to standard plant design under the 10 CFR Part 52 process.

As the NRC staff has made significant progress in the review of DC and COL applications under 10 CFR Part 52, the staff also identified a number of technical issues related to seismic analysis and structural design in the 2007 SRP revision which could be enhanced to provide additional guidance. To a large extent, the issues identified are related to the technical aspects associated with standard (generic) plant designs that can be built based on assumed site parameters representative of various site conditions in the United States. These issues have often led to the issuance of more requests for additional information (RAIs) and have lengthened the review process. The lessons learned from application reviews can be used to identify the technical areas in which staff guidance could be improved; in turn, the improved guidance will enhance efficiency and effectiveness in facilitating the review for future applications.

To this end, Revision 4 to SRP Sections 3.7 and 3.8 has incorporated the lessons learned to date and provided additional technical guidance for future application reviews. This report is intended to document technical rationale and basis for technical enhancements included in Revision 4 to SRP, Sections 3.7 and 3.8 and provides the supporting materials which the reviewers may find useful in aiding their understanding of the technical issues and the acceptance criteria which address review challenges.

Objective

The objective of this report is to provide documentation which (1) identifies the key design challenges that arose from the seismic and civil structural reviews of new reactor applications under the Part 52 process, (2) describes the underlying technical issues, (3) describes

enhanced criteria included in Revision 4 to SRP Sections 3.7 and 3.8 related to these issues, and (4) provides a technical rationale for the enhancements to SRP acceptance criteria. More specifically, the report describes twelve key issues related to seismic analysis and structural design identified during the staff's review of licensing applications; explains why the SRP should be enhanced in these areas; presents technical enhancements incorporated in Revision 4 to SRP, Sections 3.7 and 3.8; and finally provides the technical basis and/or rationale for the enhancements. The enhanced criteria are also intended to improve clarity of technical issues and ensure a more uniform review process that will benefit the nuclear industry and better assist the staff in future technical reviews of applications. With these enhancements, the staff expects that both applicants and reviewers will be better able to ascertain technical issues in preparation and review of future applications related to seismic analysis and structural design, which will lead to a more effective and efficient licensing process.

Scope

The scope of this effort is to document the technical rationale and basis, considered in the technical enhancements incorporated in Revision 4 to SRP Sections 3.7 and 3.8, and to address the significant technical issues associated with the technical challenges identified during the licensing review process of past DC and COL applications. Twelve technical issues were identified as listed below.

1. Seismic uplift in soil-structure interaction (SSI) analysis
2. Seismic stability evaluation for design of structures
3. Interaction of non-seismic Category I structures with seismic Category I structures, systems and components (SSCs)
4. Seismic soil pressure on embedded walls
5. Ground motion incoherency effect on seismic soil-structure interaction (SSI)
6. Cracking effect on seismic analysis of concrete structures
7. Differential settlement and construction sequence considerations in foundation design
8. Artificial time history development
9. Standard plant site parameters and consideration for seismic design basis
10. Issues with SASSI subtraction method
11. Guidance on spent fuel pool racks
12. Guidance on minimum power spectral density for NUREG/CR-6728 based design spectra or other spectra

It is important to note that these technical issues above have been extensively discussed between the staff and the applicants during new reactor application reviews. The technical rationales described in this report are intended to be consistent with the resolution of the technical issues reached and to reflect the lessons learned during application reviews.

For each technical issue listed above, the subsequent sections of this report describe: issue; reason why the SRP was enhanced; enhancements to SRP acceptance criteria included in SRP Revision 4, Sections 3.7 and 3.8; and technical rationale for the enhancements.

Technical Issue No. 1

Seismic Uplift in Soil-Structure Interaction (SSI) Analysis

1. Description of Issue

The design of seismic Category I structures requires consideration of the potential uplift of the foundation from the supporting soil media. Limiting the foundation uplift is necessary to ensure the validity of the SSI analysis which is typically performed using linear techniques. Consideration of foundation uplift is also important in the analysis to estimate the overturning moments induced by the seismic input and the soil bearing pressures at the foundation toe.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

Prior to Revision 4, SRP Section 3.7.2 II.4 indicated that sensitivity studies are needed to identify the potential for separation and sliding of soil from sidewalls, among other issues, and to assist in judging the adequacy of the final results of the SSI analyses. SRP Section 3.8.5 II.4.D provided guidance on how to define the dead load for uplift evaluations, including the treatment of the stored water in pools inside the structures. Other SRP Section 3.8.5 subsections provided guidance on seismic stability evaluations, foundation design, and maximum toe bearing pressures.

2.2 Why Technical Criteria Were Enhanced

Prior to Revision 4, SRP did not provide specific guidance on permissible levels of foundation uplift to be considered in the SSI analysis and the foundation design.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.2

The applicable portion of SRP Section 3.7.2 II.4 and the technical enhancements (highlighted in italics) are shown below.

4. Soil-Structure Interaction A complete SSI analysis should properly account for all effects *(i.e.,* due to kinematic and inertial interaction*, as applicable)* for surface or embedded structures. Any analysis method based on either a direct approach or a substructure approach can be used provided the following conditions are met:

   A. The structure, foundation, and soil are properly modeled to ensure that the results of analyses properly capture spatial variation of ground motion, three dimensional effects of radiation damping and soil layering, as well as nonlinear effects from site response analyses*. The analyses should incorporate the appropriate soil profiles determined in SRP Sections 2.5.1 through 2.5.3, as well as SRP Section 3.7.1.*

   B. *For structures with either surface or shallow embedded foundations, the seismic input motions to the SSI analyses can be typically placed at the free ground surface or at the foundation level using the guidance in SRP Section 3.7.1, as supplemented by DC/COL- ISG-017. However, the input*

motions defined at locations other than the foundation level may not be appropriate for embedded structures. In these situations, the seismic input should only be specified at the foundation level as the FIRS.~~The design earthquake ground motions used as input to the SSI analyses should be consistent with the design response spectra as defined in SRP Section 3.7.1.~~

It is noted that there is enough confidence in the current methods used to perform the SSI analysis to capture the basic phenomenon and provide adequate design information; however, the confidence in the ability to implement these methodologies is uncertain. Therefore, in order to ensure proper implementation, the following considerations should be addressed in performing SSI analysis:

A. Perform sensitivity studies to identify important parameters (e.g., potential *foundation uplift,* separation and sliding of soil from sidewalls, non-symmetry of embedment, location of boundaries) and to assist in judging the adequacy of the final results. These sensitivity studies can be performed by the use of well-founded and properly substantiated simple models to give better insight;

B. Through the use of some appropriate benchmark problems, the user should demonstrate its capability to properly implement any SSI methodologies; and

C. Perform enough parametric studies with the proper variation of parameters (e.g., soil properties) to address the uncertainties (as applicable to the given site) discussed in *S*~~s~~ubsection I.4 of this SRP section.

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. If the SSI analysis is not required, the input motion at the base of the structures will be the design motion reviewed in SRP Section 3.7.1.

If the SSI analysis using linear techniques results in tension stresses between the foundation basemat and the underlying soil, for load combinations that include seismic, dead and other applicable gravity loads (per SRP Section 3.8.5 II.4.D uplift evaluations), the effect of the foundation uplift should be evaluated. The staff reviews the calculation of the ground contact ratio to ensure the linear SSI analysis remains valid. The ground contact ratio is defined as the minimum ratio of the area of the foundation in contact with the soil to the total area of the foundation, computed in each time step throughout the SSI analysis.

Uplift for non-symmetric structures may be more affected by the phasing between the three directions of input motions. Therefore, technical justification should be provided if the effect of different phasing of the input motions is not considered in the calculation of the foundation uplift. If the non-symmetric

conditions need to be addressed, then the effect of in-phase and out-of phase input motions can be considered in the SSI analyses by using plus and minus 1.0 times the magnitude of the input motions.

Linear SSI analysis methods are acceptable if the ground contact ratio is equal to or greater than 80 percent. The ground contact ratio can be calculated from the linear SSI analysis using the minimum basemat area that remains in compression with the soil. If the ratio is less than 80 percent, then the effect of the nonlinearity due to the foundation uplift should be evaluated. If the uplift effect on structural responses (e.g., in-structure response spectra, member forces, soil bearing pressure, and building displacements, etc.) is found to be significant (e.g., an increase in response of more than 10 percent), then the uplift effect should be accounted for in the seismic design, which is reviewed on a case-by-case basis.

4. Technical Basis and/or Rationale

The seismic analysis used in the design of seismic Category I structures requires consideration of the potential uplift of the foundation from the supporting soil media. Limiting the foundation uplift is necessary to ensure the validity of the SSI analysis that is based on linear methodologies. Consideration of foundation uplift is also important in order to appropriately estimate the overturning moments induced by the seismic input and the soil bearing pressures at the foundation toe.

SRP Section 3.7.2 II.4 indicates that sensitivity studies are needed to identify the potential for the separation and sliding of soil from sidewalls, among other issues, and to assist in judging the adequacy of the final results of the SSI analyses. SRP Section 3.8.5 II.4.D provides guidance on how to define the dead load for uplift evaluations, including the treatment of the stored water in pools inside the structures. Other SRP Section 3.8.5 subsections provide guidance on seismic stability evaluations, foundation design, and maximum toe bearing pressures. However, the SRP does not provide specific guidance on permissible levels of foundation uplift to be considered in the SSI analysis and the foundation design.

The enhancements incorporated in SRP Revision 4 are based on Japanese design criteria [1, 2, 3], which utilize the ground contact ratio concept. The ground contact ratio is defined as the ratio of the minimum area of the foundation in contact with the soil to the total area of the foundation. The seismic response computed over the entire duration of the seismic ground motion needs to be considered to determine the minimum value of this ratio.

In the context of linear SSI analysis, the calculation of the ground contact ratio should be based on total response from input in the three directions of seismic ground motion acting simultaneously, as well as dead and other applicable gravity loads per SRP Section 3.8.5 II.4.D acting on the structure. Uplift for non-symmetric structures may be more affected by the phasing between the three directions of input motions. If the non-symmetric configuration is determined to be potentially significant, then the analysis should consider the plus and minus values of the three directions of input motions. This can be achieved by multiplying the magnitude of the time histories by the factors 1.0 and -1.0. Then all permutations of these results should be considered in determining the uplift response. In cases where the vertical seismic motion is determined to be relatively small, then only the two horizontal motions would need to consider the plus and minus variation.

SRP Revision 4 sets a limit of 80 percent to the ground contact ratio to accept the results from the linear SSI analysis. The corresponding limit in the Japanese criteria is 75 percent (Section 3.5.5.4 in the Japanese design code JAEC 4601-2008 [2]; see also Figure 1 in Nakamura et al. [3]).

A more conservative value is used in SRP Revision 4 because of two important differences between U.S. and Japanese design practice. First, in current U.S. practice, all three components of ground motion are considered to act simultaneously, while in Japanese practice only the vertical plus one horizontal component are considered to act simultaneously (prior to 2006; the vertical seismic input was considered as a static force [3]). Second, the Japanese criteria assume that the numerical results are computed using a lumped-mass SR ("Sway-Rocking") analytical model together with lumped parameter spring and dashpot SSI model to represent interaction with the underlying soil media. This type of SSI analysis is different from what is typically performed in U.S. practice. The approach of considering all three components of ground motion would yield more conservative seismic overturning demands than considering only the vertical plus one horizontal component; however, the effect of this conservatism may only be significant for the case of non-symmetric structures. Conversely, seismic overturning demands computed using SR models, as in the Japanese approach, are recognized as more conservative than corresponding seismic demands computed using the detailed SSI finite element models that are typically used in the United States. Therefore, the proposed limit of 80 percent on the ground contact ratio for validity of linear SSI analysis incorporates a level of conservatism relative to the Japanese limit of 75 percent. It is also important to note that the 80 percent limit is consistent with the findings of previous analytical studies (e.g., Wolf [4] and Miller [5]).

SRP Revision 4 also indicates that ground contact ratio can be calculated directly from the linear SSI analysis using the minimum basemat area that remains in compression with the soil. This implies that, the ground contact ratio calculated from the linear SSI analysis is an approximation of the area that would be in contact with the soil from a nonlinear uplift evaluation. This approximation is considered valid when the contact is not less than about 80 percent.

Revision 4 of SRP Section 3.7.2 addresses the validity of linear SSI analysis if some foundation uplift occurs. If the limit of 80 percent is not met then the nonlinearity due to the foundation uplift should be assessed, and if found important, then it should be accounted for in the seismic design, which is then reviewed on a case-by-case basis. Additional guidance for foundation design is incorporated in Revision 4 to SRP Section 3.8.5.

5. References

[1] Park, Y.J., Hofmayer, C. H., "Technical Guidelines for Aseismic Design of Nuclear Power Plants: Translation of JEAG 4601-1987," U.S. Nuclear Regulatory Commission, Washington, DC, 1994 (in Japanese; English translation available as NUREG/CR-6241).

[2] Japan Electric Association, "Technical Rule for Seismic Design of Nuclear Power Plants JEAC 4601-2008" (in Japanese).

[3] Nakamura, N., Ino, S., Kurimoto, O., and Miake, M., "An Estimation Method for Basemat Uplift Behavior of Nuclear Power Plant Buildings," *Nuclear Engineering and Design*, 237(12-13):1275-1287.

[4] Wolf, J.P., “Soil-Structure Interaction with Separation of Basemat from Soil (Lifting-Off),” *Nuclear Engineering and Design*, 38(2):357-384.

[5] Miller, C.A., “Soil-Structure Interaction Influence of Lift-Off,” Vol. 2, Brookhaven National Laboratory Report BNL-NUREG-51983, Upton, NY, 1986. (Also available as NUREG/CR-4588.)

Technical Issue No. 2

Seismic Stability Evaluation for Design of Structures

1. Description of Issue

To ensure the safety of nuclear power plant (NPP) structures, the seismic design of these structures also includes an evaluation of their seismic stability against sliding and overturning. In recent licensing reviews of standard designs, which utilize bounding calculations for seismic loads and soil parameters, difficulties arose in satisfying the acceptance criteria in SRP Section 3.8.5 prior to Revision 4 using the pseudo-static approach for calculating factors of safety (FOS) against sliding and overturning. More detailed seismic analysis methods aimed at reducing some of the conservatisms inherent in the calculations can provide more realistic results. Unlike the pseudo-static approach, the more detailed seismic analysis methods utilize the time history approach and incorporate sliding and liftoff capabilities in the model. To provide guidance to the performance and the review of such analyses, enhancement to criteria incorporated in Revision 4 to SRP Section 3.8.5 is discussed below.

2. Why SRP Section 3.8.5 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The guidance related to the seismic stability evaluation of seismic Category I structures is presented in SRP Section 3.8.5 II "Acceptance Criteria."

2.1.1 The acceptance criteria in SRP Section 3.8.5 II.3 "Loads and Load Combinations" define the load combinations used to check against sliding and overturning as follows:

A. D + H + E
B. D + H + W
C. D + H + E'
D. D + H + W_t
E. D + F'

D, E, W, E', and W_t are defined in Subsection II.3 of SRP Section 3.8.4; H is the lateral earth pressure; and F' is the buoyant force of the design-basis flood. Justification was provided for including live loads or portions thereof in these combinations.

2.1.2 The acceptance criteria in SRP Section 3.8.5 II.4, "Design and Analysis Procedures" stated that: "the methods for determining the overturning moment attributable to an earthquake should be in accordance with the approach described in SRP Section 3.7.2."

The design and analysis procedures were enhanced to consider (1) the performance of the sliding analysis method and how the analysis adequately accounts for potential foundation uplift effects, if appropriate; (2) the method to calculate the factor of safety against sliding; and (3) if sliding resistance is the sum of shear friction along the basemat and passive pressures induced by embedment effects, how these effects are considered in an analysis based on a consistent lateral displacement criterion.

2.1.3 The acceptance criteria in SRP Section 3.8.5 II.5 “Structural Acceptance Criteria” provided the factors of safety for the five load combinations defined in SRP Section 3.8.5 II.3.

2.2 Why Technical Criteria Were Enhanced

Applicants calculate the lateral forces (shear and moment) on building foundations due to seismic and other loads; this is referred to as the demand. Applicants also calculate the lateral resisting forces on the foundations due to the structure bearing on the soil beneath the basemat and along the vertical foundation walls, as well as from the friction between the structure and soil. This is called the resisting capacity. The ratio of the lateral resisting capacity to the demand forces should be shown to be equal to or greater than the factor of safety of 1.1 in the case of load combination C, which includes the safe shutdown earthquake (SSE). The conventional practice has been to consider the seismic lateral demand as a constant statically applied force. The seismic demand is obtained from a seismic equivalent static analysis, response spectra analysis, or time history analysis. On the capacity side, the resisting forces are usually based on calculations of the frictional resistance and lateral soil resisting forces up to the full passive pressure capacity if needed.

When designs are subject to high seismic loads and bounding soil properties, as is the case in standard designs or for plants located in high seismicity regions, achieving the needed factors of safety for sliding and overturning may be more difficult to demonstrate using the static approach. Several recent design certification (DC) applicants have resorted to more complex analytical methods to reduce the conservatisms inherent in the static approach. These methods rely on time history analyses using three directions of statistically-independent seismic loadings applied simultaneously. This approach eliminates the static analysis assumption that the maximum vertical and maximum horizontal demand forces occur at the same time. The oscillatory nature of the response in a seismic time history analysis may demonstrate that the specified factors of safety are maintained at each instant in time. If the linear time history analysis indicates that some sliding and uplift may occur, then a nonlinear time history analysis can be performed to include these effects.

To provide guidance about the staff’s expectations when the time history evaluations discussed above are performed, the guidance in SRP Section 3.8.5 was enhanced to address several issues:

(1) since there is no single value to be used for the seismic demand, how the FOS should be calculated in the evaluation
(2) for nonlinear analysis, how many time histories should be considered and how should the results from each of the time histories be evaluated
(3) the adequacy of the mathematical model
(4) enhancement of the criteria for selection of the appropriate friction values
(5) acceptance criteria if minimal sliding displacements do occur

<u>3. Enhancements Incorporated in Revision 4 to SRP Section 3.8.5</u>

3.1 The criteria for design and analysis procedures in SRP Section 3.8.5 II.4, “Design and Analysis Procedures” were enhanced to provide additional guidance to the issues associated with seismic stability evaluation. The applicable portion of this SRP section and the enhancements to the acceptance criteria included in SRP Revision 4, Section 3.8.5 (highlighted in italics) are given below.

4. Design and Analysis Procedures. The design and analysis procedures used for seismic Category I …

F. The structural audit is conducted in accordance with SRP Section 3.8.4, Appendix B.

G. Methods for determining the *sliding forces and* overturning moment attributable to an earthquake should be in accordance with the methods described in SRP Section 3.7.2.

H. Computer programs are acceptable if the validation provided is found to be in accordance with the procedures delineated in Subsection II.4.E of SRP Section 3.8.1.

In addition to the above, the design and analysis procedures for the following details are reviewed on a case-by-case basis:

A. *Appropriateness of the ~~M~~method* for determination of the bending moments and shear forces in the foundation mat for seismic loads~~?~~.

B. *Adequacy~~Performance~~* of the sliding analysis method and *~~how~~* the analysis *results ~~adequately~~ to* account~~s~~ for potential foundation mat liftoff effects.~~, if appropriate?~~ The *~~method to~~ staff should also review the* calculation~~calculate~~ of the factor of safety against sliding. If sliding resistance is the sum of shear friction along the basemat *and contribution of soil lateral pressure up to the full* passive *~~pressures~~* pressure capacity induced by embedment effects, *the adequacy of the analysis to consider ~~how~~* these effects *~~are considered in an analysis based on~~ is addressed using* a consistent lateral displacement criterion.~~?~~ *This involves the use of static versus dynamic coefficient of friction consistent with the use of partial versus full passive pressure. The reviewer should also consider whether the selection of the coefficient of friction used in the sliding stability analysis considers the various sliding interfaces (e.g., soil shear failure, concrete to soil, waterproofing to soil, concrete basemat to concrete mudmat).*

If the stability evaluation is performed based on a pseudo-static approach, using the maximum seismic demand loads (e.g., maximum forces in the two horizontal directions and one vertical direction), then the factors of safety for sliding and overturning can be determined by the ratio of capacity to demand loads.

However, if a linear time history analysis approach is utilized, then the factor of safety can be calculated at each time step throughout the time history. The minimum value of the factors of safety calculated in this manner should be compared against the acceptance criteria for that load combination. For the pseudo-static and time history analysis methods, all three directional demand forces should be considered to act simultaneously. Therefore, the resultant seismic forces (horizontal

resultant force for sliding from the two horizontal forces and similarly the resultant overturning moment for overturning stability) should be considered. In the case of the sliding evaluation, if instead of using the resultant horizontal force with the vertical force, each pair of horizontal force and vertical force is evaluated separately, then the frictional resistance in the horizontal directions should be apportioned considering the existence of the two horizontal forces.

If the stability evaluation is performed using a nonlinear time history analysis that includes foundation sliding and uplift, the analysis should consider the following criteria:

i. The development of the set of time histories should follow the guidance described in SRP Section 3.7.1. This includes identification of the number of input time histories needed to perform the nonlinear time history analyses and the development of each of the individual time histories. In this case, the guidance in SRP 3.7.1 II.B, Option 2, for multiple sets of time histories is applicable.

ii. To demonstrate an adequate factor of safety, the seismic input time histories should be increased by a factor equal to the factor of safety for the applicable load combination (e.g., increase the seismic input time history amplitudes by a factor of 1.1 for load combination C). No or minimal sliding, and no overturning should be demonstrated for each of the time history analyses.

iii. The mathematical model should include the effects of sliding and uplift between the foundation and the soil media using appropriate finite elements that can simulate sliding once the frictional limit is reached and can simulate contact surfaces that can transmit compression but not tension.

iv. The sliding and overturning stability evaluation should consider the various significant parameters that were evaluated in the design basis seismic SSI analysis (e.g., range of soil profiles, concrete stiffness variation).

v. If the input motion applied at the foundation of a structural model without the soil is developed from the response of the linear SSI analysis, justification is needed to demonstrate that any minimal sliding or uplift would not affect the assumed seismic input motion taken from the SSI analysis that does not consider any sliding and uplift. Alternatively, the structural model could be coupled with the soil model and a nonlinear SSI analysis performed.

vi. The mathematical model should adequately represent the dynamic characteristics of the structure and capture the vibration modes important for the sliding and overturning stability analysis.

vii. If some minimal sliding does occur, the justification for incurring a small magnitude of sliding needs to be provided. In this case, the magnitude of sliding should be based on the envelope of the values obtained from the individual time history analyses. In addition, the magnitude of sliding/overturning plus the SSI building displacements need to be evaluated for adequate seismic gaps between structures, and the design adequacy of commodities attached to the structures (e.g., piping and conduit between adjacent structures above grade; buried piping, conduit, and tunnels) need to be evaluated.

4. Technical Basis and/or Rationale

The conservatism in the SRP pseudo-static approach for demonstrating the factors of safety for sliding and overturning stability has led to difficulties when such analyses for generic plant designs are performed where higher seismic loads are defined and a range of soil profiles are typically considered. Therefore, it is reasonable to utilize more realistic analytical methods that reduce some of the conservatisms inherent in the static type stability evaluation methods.

4.1 Enhancement to SRP Section 3.8.5 II.4.B, First Paragraph:

Based on past licensing applications, it became apparent that in many cases, the coefficient of friction selected for the sliding stability analysis did not represent the governing coefficient considering the various potential sliding interfaces. This is important because if the lowest coefficient of friction is not used for this evaluation, then the sliding resistance will be over predicted. Also, it is important to note in the revision that if a meaningful portion of the passive soil resistance is relied upon to provide lateral soil resistance, then the static coefficient of friction would not be appropriate; instead, a dynamic coefficient of friction that is less than the static coefficient of friction would be applicable.

4.2 Enhancement to SRP Section 3.8.5 II.4.B, Second Paragraph:

One approach to reducing conservatism in the static type stability evaluation is to use the linear time history analysis method, which includes the phasing of the building responses in the three perpendicular directions (two horizontal and one vertical). The pseudo-static approach inherent in the SRP Section 3.8.5 stability evaluation criteria did not account for the phasing effect of the three ground motions and assumes that the maximum vertical seismic force acts upward which reduces the effect of dead weight, and thereby, reduces the lateral frictional resistance between the structure and soil. This minimum horizontal frictional resistance is assumed to occur at the same time as the maximum horizontal seismic force is applied. The use of a linear time history analysis reduces this conservatism by checking for sliding or overturning at each time step throughout the time history analysis. Using this approach, the peak upward seismic load would not be expected to occur at the same time as the peak horizontal seismic load. In this case, the factor of safety can be calculated at each time step throughout the time history. The minimum value of the factors of safety calculated in this manner should be compared to the acceptance criterion for that load combination.

In the pseudo-static and time history analysis methods, all three directional demand forces should be considered to act simultaneously. Therefore, the resultant seismic forces for sliding from the two horizontal forces and the resultant overturning moment for overturning stability should be considered. This is important because for sliding, for example, if one horizontal and vertical set of forces are evaluated against the full sliding resisting force, and then the other perpendicular horizontal direction with vertical set of forces are evaluated against the full sliding resisting force, this approach would overestimate the sliding resistance available for both evaluations. This approach is unconservative because it utilizes the full sliding resistance in both directions; therefore, the sliding evaluation should be performed using the resultant horizontal force with the vertical force. Alternatively, each pair of horizontal and vertical set of forces can be evaluated separately, but in this approach, the frictional resistance in each horizontal direction should be apportioned considering the existence of the two horizontal forces.

4.3 Enhancement to SRP Section 3.8.5 II.4.B, Item i:

If the stability analysis is to be performed using a nonlinear time history analysis, it is important to identify the number of time histories to be considered and how to develop the multiple time histories. Since Revision 4 to SRP Section 3.7.1 provides enhanced guidance, a reference to SRP Section 3.7.1 is included to address this item.

4.4 Enhancement to SRP Section 3.8.5 II.4.B, Item ii:

In a nonlinear time history analysis, to account for the required factor of safety (FOS), the time histories for all input motions should be increased by the FOS (e.g., multiplied by 1.1 for load combination C). The FOS is considered satisfied if the analysis shows that no or minimal sliding and no overturning occur at each time step throughout the time history.

4.5 Enhancement to SRP Section 3.8.5 II.4.B, Item iii:

To consider the effects of sliding and overturning, the mathematical structural model needs to include finite elements that can capture the sliding effects and separation of the basemat and soil. The finite elements for sliding should have the capability to maintain connectivity between the basemat and soil until the contact force times the coefficient of friction is reached. Then the finite element needs to permit sliding to occur while maintaining the friction force. The finite element for uplift should have the capability to transfer load in compression between the basemat and soil. Also, it should release connectivity and contact when the two surfaces separate from each other.

4.6 Enhancement to SRP Section 3.8.5 II.4.B, Item iv:

The sliding and overturning stability evaluation should consider the various significant parameters that were used in the design basis seismic soil structure interaction analysis. The parameters to consider include range of soil profiles and concrete stiffness variation because of cracked and uncracked conditions. In a nonlinear analysis it is usually very difficult to judge which value within a given parameter would govern. Therefore, it is prudent to consider the variation in parameters when performing the nonlinear stability evaluations.

4.7 Enhancement to SRP Section 3.8.5 II.4.B, Item v:

If a mathematical model is developed for the structure with sliding and uplift capabilities, and the input motion applied at the foundation of this model is developed from the response of the linear SSI analysis, then the sliding of the structure may affect the input motion that was taken from the SSI analysis that does not have any sliding or uplift. Therefore, justification should be provided to demonstrate that the limited sliding and uplift that occur do not affect the seismic input motion. If the stability analysis is performed using a structural model coupled with the soil model, then this concern is eliminated because the coupled model incorporates the potential effects of sliding and uplift in the overall analysis.

4.8 Enhancement to SRP Section 3.8.5 II.4.B, Item vi:

To obtain realistic results in the nonlinear seismic stability analysis, it is important that the mathematical model adequately represents the dynamic characteristics of the structure and captures the vibration modes important for the sliding and overturning stability analysis.

4.9 Enhancement to SRP Section 3.8.5 II.4.B, Item vii:

If the results from all of the nonlinear time history analyses, with the seismic input motions increased by a factor corresponding to the factor of safety, show that there is no sliding or overturning of the structure, then stability of the plant structure has been demonstrated. If some limited sliding and/or uplift do occur, then the applicant should provide the acceptance criteria for the small magnitude of sliding and/or uplift. It is difficult to develop quantitative acceptance criteria because they are problem dependent. They depend on a number of factors such as whether a coupled mathematical model of the structure and soil are used or only the structure model is used and whether the effects of embedment are included.

Because of the nonlinearity in this type of analysis, it is possible that the sliding and/or uplift magnitudes from each of the time history analyses may differ from one another more than expected. Also, the value for the coefficient of friction has some variability. Therefore, it is more appropriate to use the maximum value (i.e., envelope) of the results from the individual time history analyses.

If sliding and/or uplift do occur, an additional evaluation is needed to determine whether there are adequate seismic gaps between structures so that no impacts would occur. In addition, the design adequacy of commodities attached to the structures (e.g., piping and conduit between adjacent structures above grade; buried piping, conduit, and tunnels) needs to be demonstrated. These evaluations should consider the effects of the calculated sliding and/or uplift magnitudes with an appropriate design factor and then added to the seismic soil structure interaction building deformations.

Technical Issue No. 3

Interaction of Non-Seismic Category I Structures with Seismic Category I Structures, Systems and Components (SSCs)

1. Description of Issue

Non-seismic Category I structures are typically designed to a less restrictive criteria than seismic Category I structures. However, if the non-seismic Category I structures are located in close proximity to seismic Category I SSCs, then the failure of the non-seismic Category I structures could adversely impact the safety function of seismic Category I SSCs, which is often referred to as seismic II/I interaction.

To ensure that seismic Category I SSCs will not be damaged or rendered nonfunctional by failure of a non-seismic Category I structure due to the safe shutdown earthquake (SSE), Standard Review Plan (SRP) Revision 3 Section 3.7.2.8 provided three approaches (A, B and C) to ensure that unacceptable interactions do not occur.

One of the methods – (approach C) stated: "The non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions, such that the margin of safety is equivalent to that of Category I structures."

The criterion in approach C "…, such that the margin of safety is equivalent to that of Category I structures." was recognized to be overly conservative for the design of non-seismic Category I structures, since it apparently invokes the same design criteria as is applicable to seismic Category I structures. While adherence to seismic Category I design criteria is acceptable, the criteria associated with approach C was enhanced to reduce the inherent conservatism.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

Prior to Revision 4, SRP Section 3.7.2.8, approach C, stated, "The non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions, such that the margin of safety is equivalent to that of Category I structures."

2.2 Why Technical Criteria Were Enhanced

The seismic II/I criteria are intended to ensure that seismic interaction between the non-seismic Category I structures and seismic Category I SSCs would not adversely affect the safety function of the adjacent seismic Category I SSCs. The criterion provided in approach C, as currently defined, was overly restrictive because it imposed the margin of safety for the non-seismic Category I structures to be equivalent to that of seismic Category I structures. Therefore, this criterion was overly conservative and was enhanced in SRP Revision 4.

3. Enhancements Incorporated in Revision 4 to SRP Sections 3.7.2 and 3.7.3

3.1 Revision 4 to SRP Section 3.7.2

The applicable portion of SRP Section 3.7.2 II.8 and the enhancements to the acceptance criteria in SRP Revision 4, Section 3.7.2 II.8 (highlighted in italics) are shown below.

8. Interaction of Non-*Seismic* Category I Structures with *Seismic* Category I SSCs. All non-*seismic* Category I structures should be assessed to determine whether their failure under SSE conditions could impair the integrity of seismic Category I SSCs, or result in incapacitating injury to control room occupants. Each non-*seismic* Category I structure should meet at least one of the following criteria:

   A. The collapse of the non-*seismic* Category I structure will not cause the non-*seismic* Category I structure to strike a *seismic* Category I SSC.

   B. The collapse of the non-*seismic* Category I structure will not impair the integrity of seismic Category I SSCs, nor result in incapacitating injury to control room occupants.

   C. The non-*seismic* Category I structure will be analyzed and designed to prevent its failure under SSE conditions~~*, such that the margin of safety is equivalent to that of Category I structures*~~.

   The disposition of each non-*seismic* Category I structure should be formally documented.

   For criterion ~~*(b)*~~*B*, it is necessary to provide the technical basis for the determination that collapse of the non-*seismic* Category I structure is acceptable. This should include a description of any additional loads imposed on the *seismic* Category I SSCs and the method used to conclude that these loads are not damaging. Also, any protective shields installed to prevent direct impact on *seismic* Category I SSCs should be described.

   For criterion C, it is necessary to demonstrate that there is no physical interaction between the non-seismic Category I structure and all adjacent seismic Category I SSCs. The maximum permissible displacement of the non-seismic Category I structure in any direction is determined by subtracting the maximum calculated displacement of each adjacent seismic Category I SSC in the direction of the non-seismic Category I structure from the minimum as-designed gap, considering construction tolerances. The criterion of no physical interaction should be demonstrated for all elevations of the non-seismic Category I structure, taking into consideration the potential for sliding and rocking of the non-seismic Category I structure.

   A conservative way to address criterion C is to apply a linear elastic analysis to the non-seismic Category I structure, similar to seismic Category I structures. However, depending on the magnitude of the gap between the non-seismic Category I structure and the adjacent seismic Category I SSCs, a limited inelastic

response may be permissible for the non-seismic Category I structure, provided the structural integrity can be demonstrated, to ensure no physical interaction between the non-seismic Category I structure and all adjacent seismic Category I SSCs. In the assessment, the effect of structure-soil-structure interaction should be accounted for, if significant.

If an inelastic response method is utilized to address criterion C, the demand may be determined using several methods that consider the nonlinear behavior of the structure (e.g., nonlinear static analysis or nonlinear dynamic analysis). If a nonlinear time history analysis is utilized, then the guidance in SRP Section 3.7.1 II.1.B, Option 2, related to the use of multiple time histories for nonlinear analysis, should be followed. In this case, the acceptance criteria with respect to permissible displacements should be satisfied for each individual time history analysis. The use of inelastic response methods and acceptance criteria will be reviewed by the staff on a case-by-case basis.

To ensure an adequate evaluation of seismic Category I SSCs in a DC application, it is necessary to determine that they are not vulnerable to collapse or interaction with adjacent non-seismic Category I structures. Consequently, DC applicants should provide sufficient analysis and design information concerning interaction of the non-seismic Category I Structures with seismic Category I SSCs for staff review. In lieu of this, the DC application may describe the analysis and design approach that will be implemented by a COL applicant, and also identify a COL information item requiring that an evaluation be performed and documented to address the interaction of non-seismic Category I Structures with seismic Category I SSCs. In addition, associated ITAAC (e.g., check of as-built vs. as-designed gaps; reconciliation of as-built vs. as-designed geometry and materials for the non-seismic Category I structures) should be identified.

3.2 Revision 4 to SRP Section 3.7.3

The applicable portion of SRP Section 3.7.3 II.8 and the enhancements to the acceptance criteria included in SRP Revision 4, Section 3.7.3 II.8 (highlighted in italics) are shown below.

8. Interaction of *Non-Seismic Category I*~~Other~~ *Sub*~~S~~ystems *w*~~W~~ith Seismic Category I *SSCs*~~Systems~~. To be acceptable, each non-seismic Category I *sub*system should be designed to be isolated from any seismic Category I *SSC*~~system~~ by either a constraint or barrier, or should be remotely located with regard to the seismic Category I *SSC*~~system~~. If ~~*it*~~ *this* is not feasible or practical*,* ~~*to isolate the seismic Category I system,*~~ *then* adjacent non-seismic Category I *sub*systems should be analyzed according to the same seismic criteria as applicable to the seismic Category I *SSC*~~system~~. For non-seismic Category I *sub*systems attached to seismic Category I *SSCs*~~systems~~, the dynamic effects of the non-seismic Category I *sub*systems should be simulated in the modeling of the seismic Category I *SSC*~~system~~. The attached non-seismic Category I *subs*ystems, up to the first anchor beyond the interface, should also be designed in such a manner that during an earthquake of SSE intensity it will not cause a failure of the seismic Category I *SSC*~~system~~.

~~*The acceptance criteria provided in SRP Section 3.7.2, subsection II.8, are applicable to all seismic Category I SSCs at the system and subsystem level.*~~

4. Technical Basis and/or Rationale

The objective of Approach C is to demonstrate that failure under SSE conditions will be prevented. What constitutes "failure" depends on the proximity of the non-seismic Category I subsystem to seismic Category I SSCs.

For example, if a non-seismic Category I subsystem is in proximity to a seismic Category I structure, the absolute sum of the seismic displacements of the subsystem and structure needs to be less than the as-designed gap between them over the entire potential interface region to satisfy criterion C.

If the subsystem and SSC are very close, it may be necessary to apply seismic Category I design criteria to the non-seismic Category I subsystem to satisfy criterion C.

If there is an appreciable gap between the subsystem and the SSC, then a relaxation of seismic Category I design criteria for the subsystem may be appropriate, provided it can be demonstrated that (1) the non-seismic Category I subsystem is in a stable limit state (no gross failure), and (2) the as-designed gap (considering construction tolerances) between the subsystem and SSCs is large enough to accommodate the absolute sum of the seismic displacements of the subsystem and the SSCs.

The American Society of Civil Engineers (ASCE) 43-05 [1] presents a graded approach to design and analysis of structures for loading combinations that include seismic loads. In accordance with ASCE 43-05, nuclear seismic Category I structures require the most stringent design criteria; namely, a linear elastic limit state. This is consistent with NRC guidance for design and analysis of seismic Category I structures. ASCE 43-05 also addresses design and analysis of structures of less critical functions, allowing response beyond the elastic limit state, to a safe and predictable inelastic limit state. Such an approach is potentially applicable to satisfying approach C, in which there is sufficient gap to accommodate increased displacement of the non-seismic Category I subsystem.

5. References

[1] ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities." American Society of Civil Engineers, 2005.

Technical Issue No. 4

Seismic Soil Pressure on Embedded Walls

1. Description of Issue

The computation of seismically induced lateral soil pressures on embedded walls is complex and depends on many factors. In classical soil mechanics, lateral soil pressures on walls can vary between two limit stress states, labeled as active and passive, depending on the deformation imposed on the soil by relative motion of an adjacent wall. If one considers a simple case of a rigid wall supporting adjacent soil, the lateral stresses are defined as an at-rest stress state that is caused primarily by the dead weight of the adjacent soil, although it could be affected by other factors such as compaction effects, local groundwater or saturation, etc. The active state occurs when the wall then moves away from the soil, and the passive state occurs when the wall moves into the soil. Lateral pressures in the active and passive states are bounded, respectively, by the minimum active and the maximum passive limit states.

For granular soils under static conditions, lateral soil pressures based on the minimum active and maximum passive limit states can be readily computed using the Rankine/Coulomb formulation, which has been used in civil engineering design for more than 200 years. For seismic conditions and more general soil materials, however, the situation is significantly more complicated. A single methodology does not appear to be applicable to all possible scenarios encountered in nuclear power plant (NPP) design practice, although the minimum active and maximum passive limit states, with appropriate modifications, still can be used to establish bounds for seismically induced soil pressures.

The guidance in the SRP Revision 4 incorporated additional enhancements to clarify the applicable methods acceptable to the staff for computing both static and additional seismically induced lateral soil pressures required for the design of NPP walls. Guidance was also improved to provide clarity for reviewing the analysis assumptions for these acceptable methods. This is important because soil pressures can vary substantially depending on many factors and the uncertainties need to be adequately addressed in the soil pressure calculations.

2. Why SRP Sections 3.8.1 and 3.8.4 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

Prior to Revision 4, SRP Sections 3.8.1 II.4.E and 3.8.4 II.4.H indicated that consideration of dynamic lateral soil pressures on embedded walls is acceptable if these pressure loads are evaluated for two cases. These are: (1) the total lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with the American Society of Civil Engineers (ASCE) 4-98 Section 3.5.3.2, and (2) the maximum lateral earth pressure that could be developed against the wall equal to the maximum passive earth pressure. It is also indicated that if these methods are shown to be overly conservative, then alternative methods are reviewed on a case-by-case basis.

2.2 Why Technical Criteria Were Enhanced

Prior to SRP Revision 4, the two approaches for computing seismically induced lateral soil pressures that were described as acceptable in SRP Sections 3.8.1 II.4.E and 3.8.4 II.4.H had limitations that should be more clearly articulated.

The limitations of Wood's method described in ASCE 4-98 Section 3.5.3.2 should be identified. For example, in situations where the embedded walls tend to rotate relative to the free-field, when rocking of the structure is important, then the corresponding kinematic conditions deviate from the assumptions of the method. In such cases, an alternative method should be considered to address these limitations.

It is also necessary to ensure that all or part of the passive pressure is incorporated in the design regardless of whether the maximum passive state condition has been reached in the soil.

Finally, the SRP was enhanced to provide guidance on the review of the analysis assumptions for the methods that the staff considers acceptable.

3. Enhancements Incorporated in Revision 4 to SRP Sections 3.8.1 and 3.8.4

3.1 The applicable portions of SRP Sections 3.8.1 II.4.E and 3.8.4 II.4.H, as well as the enhancements to the acceptance criteria included in SRP Revision 4, Sections 3.8.1 II.4.E and 3.8.4 II.4.H (highlighted in italics) are shown below.

SRP Section 3.8.1 II.4.E

E. Dynamic Soil Pressure. Consideration of dynamic lateral soil pressures on embedded walls of a concrete containment (if applicable) is acceptable if the lateral earth pressure loads are evaluated for ~~*two cases. These are (1) lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98 Section 3.5.3.2 and (2) lateral earth pressure equal to the passive earth pressure*~~ *the three cases identified in SRP Section 3.8.4 II.4.H*. If the *above* methods *identified in SRP Section 3.8.4 II.4.H* are shown to be overly conservative for the cases considered, then any alternative methods proposed are~~will be~~ reviewed on a case-by-case basis.

SRP Section 3.8.4 II.4.H

H. Consideration of dynamic lateral soil pressures on embedded walls is acceptable if the lateral earth pressure loads are evaluated for *the governing of the following* ~~two~~ *three* cases. These are (1) lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2(2);~~,~~ *(2) lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated using an embedded SSI/Finite Element Model (FEM) analysis model;* and ~~(2)~~ *(3)* lateral earth pressure equal to *the fraction of* the passive earth pressure *that is effectively mobilized, which is dependent on the relative magnitude of the wall displacements against the soil that may occur for a given wall configuration. For case (3), the analysis should include, to a minimum, the fraction of the passive*

earth pressure assumed in the stability calculations performed in accordance with SRP Section 3.8.5.

ASCE 4-98 Section 3.5.3.2(2) describes a method based on the well-known elastic solution by Wood (1973). This method assumes linear elastic strains in a homogeneous soil mass, a rigid wall with fixed base supported on stiff soil, and no displacement or sliding of the wall base relative to the underlying soil. Soil dynamics and wave propagation effects in the soil-wall system are also not considered. These assumptions may not be satisfied, for example, in the case of massive structures in deep soil sites where rocking could be important. Nevertheless, for cases where the assumptions of Wood's solution are realistic, the method yields conservative estimates of the dynamic pressures.

To account for a broad range of kinematic conditions, heterogeneity of the soil, as well as soil dynamics and wave propagation effects, a second method should be included based on soil-structure interaction (SSI) analysis of an embedded SSI/FEM model, as described in SRP Section 3.7.2. A limitation of such analysis is that it also assumes linear (or equivalent-linear) elastic strains in the soil. Therefore, a third method based on passive pressure should also be included to account for potential inelastic strains.

The staff reviews the validity of the assumptions that are the basis of each of these three methods and the extent to which they correspond to the actual site conditions. In particular, the staff reviews the SSI/FEM model used in method (2) to ensure it is appropriate to this type of application.

If other effects such as structure-soil-structure interaction are important, these should be included in addition to the pressures computed using the methods described above.

If these methods are shown to be overly conservative for the cases considered, then the staff reviews alternative methods on a case-by-case basis. For earth retaining walls *that are not restrained by a building*, the guidance in ASCE 4-98 Sections 3.5.3.1 through 3.5.3.3 is acceptable.

4. Technical Basis and/or Rationale

The computation of seismically induced lateral soil pressures on embedded walls is complex and dependent on many factors. Under static at-rest conditions, the soil pressures are primarily controlled by soil type, soil compaction (or relative density), and groundwater conditions. For granular materials, typically placed as backfill in the immediate vicinity of the wall, the static at-rest pressures at a given depth are estimated by K_o x σ_v, where σ_v is the vertical intergranular stress due to the weight of the soil above the depth of interest and K_o is called the at-rest coefficient of earth pressure. The parameter K_o is typically estimated to be approximately 0.5 (or more appropriately 1 – sin∅, where ∅ is the effective friction angle of the granular backfill material).

Under seismic conditions, the pressures change from the static case and are primarily controlled by the relative motion developed between the wall or structure and the free-field. If the wall or structure moves away from the soil, these dynamically induced pressures decrease

from the static at-rest pressures. This stress state is termed the active state. The minimum value of this active state is estimated for ordinary conditions as K_a x σ_v, where K_a is called the Rankine active coefficient of earth pressure. The value of K_a is less than K_o and is approximately 0.33. If the wall or structure moves into the surrounding soil, the pressures increase above the static at-rest condition. This increased stress state is termed the passive state. The maximum value of this passive state is estimated for ordinary conditions as K_p x σ_v, where K_p is called the Rankine passive coefficient of earth pressure. For ordinary granular soils, the value of K_p is approximately 3.0 or more.

For typical granular materials, it is expected that the total pressures acting on the wall under seismic conditions will change from the static at-rest case to no less than the Rankine active pressure and to no more than the Rankine passive pressure. Therefore, for ordinary granular materials, the total horizontal pressure coefficients (static plus dynamic) can be expected to vary during the seismic motions from about 0.5 to no less than 0.33 and to no more than 3.0.

For more general soil materials that possess both cohesive and frictional shear strength, the formulation of the maximum and minimum Rankine states is more complex to delineate. Nevertheless, under seismic conditions, the total horizontal pressures are also bounded by the maximum and minimum Rankine states in a similar way as typical granular materials.

It is important to note that the magnitude of soil deformations required to fully develop the maximum and minimum Rankine states could be relatively large. The kinematic configuration of the problem is thus fundamental. This is the reason why the seismic design of embedded or basement walls (so-called “non-yielding” walls or “restrained” walls, that are fixed at the base, at the top, and possibly at other intermediate bracing points) should be clearly differentiated from the seismic design of earth retaining walls (so-called “yielding” or “unrestrained” walls, that are free to displace or rotate at the base).

In the case of unrestrained retaining walls, the standard seismic design approach is the Mononobe-Okabe method [1], which is based on the assumed development of the minimum active state that is modified to include, in a pseudo-static manner, the additional horizontal and vertical seismic inertial loads exerted by the soil. The active state assumption is valid in this case because a standalone retaining wall is free to deform away from the soil but is unlikely to deform into the soil.

In the case of restrained embedded walls, the typical configuration of the problem precludes the development of the minimum active state assumption. In NPP applications, the embedded walls are not stand-alone but are part of a much larger and more massive structure that, under seismic conditions, interacts dynamically with the surrounding soil in a complex oscillatory manner. The maximum passive state condition is a potential upper bound that would correspond to the walls being pushed into the soil by the overall motion of the structure. However, the magnitude of soil deformations and strains computed from typical seismic soil-structure interaction (SSI) analysis indicates that this magnitude is much smaller than what is required to fully develop the maximum passive state. As a result, seismic design practice in the past has been based on methods that assume linear elastic or equivalent-linear elastic soil stresses and strains (e.g., the methods proposed by Wood [2], Veletsos and Younan [3, 4], and Ostadan and White [5, 6]).

It is clear that the true stress and strain state in the soil under seismic conditions is likely to deviate from the inelastic limit states discussed above. Important additional factors to consider are the following:

- Kinematics of the problem. Significant differences in the pressure distribution profile and the stress and strain state in the soil could occur depending on whether the embedded walls are assumed to be rigid or flexible, whether the base of the walls are allowed to rotate or slide relative to the soil, whether the structure is supported on stiff or flexible soil, and whether the overall motion of the structure includes a significant rocking component or not
- Heterogeneity of the soil mass. An additional complication occurs if there is significant difference in stiffness between different backfill and in situ soil layers, and especially if the structure is partially embedded in rock. In the latter case, a large stress discontinuity is expected in the interface region between soil and rock
- In typical NPP configurations, linear or equivalent-linear strains would tend to produce conservative estimates of pressures; however, there may be specific configurations for which the opposite occurs
- Separation of the soil from the wall
- Structure-soil-structure interactions between structures
- Effects of groundwater on static and dynamic soil pressure

Field measurements and experimental investigations confirm the wide variation in soil pressures depending on the different factors identified above (see, e.g., [5, 6, 7] and references therein). Therefore, the included enhancement to the SRP describes three methods to compute seismically induced lateral soil pressures on embedded walls, which should bound the uncertainties in the estimates for most design situations. The governing pressures of the three methods should be considered in the design. The governing pressures should also be determined based on the pressure distribution that generates the maximum member forces used for design of the foundation walls.

The first two methods are based on linear or equivalent-linear elastic assumptions, while the third method is based on the passive pressure and accounts for inelastic strains, albeit in a simplified manner. A clarification is also made relative to the passive pressure; the intent is to ensure that all or part of the passive pressure is incorporated in the design regardless of whether the maximum passive state condition has been reached in the soil (as indicated above, the latter is rarely the case for embedded walls in NPP structures). The displacement-dependent fraction of the passive pressure that is effectively mobilized can be determined from nonlinear finite element model (FEM) computations or from experimental results.

The included enhancement also emphasizes the review of the analysis assumptions. This is important because soil pressures can vary substantially depending on the different factors identified above. Conservative assumptions are thus critical.

The second method has been added to the SRP in light of recent NPP designs, in which seismic soil pressures are computed using an embedded SSI/FEM analysis model. This is a general approach that is appropriate under the linear elastic or equivalent-linear elastic strain assumption and may address some of the issues discussed above. However, a careful review

is still needed to ensure the validity of the modeling for this type of application. For example, the computed pressures may be overestimated in the upper soil layers near the surface. Also, the strain iterated soil profiles utilized in typical SSI analysis may not be consistent with the range of strains expected in the backfill soil adjacent to the walls.

The best estimate, lower bound, and upper bound soil properties should be considered in the second method if the SSI analysis performed in accordance with SRP Section 3.7.2 includes those soil cases.

Hydrodynamic effects on dynamic soil pressures are difficult to estimate accurately but are generally not a concern except for backfill soils with high permeability. In most design situations where saturated conditions exist, it is sufficient to compute the effects of groundwater on dynamic soil pressures on the basis of saturated unit soil weight.

5. References

[1] Kramer, S. L., “Geotechnical Earthquake Engineering.” Prentice-Hall, Englewood Cliffs, 1996.

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[3] Veletsos. A. S. and Younan, A. S., “Dynamic Soil Pressure on Rigid Vertical Walls.” Earthquake Engineering and Structural Dynamics, Vol. 23, pp. 275-301, 1994.

[4] Veletsos. A. S. and Younan, A. S., “Dynamic Modeling and Response of Soil-wall Systems.” ASCE Journal of Geotechnical Earthquake Engineering, Vol. 120, pp. 2155 2179, 1994.

[5] Ostadan, F. and White, W. H., “Lateral Seismic Soil Pressure: An Updated Approach.” 1st US-Japan SSI Workshop, U.S. Geological Survey, Menlo Park, California, September 22-23, 1998.

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[7] Al Atik, L. and Sitar, N., “Experimental and Analytical Study of the Seismic Performance of Retaining Structures.” PEER Report 2008/104, Pacific Earthquake Engineering Research Center, University of California, Berkeley, 2008.

Technical Issue No. 5

Ground Motion Incoherency Effect on Seismic Soil-Structure Interaction (SSI)

1. Description of Issue

Revision 3 of the Standard Review Plan (SRP) Section 3.7.2 (March 2007) recognized that the nuclear industry was considering advanced analytical methods (e.g., the effects of incoherent ground motion) to reduce the potential effects of high frequency ground motion input, and that these methods may be used when a site acceptability determination is performed, as discussed in Subsection II.4 of SRP Section 3.7.1. SRP Section 3.7.2 also noted that if incoherency is used to reduce the high frequency response, the potential effects of increasing other responses (e.g., overturning and torsional responses) need to be considered.

The staff subsequently issued interim staff guidance (DC/COL-ISG-01 [1]) in May 2008, which addresses high-frequency seismic loading, and also describes methods acceptable to the staff for evaluating the effects of ground motion incoherency on the high-frequency seismic response of seismic Category I structures, systems, and components (SSCs).

In past implementation of this guidance, the staff performed independent confirmatory analysis. The staff's review and independent confirmatory analysis highlighted that complex technical issues need to be addressed in the implementation of the staff-accepted methodology (per DC/COL-ISG-01) to incorporate incoherency effects in seismic response analyses of structures.

The enhancement incorporated in SRP Revision 4, Section 3.7.2 includes the guidance of DC/COL-ISG-01, and also considers the lessons learned from past application reviews. Because of the degree of uncertainty associated with analytical predictions of the effects of ground motion incoherency, the enhancement to SRP Section 3.7.2.4 stipulates maximum acceptable response reductions.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The March 2007, SRP Revision 3 criteria provided a brief discussion of incoherency but did not provide sufficient guidance for its review. Subsequent issuance of DC/COL-ISG-01 provided detailed review guidance for incorporating incoherency effects into SSI analyses.

2.2 Why Technical Criteria Were Enhanced

With the issuance of DC/COL-ISG-01 in May 2008, the staff formalized its acceptance of specific methods to account for the effect of incoherent ground motion in reducing high frequency structural response. However, SRP Section 3.7.2.4 did not contain guidance comparable to the criteria in DC/COL-ISG-01 related to considering ground motion incoherency in the SSI analysis.

In view of the uncertainties associated with the incoherency effects on structural responses and the limited experience in its use at NPPs, it is prudent to set appropriate limits on the maximum reductions obtained by considering incoherency effects.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.2.

SRP Revision 4, Section 3.7.2 II.4, under "Specific Guidelines for SSI Analysis," tenth bullet, which is given below, includes the following enhancements (highlighted in italics):

> ~~*There are advanced analytical methods that are being considered by the nuclear industry (e.g., the effects of incoherent ground motion) to reduce the potential effects of high frequency ground motion input. These might be used when a site acceptability determination is performed as discussed in subsection II.4 of SRP Section 3.7.1. If incoherency is used to reduce the high frequency response, the potential effects of increasing other responses (e.g., overturning and torsional responses) shall be considered. When approved for use by the NRC, via issuance of interim staff guidance, it should be noted that the effects of incoherent ground motion may be considered either at the Design Certification stage, or at the site-specific application stage, but not both.*~~
>
> ~~*If any advanced analytical methods are utilized, the technical basis and analysis results are subject to detailed review on a case-by-case basis.*~~
>
> *There are advanced analytical methods currently being applied in the nuclear industry to develop seismic responses to high frequency ground motion inputs, incorporating the effect of ground motion incoherency. These methods might be used when a site acceptability determination is performed, as discussed in Subsection II.4 of SRP Section 3.7.1. The phenomenon of ground motion incoherency in the free field has been investigated and characterized in terms of coherency functions, based on recorded earthquake data collected from dense array field tests. The ground motion incoherency effect on structural response is considered by incorporating coherency functions in analytical methods for SSI analyses. SSI analyses based on analytical methods that consider ground motion incoherency generally reduce structural response in high frequencies, compared to the response based on the traditional assumption of ground motion coherency. If the effect of incoherent ground motion is used to reduce the high frequency response, the potential effects of incoherent ground motion in increasing overturning and torsional responses need to be considered.*
>
> *The U.S. Nuclear Regulatory Commission (NRC) issued DC/COL-ISG-01 on May 19, 2008, describing methods acceptable to the staff for evaluating high frequency ground motion input. It includes guidance for conducting analyses that incorporate incoherent ground motion.*
>
> *Because of the complexity of such analyses, and the lack of both an experience data base and test data, the implementation of the analytical methods described in DC/COL-ISG-01, for considering incoherent ground motion, is subject to staff review on a case-by-case basis. Applicants are expected to present comparisons between calculated*

coherent and incoherent seismic demands. Based on the staff's current experience, the following maximum reductions in the amplitude of spectral accelerations are acceptable for the ISRS:

0 to 10 Hz – 0% reduction
30 Hz and above – 30% reduction
10 to 30 Hz – reduction based on linear variation between 0% at 10 Hz and 30% at 30 Hz

The maximum ISRS reduction limits are applied to the calculated incoherent ISRS results only where the reduction limits are exceeded by the calculated reductions. Where the reduction limits are not exceeded, the calculated incoherent ISRS results are to be used, including where the incoherent results exceed the coherent results. The corresponding adjusted incoherent ISRS results are to be included in the ISRS comparison plots described above.

Larger ISRS reductions than specified above may be acceptable to the staff, if there is sufficient technical information supporting the larger reductions. The staff reviews and accepts the technical justifications for larger reductions on a case-by-case basis.

For structural loads, which are predominantly controlled by seismic input up to 10 Hz, the maximum acceptable reduction, due to the effects of incoherent ground motion, is 10 percent. If the structural loads increase due to the effects of incoherent ground motion, then the higher incoherent structural loads are to be used for structural design.

It is noted that the effects of incoherent ground motion may be considered at the DC application stage in a generic evaluation of high-frequency ground motion input. In such a case, a COL applicant would confirm that the site-specific high-frequency ground motion input and the underlying site profile are encompassed by the generic evaluation. When referencing a certified design, a COL applicant may also conduct site-specific SSI analysis that considers incoherency effects to reduce the high-frequency response. In this case, the site-specific in-structure responses should be enveloped by the responses obtained from the analysis of the CSDRS; further guidance can be found in SRP 3.7.1 II.4.

4. Technical Basis and/or Rationale

Advanced analytical methods are being applied in the nuclear industry, to develop seismic responses to high-frequency ground motion inputs by incorporating the effect of ground motion incoherency. The phenomenon of ground motion incoherency in the free field has been investigated and characterized in terms of coherency functions, based on recorded earthquake data collected from dense array seismic data. The ground motion incoherency effect on structural response is considered by incorporating a coherency function into analytical methods for soil-structure interaction (SSI) analyses. SSI analyses based on analytical methods that consider ground motion incoherency generally reduce structural response in high frequencies, compared to the response based on the traditional assumption of ground motion coherency. However, the effect of incoherent ground motion may cause an increase in motions associated with overturning and torsional structural responses.

Although the development of the coherency function incorporated a degree of conservatism that was intended to compensate to a certain extent for the uncertainty associated with the lack of data for the high-frequency hard rock sites in the eastern United States and the ground motion variability with the depth, it is important to recognize that the reductions in structural response due to ground motion incoherencies are quantified based on analytical models which utilize the coherency function described in the EPRI report [2] and also recognize that their application to licensing activities is rather limited to date. In addition, there are no field tests available to confirm the extent to which the incoherent free-field ground motion reduces the structural response in terms of in-structure response spectra and member forces. Therefore, it is reasonable to place a limit on the reductions due to incoherency at this time. The staff will reconsider the limit when more information and associated data become available to better quantify the in-structure response reductions due to ground motion incoherency effects.

The analytical studies performed by EPRI [2] are the basis of the staff's guidance in DC/COL-ISG-01. A comparison of coherent ISRS results to incoherent ISRS results, based on the NRC-accepted coherency function, is presented in Appendix B of the EPRI report. The structural model used has stick model representations of the AP1000 steel containment vessel, shield building, and containment internal structures, sitting on a large basemat. Review of the comparisons presented in Appendix B indicates that the ISRS are generally similar up to about 10 Hz. Above 10 Hz, there is generally a gradually increasing difference. Above 30 Hz, differences as high as 40-50 percent can be observed. The staff has accepted this level of reductions on a case-by-case review along with independent confirmatory analysis.

Based on the uncertainties associated with the coherency function described above and the limited experience in its implementation to date, the staff has determined that it is reasonable to impose limits on the maximum acceptable reductions between coherent results and incoherent results. These reductions generally are in line with the results reported in Appendix B of the EPRI report. It should be noted that the enhanced criteria allow for larger ISRS reductions than the specified limits if sufficient technical information is provided which supports the larger reductions. The staff reviews and accepts the technical justifications for larger reductions on a case-by-case basis.

5. References

[1] DC/COL-ISG-01, "Interim Staff Guidance on Seismic Issues Associated with High Frequency Ground Motion in Design Certification and Combined License Applications." U. S. Nuclear Regulatory Commission May 19, 2008.

[2] Electric Power Research Institute (EPRI) Report, "Program on Technology Innovation: Validation of CLASSI and SASSI Codes to Treat Seismic Wave Incoherence in Soil-Structure Interaction (SSI) Analysis of Nuclear Power Plant Structures." Palo Alto, CA, November 2007.

Technical Issue No. 6

Cracking Effect on Seismic Analysis of Concrete Structures

1. Description of Issue

When developing mathematical models to perform seismic analysis of concrete structures, the stiffness of the structural elements is affected by the degree of concrete cracking. To ensure that the mathematical models realistically represent the concrete structures, enhancements were incorporated in the Standard Review Plan (SRP) Revision 4 to provide additional criteria on how to consider the effects of concrete cracking on the structural stiffness of members used in the models.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The guidance related to the effects of concrete cracking on structural stiffness is presented in SRP Section 3.7.2 II "Acceptance Criteria" and 3.7.2 III "Review Procedures." Some additional information on concrete cracking is provided in SRP Section 3.8.1 "Concrete Containments" which is also referenced in SRP Sections 3.8.3 through 3.8.5.

2.1.1 Prior to SRP Revision 4, the acceptance criteria in SRP Section 3.7.2 II.9 "Effects of Parameter Variations on Floor Response Spectra" indicated that when developing floor response spectra (which include peak broadening), consideration should be given in the analysis to the effects of expected variations of structural properties, damping values, soil properties, and soil-structure interaction (SSI). The acceptance criteria for the consideration of the effects of parameter variations were presented in Subsection II.5 of SRP Section 3.7.2. The SRP criteria also indicated that for concrete structures, the effect of potential concrete cracking on the structural stiffness was not specifically addressed.

2.1.2 The review procedures in SRP Section 3.7.2 III.9 "Effects of Parameter Variations on Floor Response Spectra" had similar information as SRP Section 3.7.2 II.9 which indicated that among the various structural parameters analyzed, the effect of potential concrete cracking on structural stiffness was not addressed.

2.1.3 The acceptance criteria in SRP Section 3.8.1 II.4 "Design and Analysis Procedures" provide guidance on the treatment of creep, shrinkage, and cracking of concrete in containment design. The criteria describe why cracking can occur, the effect cracking has on stiffness, the potential shift in frequency, and the possible effect cracking can have on the building response/loads used in design. SRP Sections 3.8.3 through 3.8.5 refer to SRP Section 3.8.1 for design and analysis procedures.

2.2 Why Technical Criteria Were Enhanced

The SRP is intended to provide evaluation criteria that would ensure that the seismic analysis and design methods satisfy the U.S. Nuclear Regulatory Commission (NRC) regulations. The performance of seismic analysis requires accurate mathematical modeling of the plant structures in order to determine building responses such as in-structure response

spectra (ISRS), member forces, and displacements. For steel structures, the member properties used for the finite elements in the mathematical model are reasonably well understood and can be accurately modeled. However, for concrete members, the member properties are very much a function of the stress level and degree of concrete cracking. Concrete cracking can reduce the stiffness of concrete members, reduce the frequencies of the structure, increase the damping of the members, and can decrease or increase the demand (e.g., ISRS, member forces, displacements) which are used for the design of the structures, systems and components (SSCs) which are supported in the structure.

2.2.1 The criteria in SRP Section 3.7.2 only identified the need to consider the effects of concrete cracking on the structural stiffness with respect to development of floor response spectra. The criteria did not provide specific guidance on how to accomplish that. In addition, the criteria did not describe how to consider the effect of cracking on the seismic analysis used to determine other seismic responses (e.g., member forces, displacements, soil bearing pressures) needed for design.

2.2.2 The criteria in SRP Section 3.8.1, primarily in SRP Section 3.8.1 II.4 "Design and Analysis Procedures," under Item D "Creep, Shrinkage, and Cracking of Concrete," indicated that cracking of concrete for containments is expected and discuss the effects of concrete cracking on stiffness, frequencies, and member forces. Therefore, the criteria concluded that the effects of concrete cracking need to be considered if significant. The criteria indicated that this can be done by using computer programs that can evaluate cracking directly within the finite element model (FEM) or by determining the response of the containment to variation in the stiffness characteristics of the structure due to cracking. Reference was also made to additional guidance for modeling the stiffness of concrete elements described in the American Society for Civil Engineers (ASCE) Standard 4-98, Sections 3.1.3 and C3.1.3. SRP Section 3.8.3 "Concrete and Steel Internal Structures of Steel or Concrete Containments," SRP Section 3.8.4 "Other seismic Category I Structures," and SRP Section 3.8.5 "Foundations," referred to SRP Section 3.8.1 for design and analysis procedures. All of the criteria in SRP Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5, primarily addressed the effects of cracking with respect to design of concrete structures. They did not address how to treat cracking when seismic soil structure interaction analyses are performed.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.2

3.1 The criteria for considering concrete cracking in the seismic analysis of concrete structures in SRP Revision 4, Section 3.7.2 II.3.C "Modeling of Structures" was enhanced with the following new item iv (highlighted in italics).

3. Procedures Used for Analytical Modeling. A nuclear power plant facility of …

C. Modeling of Structures. Two types of structural models are widely used by …

iv. Modeling of the appropriate stiffness and damping for the various structural elements in the mathematical model is essential to obtain realistic seismic responses (e.g., ISRS, building accelerations, member forces, and displacements). For reinforced concrete structures, the stiffness used in the model depends on the degree of concrete cracking which is a function of the level of stress due to the most critical load combination. The effects of concrete cracking on membrane, bending, and shear stiffness should be considered as

appropriate in the mathematical model. Because the effect of cracking on the stiffness of concrete members is complex and depends on a number of factors, the approach used should be shown to be conservative. One approach for considering the cracked concrete properties is to reduce the stiffness properties of the uncracked members by a reduction factor. Acceptable stiffness reduction factors for cracked concrete members are given in American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) 43-05 (e.g., 0.5 for cracked walls for flexure and shear).

If structural responses (e.g., member forces, displacements, soil bearing pressures) are determined from a separate detailed finite element analysis (what is referred to as a two-step approach), the effects of concrete cracking should be considered both in the SSI analysis and the detailed structural analysis.

Further guidance on consideration of concrete cracking in the analysis and design for seismic Category I structures is provided in the acceptance criteria for design and analysis procedures presented in SRP Sections 3.8.1 and 3.8.3 through 3.8.5.

For the generation of ISRS, the guidance given below should be followed.

For a generic design, where the design-basis ISRS represent the envelope of the in-structure responses obtained from multiple analyses conducted to consider the range of expected site soil conditions associated with the certified seismic design response spectra (CSDRS), the cracked concrete properties and the associated SSE damping values in Table 1 of RG 1.61, can be used. If a CSDRS is associated with a single site condition, such as the hard-rock high-frequency (HRHF) spectra for a specific site, then the use of uncracked concrete properties with OBE damping values in Table 2 of RG 1.61, are acceptable to develop ISRS.

An acceptable approach for existing structures or site-specific designs, where it is not desirable to utilize the approach described above, a seismic analysis can be performed based on the BEs of the stiffness properties of the structural members. A mathematical model of the structure should be developed to be representative of the structure and analyzed for the uncracked stiffness properties. The analysis may be performed by assuming, for shear walls as an example, in-plane bending and shear stiffness values corresponding to the uncracked properties, and a damping value of 4 percent. After performing the seismic analysis, the calculated state of stress in the concrete members should be compared to the stresses that would cause cracking, for all load combinations that include seismic effect. If extensive cracking is determined based on this stress comparison, then the stiffness of those members should be reduced (e.g., using stiffness reduction factors). In other regions of the model where cracking does not occur, the same uncracked properties should be used, and the seismic analysis would be re-run. For those regions that are cracked, 7 percent damping may be used, while 4 percent damping should be used for the uncracked regions. The results of this analysis may be used as the basis for the ISRS,

provided there are no additional members whose state of stress leads to further significant cracking in the model. If further significant cracking is identified in some of the remaining uncracked members, then reductions in the stiffness representation of those members should be made and a re-analysis of the model performed. If the state of stress in any cracked members demonstrates that the cracked members are no longer cracked, then it is not necessary to revise the cracked member properties back to the original uncracked properties.

If any alternative methods are utilized, then adequate justification should be provided to demonstrate that the BE stiffness properties used for concrete are appropriate and that uncertainty associated with the BE stiffness values have been considered. In addition, it should be demonstrated that the SSI frequencies in both the horizontal and vertical directions are sufficiently below the amplified portion of the input design spectra so that if further cracking were to occur, then any reduction in stiffness would not increase the seismic demand. If the SSI frequencies fall above the amplified portion of the input design response spectra, then the analysis needs to evaluate the effects of further concrete cracking since this may lead to higher demand loads on the structure.

4. Technical Basis and/or Rationale

4.1 Concrete cracking can reduce the stiffness and increase damping of structures, which would reduce the frequencies and thereby may affect the seismic response analysis. In past licensing applications, questions have arisen regarding the proper consideration of the effects of concrete cracking when performing seismic analysis of structures. Part of this reflected a need for additional guidance in the SRP for acceptable methods to treat the effects of concrete cracking.

It is well recognized that the use of the appropriate stiffness for the various structural elements in the mathematical model is essential to obtain realistic seismic responses (e.g., ISRS, building accelerations, member forces, and displacements). The stiffness representation for reinforced concrete structures depends on the level of concrete cracking which is a function of the level of stress due to the most critical load combination. Concrete cracking can affect the membrane, bending, and shear stiffness of the members that should be considered in the mathematical model. Because concrete cracking phenomena are complex and depend on a number of factors, the approach used to represent its stiffness should be shown to be conservative. An acceptable approach for representing cracked concrete properties is to reduce the stiffness properties of the uncracked members by a reduction factor. Acceptable stiffness reduction factors for cracked concrete members are given in ASCE/SEI 43-05 (e.g., 0.5 for cracked walls for flexure and shear).

For the design of structures, the responses (e.g., member forces, displacements, soil bearing pressures) are typically determined from a separate detailed finite element analysis. The effects of concrete cracking in these finite element analyses should be considered in a manner consistent with the representation of cracking in the SSI analysis. Further guidance on whether cracked concrete properties should be considered in the design of seismic Category I structures is provided in the acceptance criteria for design and analysis procedures presented in SRP Sections 3.8.1 and 3.8.3 through 3.8.5.

4.2 When generating ISRS for a generic design, two situations arise in considering the site conditions. In the first situation, in which the design-basis in-structure response spectra represent the envelope of the in-structure responses obtained from multiple analyses conducted to consider the range of expected site soil conditions associated with the CSDRS, the cracked concrete properties can be used along with corresponding SSE damping values in Table 1 of RG 1.61, Revision 1. This approach is acceptable because multiple SSI analyses are performed on a range of soil conditions associated with the CSDRS and the results are enveloped which lead to conservative ISRS. The second situation deals with a single site condition where site-specific hard rock hard high-frequency (HRHF) spectra are used as part of the CSDRS. In this case, the use of uncracked concrete properties with OBE damping values in Table 2 of RG 1.61, Revision 1, is acceptable to develop ISRS. This approach is appropriate because the site-specific HRHF spectra should not cause cracking in the structural design based on the CSDRS, because the HRHF spectra typically have frequencies in the amplified region above the frequencies of structures. Therefore, the use of OBE damping with uncracked concrete properties ensures damping compatible structural response, which is consistent with the staff regulatory position C.1.2 of RG 1.61, Revision 1.

4.3 In a site-specific design, the extent of concrete cracking can be determined. In this situation, the seismic analysis should be performed using the best estimates of the stiffness properties of the structural members. This approach requires the use of a multistep process. First, the mathematical model of the structure is developed based on the uncracked stiffness properties of the structural members. Thus, for shear walls, as an example, the in-plane bending, shear stiffness, and membrane values correspond to the uncracked (monolithic) section properties, with a corresponding value of 4 percent for damping. The seismic analysis would be performed for this model and then in the second step, the calculated stresses in the concrete members should be compared to the flexural, shear, and membrane stresses that would cause cracking, for all load combinations that include seismic. In those regions where the calculated stresses exceed the cracking stress values, then the stiffness of those members should be reduced to the cracked condition and a corresponding damping value of 7 percent should be used. In the other regions of the model, where cracking does not to occur, the uncracked properties should be used along with a damping value of 4 percent. Then the seismic analysis should be run again. If the state of stress of the remaining uncracked members shows that no significant additional cracking occurs, then the solution has converged, and the results of this analysis may be used to determine the ISRS. However, if further significant cracking does occur in the remaining uncracked members, then the stiffness values of those members should be reduced and a re-analysis of the model performed. If an examination of the stresses in the cracked members demonstrate that the cracked members are no longer cracked, then it is not necessary to revise the cracked member properties back to the original uncracked properties. To do so would be unrealistic and potentially may lead to iterations back and forth that would not converge.

4.4 Alternatively, one can examine the effect of concrete cracking on the SSI frequencies to determine the impact of the extent of cracking on the seismic response analysis. The alternative approach, discussed in Item iv) of the proposed SRP criteria, is based on developing the best estimate for stiffness cracking and then considering the uncertainty associated with the best estimate stiffness value. This alternative approach requires providing adequate justification for the best estimate value and the uncertainty considered in that value.

Two examples are provided for this alternative method. In the first example, if the SSI frequencies in both the horizontal and vertical directions are sufficiently below the amplified portion of the input design spectra, then any further cracking from the best estimate stiffness values would reduce the SSI frequencies, and thereby reduce the seismic response of the structure. If the SSI frequencies are determined to be greater than the amplified portion of the input design spectra, then the effects of further concrete cracking should be evaluated because this may lead to an increase in the seismic response of the structure.

5. References

[1] ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.” American Society of Civil Engineers, 2005.

Technical Issue No. 7

Differential Settlement and Construction Sequence Considerations in Foundation Design

1. Description of Issue

Seismic Category I structures (foundations and superstructures) should be designed to take into account the additional member forces and moments induced by the combined effects of the construction sequence and the short term differential settlement of the soil under the foundation, as well as the long-term settlement expected to occur during the life of the structure. Past experience and current industry codes and standards indicate that these are important design considerations [1, 2, 3].

Standard Review Plan (SRP) Section 3.8.5 I.4 provides guidance to the staff for assessing the effects of differential settlements, construction sequence, and mat flexibility on the design of seismic Category I foundations. SRP Section 3.8.5 II.4 indicates that the review of the design and analysis procedures to address these effects is conducted on a case-by-case basis. In addition, SRP Section 3.8.5 I.7 indicates that the review of post-construction testing and in-service surveillance programs, including the monitoring of settlements and differential displacements, is performed by the staff on a case-by-case basis.

Enhancements to SRP Revision 4 Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5 were included to provide additional guidance on the following:

(1) how to consider the effects of the construction sequence and the differential settlements (including the long-term settlements expected to occur during the life of the structure) in the standard design process which postulates the geotechnical parameters that are generic (i.e., not the result of a site-specific geotechnical investigation), and does not fully specify the construction sequence
(2) the need to establish a clear interface between design certification (DC) and combined license (COL) applications that permits verification of the foundation design by the COL applicant, including implementation of a settlement monitoring program
(3) specific identification of the loads due to differential settlements and construction sequence as an area of staff review under the “loads and load combinations” section

2. Why SRP Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

2.1.1 SRP Sections 3.8.1 I.3, 3.8.3 I.3, 3.8.4 I.3, and 3.8.5 I.3 described the loads and load combinations applicable to the design of seismic Category I foundations.

2.1.2 SRP Section 3.8.5 II.4 indicated that the staff should perform a case-by-case review of the design and analysis procedures used to evaluate the effects of settlement on construction procedures, as well as the allowable settlement (total and differential) that can be accommodated by the foundations and structures. SRP Section 3.8.5 II.4 indicated that the staff should perform a case-by-case review of how loads attributable to construction are considered in the design, including loads from the construction sequence of the foundation mat and walls.

2.1.3 SRP Section 3.8.5 II.7 provided criteria for reviewing testing and in-service surveillance provisions for seismic Category I foundations.

2.2 Why Technical Criteria Were Enhanced

2.2.1 SRP Sections 3.8.1 I.3, 3.8.3 I.3, 3.8.4 I.3, and 3.8.5 I.3 did not specifically describe the loads associated with differential settlements or construction sequence as an area of staff review.

2.2.2 SRP Section 3.8.5 II.4.E and J did not provide specific guidance on how to consider the effects of the construction sequence and the differential settlements (including the long-term settlements expected to occur during the life of the structure) in the standard design process which postulates the geotechnical parameters that are generic (i.e., not the result of a site-specific geotechnical investigation), and did not fully specify the construction sequence. In addition, the acceptance criteria did not provide guidance regarding the need to establish a clear interface between DC and COL applications that permits a site-specific verification of the foundation design by the COL applicant, as Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52.47, "Contents of Applications; General Information," requires.

2.2.3 SRP Section 3.8.5 II.7 did not specifically describe the implementation of a settlement monitoring program as part of the testing and in-service surveillance provisions for seismic Category I foundations.

3. Enhancements Incorporated in Revision 4 to SRP Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5

3.1 The applicable portions of SRP Revision 4, Sections 3.8.1 I.3, 3.8.3 I.3, 3.8.4 I.3, and 3.8.5 I.3, shown below, were enhanced with the following additional statements (highlighted in italics):

SRP Section 3.8.1 I.3

A. Those loads encountered during construction of the containment, including dead loads, live loads, prestress loads, temperature, wind, earth pressure, snow, rain, and ice, and construction loads that may be applicable such as material loads, personnel and equipment loads, horizontal *and vertical* construction loads, *loads that are induced by the construction sequence and by the differential settlements of the soil under and to the sides of the containment building*, erection and fitting forces, equipment reactions, and form pressure.

SRP Section 3.8.3 I.3

A. Loads encountered during construction of containment internal structures, including dead loads, live loads, prestress loads, temperature, wind, earth pressure, snow, rain, and ice, and construction loads that may be applicable, such as material loads, personnel and equipment loads, horizontal *and vertical* construction loads, *loads that are induced by the construction sequence and by the differential settlements of the soil under and to the sides of the containment building,* erection and fitting forces, equipment reactions, and form pressure.

SRP Section 3.8.4 I.3

A. Those loads encountered during construction of the seismic Category I structures which include dead loads, live loads, prestress loads, temperature, wind, earth pressure, snow, rain, and ice, and construction loads that may be applicable such as material loads, personnel and equipment loads, horizontal *and vertical* construction loads, *loads that are induced by the construction sequence and by the differential settlements of the soil under and to the sides of the structures,* erection and fitting forces, equipment reactions, and form pressure.

SRP Section 3.8.5 I.3

3. Loads and Load Combinations. The review includes information pertaining to the applicable design loads and their various combinations. The loads normally applicable to seismic Category I foundations are the same as those applicable to the structures that the foundations support. SRP Section 3.8.1, Subsection I.3 describes these loads for the containment foundation, and SRP Section 3.8.4, Subsection I.3 details such loads for all other seismic Category I foundations. *These should also include the loads that are induced by the construction sequence and by the differential settlements of the soil under and to the sides of the structures.*

3.2 SRP Section 3.8.5 I "Review Interfaces" was enhanced by the following additional statements (highlighted in italics):

1. The determination of structures that are subject to a quality assurance program in accordance with the requirements of Appendix B to 10 CFR Part 50 is performed under SRP Sections 3.2.1 and 3.2.2. The review of safety-related structures is performed on that basis.

2. The determination of pressure loads from high-energy lines located in safety-related structures other than containment is performed in accordance with SRP Section 3.6.1. The loads thus generated are included in the load combination equations of this SRP section.

3. The determination of loads generated by pressure under accident conditions is performed in accordance with SRP Section 6.2.1. The loads thus generated are included in the load combinations in this SRP section.

4. The review for quality assurance is coordinated and performed in accordance with Chapter 17.

5. The review for foundation settlement, effects of settlement on construction procedures, and modeling of soil stiffness for various loading conditions, as described in SRP Sections 3.8.5 II.4 E, J, and K, is coordinated with the review under SRP Section 2.5.4. The modeling of soil stiffness for seismic loading is coordinated with the review under SRP Section 3.7.2.

3.3 SRP Section 3.8.5 II.4 was enhanced with the following additional statements (highlighted in italics):

E. Detail*ed* explanation of how settlement *is evaluated,* ~~(~~including potential effects of static or dynamic differential settlement, *dependence on time (i.e., short term vs. long term), effect of the soil type (i.e., granular vs. cohesive), and effect of the foundation type and size (e.g., basemats, spread footings)*~~was considered~~. Evaluation~~*and consideration*~~ of the effects of settlement on construction procedures. Evaluation of the allowable settlement (total and differential) that can be accommodated in the foundation/structures.~~?~~

F. The maximum toe pressure for basemat design under worst-case static and dynamic loads and its justification.

G. The *evaluation of* stiff and soft spots ~~*evaluation*~~ in the foundation soil to maximize the bending moments used in the design of the foundation mat.

H. Description of the design details of critical locations, such as the junction of sidewall and basemat and the junctions of basemat to sumps.

I. Detail explanation of the load path from all superstructures to the foundation mat to the subgrade. Discussion of any unique design features that occur in the load path (e.g., any safety-related function that the tendon gallery may have as part of the foundation in a prestressed containment or the connection of any internal structures to a steel containment and its supporting foundation).

J. Explanation of how loads attributable to construction are ~~*considered*~~*evaluated* in the design. Some examples of items to be discussed include the excavation sequence and loads from the construction sequence of the *mat* foundation ~~*mat*~~ and walls, as well as the potential for loss of subgrade contact (e.g., because of loss of cement from a mud mat) that may lead to a differential pressure distribution on the mat.

K. An essential aspect of the design and analysis procedures for seismic Category I foundations is the stiffness modeling of the soil material under and to the sides of the structures. Soil stiffness can be represented by means of analytical or numerical (e.g., solid finite elements, distributed springs) formulations that are appropriate for the loading conditions as well as for the soil type, foundation type and size, and time scale being considered.

In the case of seismic dynamic loads, the soil stiffness parameters should be consistent with the magnitude of soil strains assumed in the SSI analysis described in SRP Section 3.7.2, which are associated with the relatively short time scale of the seismic input. The distribution of toe bearing pressures used in foundation design should be consistent with the distribution of toe bearing pressures obtained from the SSI analysis.

In the case of gravity loads and basemat foundations, the soil stiffness parameters should be consistent with: (a) dishing or Boussinesq effects (if

uncoupled distributed springs are used then it may be necessary for the stiffness to be increased at the edges and reduced at the center of the basemat footprint); (b) basemat size (subgrade modulus could be highly dependent on basemat dimensions); (c) time scale of the loads (i.e., short term construction loads vs. long term loads present throughout the life of the structure); and (d) soil type (i.e., granular vs. cohesive soils).

Appropriate stiffness parameters are particularly important when evaluating loads induced by the construction process and by differential settlements, as described in items E and J above. Additional guidance to consider in the review of DC and COL applications is given below.

i. In the case of a DC application, the staff reviews the following information:

- *Postulated set of soil stiffness parameters for the construction phase and the technical bases for its selection, for all soils within the zone of influence surrounding the structures. The zone of influence is defined as that region to the side and below the structure that may induce loads on the structure if induced settlements occur and/or loads are applied within the zone.*

- *Postulated set of soil stiffness parameters for the post-construction phase and the technical basis for its selection.*

- *Postulated construction sequence and corresponding set of construction loads and the technical basis for its selection.*

- *Analysis methodology for computing soil settlements (total and differential, short term and long term), which should incorporate the postulated soil stiffness and construction sequence, and include potential long term settlement effects through the life of the structure.*

- *Analysis methodology for computing member forces and moments induced by the settlements and construction sequence, which should then be taken as a separate construction sequence/soil settlement load case, to be included in the structural design of the foundation and superstructure in addition to all other load cases.*

- *Interface considerations between DC and COL applications (e.g., COL action items and appropriate acceptance criteria) that permit verification of the foundation design by a COL applicant. An acceptable interface consideration is to incorporate the settlement profiles computed for a postulated construction sequence (the various stages of construction and post construction), which a COL applicant can then use for verification purposes in conjunction with predictive calculations (associated with the actual construction sequence) and a settlement monitoring program.*

The acceptance criteria for these verifications need to be clearly identified.

- *Development of a short term and long-term settlement monitoring program that can detect both vertical and horizontal movements in and around the structures, as well as differential distortion across the foundation footprint, from the beginning of construction at the site. The movements induced by site excavation, backfill, and re-compaction should be included in the monitoring program. Consideration should be given to maintaining all or pieces of the monitoring program over the life of the structure.*

ii In the case of a COL application that incorporates a DC application by reference, the staff reviews the following information:

- *In addition to the geotechnical investigation performed in SRP Section 2.5.4, the staff reviewer for this section should review, the site-specific geotechnical investigation program to determine predicted settlements during construction and post construction, based on the construction sequence to be used. The investigation program should be carried to sufficient depths to be able to ascertain these properties over the depth considered important to the settlement analyses. The methodology for site-specific settlement analyses should be consistent with the corresponding methodology in the DC application.*

- *Settlement monitoring program to verify whether measured settlements and distortions are consistent with predicted site-specific settlements during construction and post-construction phases, through the life of the structure.*

- *Verification of the interface considerations between DC and COL applications, which should be based on the information described in the above two bullets.*

The above considerations are appropriate for major seismic Category I foundations. Alternative, more simplified, approaches are acceptable for the case of smaller structures if adequate justifications are provided.

In the case of adjacent structures connected by appurtenances (non-flexible commodities, such as piping and conduit), the staff reviews the design criteria for total settlement and for relative settlement between adjacent structures to ensure consistency with the criteria used in the design of the appurtenances.

3.4 SRP Section 3.8.5 II.7 was enhanced by the following additional statements (highlighted in italics):

For *seismic* Category I foundations, it is important to accommodate inservice inspection of critical areas. The staff considers monitoring and maintaining the condition of *seismic* Category I foundations as essential for plant safety. *It is also important that a foundation*

monitoring program include monitoring of settlements (both differential and total) during construction and post construction to ensure that the foundation continues to perform as designed.

Any special design provisions (e.g., providing sufficient physical access, supplying a means for identification of conditions in inaccessible areas that can lead to degradation, performing remote visual monitoring of high-radiation areas) to accommodate inservice inspection of *seismic* Category I foundations are reviewed on a case-by-case basis.

4. Technical Basis and/or Rationale

The bearing pressures imposed by a seismic Category I foundation on the surface of the underlying soil introduce stresses that cause the latter to deform, which ultimately leads to settlement of the foundation and the superstructure. Because soils are nonhomogeneous media and loads are not applied uniformly to the foundation footprint, the resulting settlements are not uniformly distributed. This means that certain areas of the foundation settle more or less than others; this is known as differential settlement. Differential settlement induces additional stresses on the foundation and superstructure that need to be evaluated and accounted for in the structural design. In addition, settlement does not occur instantaneously but is recognized as a time-dependent process. For certain types of soils (e.g., sandy soils), most of the settlement occurs during and shortly after construction due to soil compaction; however, for other types of soils (e.g., clayey or silty soils), settlement may continue for significantly longer periods of time because of soil consolidation. The structural design needs to account for these time-dependent effects.

The construction sequence also imposes additional stresses on the foundation and superstructure that are to a large extent dependent on the (short term) stiffness of the soil under the foundation. In particular, the construction process for pouring heavy foundation sections, such as those for a typical Nuclear Island basemat, needs to be carefully reviewed to ensure that differential settlements, particularly at softer soil sites, do not cause segmental cracking or any other distress to the structural system. For example, in past nuclear power plant (NPP) designs, applicants have performed studies to identify limitations to the construction process to ensure that relatively uniform loads are applied over the foundation footprint. In light of these considerations, it is clear that the review of the effects of the construction sequence and (short-term) differential settlements should be performed concurrently since they are closely related.

From the point of view of a DC application, it is necessary to consider the following:

- how the effects of the differential settlements are accounted for in the standard design process, where assumptions need to be made regarding generic soil parameters
- what specifications are established such that a COL applicant can demonstrate, for a particular construction sequence, that forces and moments induced by predicted and measured settlements at a particular site are bounded by those considered in the standard design

It is emphasized that these issues are inherently site specific. Therefore, it can be challenging for a DC applicant to establish an interface that allows for the standard design to account for construction sequence and settlement loads. It can also be challenging to permit a COL applicant to verify that these loads are not exceeded during or after construction [1]. The

enhancements included in SRP Revision 4 are intended to provide guidance to applicants about the interface issue.

The standard design should consider (1) a postulated set of soil stiffness parameters for the construction phase, (2) a postulated set of soil stiffness parameters for the post construction phase, and (3) a postulated construction sequence and corresponding set of construction loads.

To account for construction sequence and settlement loads in the standard design, it may be necessary to perform sequential finite element (FE) analysis of the foundation and superstructure with detailed modeling of the supporting soil stiffness and construction sequence, including anticipated effects through the end of the operating life of the structure. The sequential FE analysis should be based on the postulated soil conditions and validated with geotechnical soil settlement analysis (see discussion below on geotechnical analysis codes, a certain degree of iteration/feedback between the structural and geotechnical analyses would probably be necessary for this validation). The envelope of forces and moments computed during the sequential FE analysis can then be compared with corresponding forces and moments obtained from a “reference” FE analysis that does not include construction sequence or settlement effects. From this comparison, any difference in forces and moments are taken as a separate construction sequence/soil settlement load case, to be considered in the structural design of the foundation and superstructure in addition to all other load cases, in accordance with ACI 349-06, Section 9.2.2. Settlement profiles at all stages of the sequential FE analysis should also be computed; a COL applicant can then use these profiles for verification purposes, as described below. The intent of the proposed revision is that such a detailed analysis be carried out as part of the standard design process for major seismic Category I foundations such as the Nuclear Island basemat. Alternative approaches are also acceptable if adequate justifications are provided. For example, in the case of smaller structures it may be sufficient to prescribe a uniform construction for the entire structure if the magnitude of the differential settlements and corresponding induced stresses are not significant.

When evaluating the effect of differential settlements on foundation design, the soft soil condition postulated in a standard design could be bounding for a majority of potential site conditions. However, as noted above, soft clay soils behave differently from soft sandy soils, especially on long-term settlements. Therefore, sequential FE analysis should consider the soil conditions, clay or sand, whichever results in greater induced moments and forces on the foundation and superstructure. In addition, the possibility that a stiffer soil could result in greater induced moments or forces in certain areas of the foundation or superstructure should also be investigated. In the proposed revision, the staff reviews the technical basis for the postulated set of soil stiffness parameters (more than one may be needed) to ensure these issues are addressed.

In the discussion above, it is assumed that the total magnitude of soil settlements does not affect the design of the structure during either construction or post-construction phases. It should be noted that it is not so much the total magnitude of the settlements that affects the structural performance of the foundation and superstructure; rather, it is the relative shape of the settlement profile in terms of slope and curvature because only the latter induces stresses. This last statement is only valid when considering an individual structure; total settlement is clearly of interest when considering adjacent structures connected by appurtenances (nonflexible commodities, such as piping and conduit), which would need to be addressed in the design. In the proposed SRP enhancement, the staff reviews the criteria for total settlement and for

relative settlement between adjacent structures. This review is in addition to the review for sufficient gap between structures for seismic loading plus building settlement/tilt.

Based on the settlement profiles established in the DC, a COL applicant should perform a site-specific geotechnical investigation to determine predicted settlement profiles for construction and postconstruction phases, based on the construction sequence to be used and the same methodology as the DC application. If the predicted settlement profiles compare favorably to the DC settlement profiles—in terms of slope and curvature across the foundation footprint, not necessarily in absolute magnitude—then it is inferred that the forces and moments induced by the predicted settlements are bounded by the forces and moments considered in the standard design. This comparison can be made in terms of the "angular distortion" concept, as described in the U.S. Army Corps of Engineers (USACE) Manual No. 1110-1-1904 [5]. In addition to the predictive calculations, a settlement monitoring program should to be established to verify whether measured settlements across the foundation footprint are consistent with predicted settlements during the operating life of the structure. The intent of the SRP enhancement is to have a COL applicant perform these verification activities as part of the interface considerations between the DC and COL applications.

Finally, the SRP enhancement includes general guidance regarding key issues to consider in the review of soil stiffness models utilized for the design of seismic Category I foundations, especially under static/gravity load conditions. It is indicated that soil stiffness can be represented by analytical or numerical (e.g., solid finite elements, distributed springs). formulations; however, it also is emphasized that that these formulations should be appropriate to the loading condition (seismic vs. gravity), soil type (granular vs. cohesive), foundation type and size (basemat, spread footing), and time scale of the loads (very short term seismic, short term construction, long term gravity) being considered.

Based on review experience with past DC and COL applications, structural design procedures typically utilize FE models in which the foundation and superstructure are discretized with sufficient details. However, the soil is simplistically represented with distributed springs with stiffness parameters based on the subgrade modulus concept. Therefore, the staff's review should focus on whether the stiffness of the distributed springs is appropriate to each analysis case. A certain spatial variation of stiffness may be required to capture dishing or Boussinesq effects when using distributed springs under static/gravity loads (stiffer at the edges and more flexible at the center of the basemat footprint). For seismic loads, on the other hand, it is difficult to use distributed springs to represent the overall dynamic foundation stiffness (which is frequency dependent and consists of independent vertical, horizontal, rotational and torsional components) in a manner consistent with the seismic SSI analysis. If distributed springs are used to represent dynamic soil stiffness under seismic conditions, especially for computing bearing pressures for foundation design purposes, then adequate technical justification needs to be provided.

The soil stiffness models used in structural design should be contrasted with those considered in geotechnical analysis codes (e.g., PLAXIS, FLAC, or SIGMA/W), which often incorporate relatively sophisticated constitutive models of the soil continuum but have only modest structural capabilities. The geotechnical settlement analysis used in the evaluation of differential settlement and construction sequence effects under SRP Section 3.8.5 is expected to be performed using such codes. However, it is not the intent of the SRP Section 3.8.5 enhancements to duplicate the settlement analysis reviewed under SRP Section 2.5.4. Based

on past review experience, the latter review does not typically consider the coupling between the geotechnical and structural aspects that is essential to address foundation design issues. In particular, the iteration/feedback that would be necessary between the structural and geotechnical analyses as described in the SRP Section 3.8.5 enhancements would typically not be performed under SRP Section 2.5.4.

5. References

[1] Miranda, M., Braverman, J., Wei, X., Hofmayer, C. and Xu, J., "Structural Design Challenges in Design Certification Applications for New Reactors," Paper PVP2011-57600, Proceedings of the ASME 2011 Pressure Vessels & Piping Division Conference (PVP2011), July 2011, Baltimore, MD.

[2] American Concrete Institute (ACI) Committee 336, "Suggested Analysis and Design Procedures for Combined Footings and Mats," Report ACI 336.2R-88 (reapproved 2002), American Concrete Institute, Farmington Hills, MI, 1988.

[3] American Concrete Institute (ACI) Committee 349, "Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-06) and Commentary," Farmington Hills, MI, 2006.

[4] U.S. Army Corps of Engineers (USACE), "Engineering and Design: Settlement Analysis," Engineer Manual No.1110-1-1904, Washington, DC, 1990.

Technical Issue No. 8

Artificial Time History Development

1. Description of Issue

Revision 4 to Standard Review Plan (SRP) Section 3.7.1 for developing design time histories for soil structure interaction (SSI) analyses has included additional enhancements to provide further clarities regarding the selection of the appropriate seed earthquake records, criteria to perform spectral matching, and criteria regarding the use of time histories for nonlinear analyses.

2. Why SRP Section 3.7.1 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The criteria for developing design time histories are presented in SRP Section 3.7.1 II.1.B.

2.1.1 The criteria indicated that the safe shutdown earthquake (SSE) and operating basis earthquake (OBE) design ground motion time histories can be either real time histories or artificial time histories. In addition, artificial time histories which are not based on seed recorded time histories should not be used.

2.1.2 In SRP Section 3.7.1 II.1.B.ii, corresponding to Option 1 (used for single time histories), Approach 2, criteria were given to provide guidance on how to demonstrate the adequacy of the artificial time histories generated. Item (d) of the criteria for the spectral matching process stated:

> "In lieu of the power spectrum density requirement of Approach 1, the computed 5 percent damped response spectrum of the artificial ground motion time history shall not exceed the target response spectrum at any frequency by more than 30 percent (a factor of 1.3) in the frequency range of interest. If the response spectrum for the accelerogram exceeds the target response spectrum by more than 30 percent at any frequency range, the power spectrum density of the accelerogram needs to be computed and shown to not have significant gaps in energy at any frequency over this frequency range."

2.1.3 SRP Section 3.7.1 I.1.B stated:

> "In some instances, a nonlinear analysis of the SSCs may be appropriate (e.g., the evaluation of existing structures). Multiple time history analyses incorporating real earthquake time histories are appropriate when such analyses are proposed. The adequacy of time histories used for the nonlinear analyses is reviewed."

SRP Section 3.7.1 II.1.B, Option 2, stated:

> "As discussed in Section I.1.B and Section II.1.B of this SRP section, the use of multiple real or artificial time histories for analyses and design of SSCs is acceptable. For linear structural analyses, a minimum of four times histories should be used. For nonlinear structural analyses, the number of time histories must be greater than four and the

technical basis for the appropriate number of time histories is reviewed on a case-by-case basis."

2.2 Why Technical Criteria Were Enhanced

2.2.1 When selecting seed recorded time histories for use in generating artificial records matching a given target response spectrum, it has been shown that the response of the structures determined from SSI analyses using those artificial records can be quite sensitive to the seed [1]. If the seed is not selected appropriately, difficulties may arise in the spectral matching process and also in obtaining realistic building responses. This was shown to be the case during the design certification process in which selecting a different seed recorded time history resulted in differences in responses of as much as 30 percent. As an example, selecting a seed recorded time history from a WUS earthquake event, to generate a time history intended to match a target design spectrum that includes central and eastern United States (CEUS) hard rock high frequencies (HRHFs) could result in a poor spectral match.

2.2.2 If Option 1, Approach 2, is used for developing a single set of design time histories, Item (d) of the SRP criteria provided two alternative options for demonstrating that there are no significant gaps in energy at any frequency. The process involved either (1) demonstrating that the computed 5 percent damped response spectrum of the artificial ground motion time history does not exceed the target response spectrum at any frequency by more than 30 percent (a factor of 1.3) in the frequency range of interest or (2) demonstrating that the power spectrum density of the accelerogram does not have significant gaps in energy at any frequency over this frequency range. To ensure a more accurate spectral matching process whereby gaps in energy at the frequencies of interest are minimized, both existing methods should be performed rather than presented as two alternatives.

2.2.3 For Option 2, which is applicable to the use of multiple sets of time histories, the criteria in the SRP did not provide sufficient guidance to the number of time histories that should be considered when nonlinear time history analyses are performed. In addition, the criteria did not explain whether the average or maximum values from the multiple time history analyses should be used when performing multiple time history analyses. Nonlinear response analyses are considered to be very complex, and the results are found to be sensitive to the selection of different ground motions or system properties. Therefore, some additional guidance for nonlinear analysis was needed.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.1

3.1 The applicable portion of SRP Section 3.7.1 II.1.B and the included enhancements (highlighted in italics) are shown below.

> B. Design Time Histories. The SSE and OBE design ground motion time histories can be either real time histories or artificial time histories. To be acceptable, the design ground motion time histories should consist of three mutually orthogonal directions - two horizontal and one vertical. For both horizontal and vertical input motions, either a single time history or multiple time histories can be used. When time histories are used, each of the three ground motion time histories *should~~must~~* be shown to be statistically independent from the others. Each pair of time histories are considered to be statistically independent if the absolute value of their correlation coefficient does not

exceed 0.16. Simply shifting the starting time of a given time history cannot be used to establish a different time history. ~~*Also, artificial time histories which are not based on seed recorded time histories should not be used.*~~ *When the seed time histories are selected from earthquake records, the response spectra corresponding to the seed record should be similar in shape to the target spectra across the frequency range of interest to the analysis (e.g., Houston, et al., 2010) and phasing characteristics of the earthquake records should not change significantly. If the target spectra include multiple characteristic events, a single recorded earthquake time history may not able to capture the response characteristics of the target spectra. To this end, the use of multiple time histories may be appropriate in which individual time histories are developed from earthquake records fairly representing the characteristic events embodied in the target spectra. Alternatively, an artificial time history may be developed using random generation routines or through the use of multiple time history techniques. If a random time history generator technique is used to develop the seed time histories, then acceptability of the seed will be reviewed on a case-by-case basis. For generated time histories, it should be demonstrated that acceleration, velocity, and displacement are compatible and do not result in displacement's baseline drift.*

3.2 The criteria in SRP Section 3.7.1 II.1.B .ii (Option 1, Approach 2), Item (d) were enhanced with the following (highlighted in italics):

Option 1: Single Set of Time Histories. To be considered acceptable, the …

ii. Approach 2. For Approach 2, the *design*~~artificial~~ ground motion time histories ~~*that are generated*~~ to match or envelop the design response spectra *should be developed*~~*shall comply*~~ with Steps (a) through (d) below. The general objective is to generate a modified recorded or artificial accelerogram which achieves approximately mean based fit to the target response spectrum; that is, the average ratio of the spectral acceleration calculated from the accelerogram to the target, where the ratio is calculated frequency by frequency, is only slightly greater than "1." The aim is to achieve an accelerogram that does not have significant gaps in the Fourier amplitude spectrum, but which is not biased high with respect to the target…

(d) ~~*In lieu of the power spectrum density requirement of Approach 1, the*~~ The computed 5*%* ~~percent~~ damped response spectrum of the *acceleration* ~~artificial ground motion~~ time history should not exceed the target response spectrum at any frequency by more than 30*%* ~~percent~~ (a factor of 1.3) in the frequency range of interest. *In addition,* ~~*If the response spectrum for the accelerogram exceeds the target response spectrum by more than 30% at any frequency range,*~~ the power spectrum density of the accelerogram should be computed and shown to not have significant gaps in energy at any frequency over this frequency range.

~~*Artificial*~~*If the design* ground motion time *history,*~~*histories*~~ defined *in Approach 2*~~*as described*~~ *above, is intended to be compatible to a site-specific FIRS, it* should have characteristics consistent with characteristic values for the magnitude and distance of the appropriate controlling events defined for the *corresponding* uniform hazard response spectrum (UHRS).

3.3 The criteria in SRP Section 3.7.1 II.1.B Option 2, were enhanced with the following additional guidance (highlighted in italics):

> Option 2: Multiple Sets of Time Histories. As discussed in Section I.1.B and Section II.1.B of this SRP section, the use of multiple real or artificial time histories for analyses and design of SSCs is acceptable. For linear structural analyses, a minimum of four times histories should be used *(NUREG/CR-5347)*. For nonlinear structural analyses, the number of time histories must be greater than four and the technical basis for the appropriate number of time histories are reviewed on a case-by-case basis. This review also includes the adequacy of the characteristics of the multiple time histories.
>
> The response spectra calculated for each individual time history may not envelop the design response spectra. However, the multiple time histories are acceptable if the average calculated response spectra generated from these time histories envelop the design response spectra. An acceptable method to demonstrate the adequacy of a set of multiple time histories, in terms of enveloping criteria and having sufficient power over the frequency range of interest, is to follow the procedures described for Approach 2 presented in Subsection II.1.B.ii of this SRP. When implementing Approach 2, the criteria in paragraphs (a) and (b) of this approach should be satisfied for each of the time histories. The criteria in paragraphs (c) and (d) of this approach can be satisfied by utilizing the results for the average of the suite of multiple time histories.
>
> *When calculating the response of structures (e.g., accelerations, member forces, and displacements) from linear analyses, the average value of the responses from the multiple time histories may be used. When calculating the response of structures from nonlinear analyses (e.g., seismic evaluation of as-built structures), the average value of the responses from the multiple time histories may be used if at least seven nonlinear time history analyses are performed. Otherwise, the maximum value (i.e., envelope) of the individual responses from the multiple time histories should be used.*
>
> *In addition, if the extent of the nonlinear response is found to be significant or if the nonlinear response due to one or several time histories is found to be substantially different than the other results, then additional time histories should be considered. If there is a particular ground motion or time history analysis that dominates the response values, it should not be replaced with another motion or analysis to reduce the responses. Also, if a ratcheting effect is noted (e.g., increasing deformation with subsequent cycling of earthquake motion), then the system characteristics should be reviewed to ensure that they have been conservatively considered or the design should be revised to eliminate this behavior.*

4. Technical Basis and/or Rationale

4.1 When the seed recorded time history is selected based on a reasonable comparison of the spectral shape of the seed with the target design spectra, then it would facilitate and ensure that a good spectral matching can be achieved. This occurs because the magnitude of the seed record could be increased without the need for making significant adjustments at each frequency. This approach avoids increasing the input at certain frequencies too much which could lead to significant over-prediction of the response at certain frequencies and cause underprediction at adjacent frequencies.

4.2 The first criterion in Item (d) of the SRP ensures that there is no significant overprediction of the response spectrum of the artificial ground motion time history. The second criterion ensures no significant energy gap at any frequency over the frequency range of interest. Both criteria should be satisfied to ensure that overestimation of the spectral matching at certain frequencies would not cause significant deficiencies in energy at other frequencies. In particular for assessing power spectral densities, a relatively smooth PSD curve does not necessarily indicate an assurance of no power deficiency in the spectral matching time history. Staff's experience in applying SRP 3.7.1 Option 1, Approach 1 in conjunction with Appendix A target PSD show that there are cases where a relatively smooth PSD can be lower than the target PSD for some range of frequencies.

4.3 In view of the complexities of performing nonlinear time history analyses, it is difficult to identify a specific number of time history input motions that should be used. To do so would require additional research that should include performing many nonlinear time history analyses on various structures and components. However, there is some guidance in other standards on selection of the number of earthquake motions and how to select the responses from the multiple nonlinear time history analyses. For seismic analysis and design of buildings, Section 1618 of the 2003 International Building Code (IBC) [2] identifies that dynamic analysis including nonlinear time history analysis shall be performed in accordance with the requirements of the American Society of Civil Engineer (ASCE) 7 Standard. Section 16.2.4 of ASCE/SEI 7-05 [3] indicates the following:

> "If at least seven ground motions are analyzed, the design values of member forces, Q_E, member inelastic deformations, Ψ, and story drift, Δ, are permitted to be taken as the average of the Q_{Ei}, Ψ_i, and Δ_i values determined from the analyses. If fewer than seven ground motions are analyzed, the design member forces, Q_E, design member inelastic deformations, Ψ, and the design story drift, Δ, shall be taken as the maximum value of the Q_{Ei}, Ψ_i, and Δ_i values determined from the analyses."

The commentary to IBC 2003 indicates that the code's earthquake load requirements are based on ASCE 7 and the National Earthquake Hazards Reduction Program's (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings (FEMA 368) [4]. This Federal Emergency Management Agency (FEMA) standard is superseded by the 2009 edition of FEMA P-750, entitled "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures" [5]. The commentary to FEMA P-750 discusses the number of ground motions to be used in a nonlinear time history analysis and the use of the average values or maximum values of deformations in a similar manner to the ASCE 7 standard. In addition, the commentary to FEMA P-750 indicates the following:

> "…It is very important to note, however, that assessment of deformations in this manner should not be done without careful inspection of the story displacement histories of each analysis. It is possible that the maximum displacement or drift may be completely dominated by the response from one ground motion, and such dominance, when due to ratcheting (increasing deformations in one direction resulting in a high residual deformation), may be a sign of imminent dynamic instability. Where these kinds of dynamic instabilities are present, the analyst should attempt to determine the system characteristics that produce such effects. The ground motion that produces dynamic instability should not be replaced with one that does not."

In order to capture the items discussed above and provide enhanced guidance, some enhancements and additions to SRP Section 3.7.1 are included in Section 3.3 of this document.

5. References

[1] Houston, T. W., Mertz, G. E., Costantino, M. C., Costantino, C. J., "Investigation of the Impact of Seed Record Selection on Structural Response," Proceedings of ASME 2010 Pressure Vessels and Piping Division/K PVP Conference (PVP 2010), July 18–22, 2010, Bellevue WA.

[2] 2003 International Building Code. International Code Council, Inc.

[3] American Society of Civil Engineers (ASCE)/SEI 7-05, "Minimum Design Loads for Buildings and Other Structures," Reston, VA, American Society of Civil Engineers: Structural Engineering Institute, 2010.

[4] National Earthquake Hazards Reduction Program's (NEHRP), "Recommended Provisions for the Development of Seismic Regulations for New Buildings" (FEMA 368), Washington, DC.

[5] Federal Emergency Management Agency (FEMA) P-750, "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures," Washington, DC, 2009.

[6] NUREG/CR-5347, "Recommendations for Resolution of Public Comments, Seismic Design Criteria," June 1989

Technical Issue No. 9

Standard Plant Site Parameters and Consideration for Seismic Design Basis

1. Description of Issue

The ground motion input to a facility represents the motions obtained considering the effects from all credible seismic sources affecting the site. Therefore, the ground motion input and the site profiles through which the ground motion is established are coupled, and together they form the basis for developing the seismic input to the structure. The Standard Review Plan (SRP) Section 3.7.2 II.4 (under Specific Guidelines for SSI Analysis) provides guidance on selecting site-specific soil profiles consistent with the ground motion input to the soil-structure interaction (SSI) analysis including considerations of uncertainty in soil properties throughout the soil column depth from generic bedrock to the ground surface. However, prior to Revision 4 to SRP, guidance was only related to site-specific designs. During recent and current design certification (DC) technical reviews of the seismic design basis, the staff has noted a clear need to expand this SRP guidance on the seismic design basis for a standard plant design to ensure consistent application of the generic profiles and associated certified seismic design response spectra (CSDRS). Furthermore, SRP guidance related to specification of site parameters for a standard plant should also be enhanced to foster greater consistency in both DC application content and the staff's review process. Accordingly, Revision 4 to SRP Section 3.7.2 included an update to address this enhancement.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

There was no guidance in earlier revisions of SRP for reviewing the adequacy of generic site profiles and associated free-field ground motion response spectra selected by a DC applicant for developing a standard plant seismic design basis.

2.2 Why Technical Criteria Were Enhanced

The standard plant seismic design basis consists of generic site profiles and associated CSDRS. The absence of review guidance in the SRP could potentially lead to a frequency mismatch between these seismic design basis parameters since both are frequency-dependent attributes. To ensure an adequate seismic design, the range and number of selected profiles should be sufficient for the assumed site condition and for the associated CSDRS so that the CSDRS can be appropriately amplified through the profiles into the supported structure.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.2

The following statement was inserted at the end of SRP Section 3.7.2 I.4 (highlighted in italics):

> *For a DC application, the number and characteristics of generic site profiles are reviewed to ensure that there is an adequate seismic design basis for a standard plant.*

Enhanced SRP Section 3.7.2 II.4, under "Specific Guidelines for SSI Analysis" as shown below (highlighted in italics).

Revised the fourth bullet:

- ~~*For cases using standard plant designs, where the site specific spectra fall below the standard plant design spectra, the SSI evaluations are addressed in the standard plant design.*~~*For a COL application referencing a standard plant design, where the site-specific FIRS are enveloped by the standard plant CSDRS determined at the foundation level, the SSI evaluations are addressed in the standard plant design. However, it is necessary to confirm that the site-specific, strain-dependent soil properties, including consideration of uncertainty, are consistent with the generic site profiles used in the standard plant design. If this is not the case, then a site-specific SSI analysis is needed*.

Revised the firth bullet:

- Enough SSI analyses should be performed so as to account for the effects of the potential variability in the properties of the soils and rock at the site. At least three soil/rock profiles should be considered in these analyses, namely, a ~~*best estimate (BE)*~~ profile, a ~~*lower bound (LB)*~~ and an ~~*upper bound (UB)*~~ profile in the evaluation of SSI effects. The properties of each layer of the site profile are typically defined in terms of its low-strain shear modulus and strain-dependent modulus degradation and strain-dependent hysteretic damping properties. These may be determined from dynamic laboratory testing of the site materials, information obtained from the published literature, or both. The set of properties appropriate for a given soil is reviewed for its adequacy. *Guidance is also provided in DC/COL-ISG-017*.

Inserted after the fifth bullet:

- *For a DC application, the postulated site profiles to be used in the seismic SSI analysis are defined. The CSDRS should be shown to be appropriate for these postulated site profiles in frequency content by demonstrating that the frequencies for the amplified portion of the CSDRS are consistent with the site profile column frequencies. Otherwise, the postulated site profiles will not be able to propagate the CSDRS in the SSI analysis, and thereby, will not subject the SSCs to the amplified response over the frequency range of interest to the SSI.*

4. Technical Basis and/or Rationale

The design of a standard plant is based on assumed site parameters that provide the characteristics of the generic site for the plant. As 10 CFR 52.47 (a) (1) requires, the DC application must contain the site parameters postulated for the design, and an analysis and evaluation of the design in terms of those site parameters. In 10 CFR 52.47 (a) (2), the design characteristics of SSCs are specified, with emphasis on performance requirements and adequacy to perform their intended functions. Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing of Production and Utilization Facilities," Appendix A, "General Design Criteria for Nuclear Power Plants" (GDC) 2, provides minimum requirements for the seismic design basis of safety-related SSCs.

In accordance with 10 CFR 52.47 (a) (1) and (2), described above, the DC applicant defines the postulated site parameters which include the site profiles to be used in the seismic analysis of the SSCs. The applicant defines the site profiles that are specifically applicable to the DC application. Based on the assumed site profiles, a free-field input ground motion, referred to as the certified seismic design response spectra (CSDRS) is established, which should be appropriate to the postulated site profiles. Together the postulated soil profiles and CSDRS define the seismic design basis for the plant.

The CSDRS should be appropriate to the postulated site profiles in frequency content because the site profile frequency characteristics determine the amplification of the input ground motion through the site. Using a seismic soil-structure interaction analysis, the seismic ground motion input spectra is propagated through the site profiles into the structure in terms of seismic response or demands for design of structural members and supported equipment. The amplification of the input ground motion occurs at the profile column frequencies. So a CSDRS containing significant amplification at frequencies other than those of the profile columns is not consistent with the site parameters and suggests seismic demand loads that are not realistic since they will not propagate through the postulated site profiles. Therefore, the proposed criterion indicates that the CSDRS should be shown to be appropriate to the postulated site profiles in frequency content by demonstrating that there is a reasonable distribution of the site profile column frequencies over the amplified portion of the CSDRS.

The proposed enhancement improves the SRP guidance in the following aspects:

- It expands the scope of the review to include consideration of the generic site profiles and the corresponding appropriate CSDRS to ensure an adequate seismic design basis
- It clarifies that for a COL application referencing a standard plant design, where the site-specific FIRS fall below the standard plant CSDRS, it is also necessary to confirm that the site-specific, strain-dependent soil properties, including consideration of uncertainty, are consistent with the generic site profiles used in the standard plant design. If this is not the case, then a site-specific SSI analysis is needed
- For a DC application, the CSDRS should be shown to be appropriate to these postulated site profiles in frequency content by demonstrating that there is a reasonable distribution of the site profile column frequencies over the amplified portion of the CSDRS

By incorporating this guidance into the SRP, DC applicants have a better understanding of the staff's expectations, and the staff can reference the guidance in its evaluations and requests for additional information.

Technical Issue No. 10

Issues with SASSI Subtraction Method

1. Description of Issue

The System for Analysis of Soil-Structure Interaction (SASSI) program, which Lysmer et al. [1, 2] developed originally, is an effective soil-structure interaction (SSI) analysis tool based on the substructuring approach. Several commercial versions of SASSI have been used to perform seismic analyses in support of design certification (DC) and combined license (COL) applications.

For the case of embedded structures, two analytic approaches are used in SSI computations performed with SASSI. The first approach, referred to as the flexible volume or direct method (DM), is the more reliable but also the more computationally intensive method. The DM incorporates all nodes of the finite element mesh for the excavated below-grade zone of the embedded structure (termed the interaction nodes) in the solution. The second method, known as the subtraction method (SM), uses an approximate simplification that yields significant reductions in the computational effort. It reduces the number of the interaction nodes to only those on the boundary of the excavated zone and assumes that the remaining interior nodes do not need to be connected to the boundary nodes. However, recent SSI analyses performed for certain U.S. Department of Energy facilities identified limitations associated with the SM [3, 4, 5]. It was found that, if not implemented properly, the application of the SM to the SSI analysis of embedded structures may potentially result in erroneous and unconservative SSI responses when compared to the DM.

To identify limitations and mitigate potential errors associated with the SM, and to ensure that a conservative seismic analysis is performed, Revision 4 to SRP Section 3.7.2 included additional guidance on SSI analysis to provide criteria for reviewing SSI analysis of embedded structures performed using the SM.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The criteria for reviewing SSI analysis were presented in SRP Section 3.7.2 II.4.

2.1.1 The criteria in Item “B. Modeling of Supporting Soil” stated:

> “the effect of embedment of structure, groundwater effects, and the layering effect of soil should be accounted for. For the half-space modeling of the soil media, it is indicated that the lumped parameter (soil spring) method and the compliance function methods are acceptable provided that frequency variations and layering effects are incorporated.”

2.1.2 The criteria in Item “C. Input Ground Motion,” under the bullet entitled “Half Space or Substructure Solution Technique,” provided a brief description of the half space or substructure approach, and also state that “The procedures, modeling assumptions and analytical bases adopted for performing the half space or substructure analysis, including use of frequency-

independent soil spring parameters, and the spring and damping coefficients, will be reviewed on a case-by-case basis."

2.2 Why Technical Criteria Were Enhanced

The criteria in SRP Section 3.7.2 II.4 did not provide any specific guidance for reviewing the SSI analysis of embedded structures performed using the SM of SASSI (which falls under the "substructure approach" designation in the SRP). To identify limitations of this approach and mitigate potential errors associated with the SM, and to ensure that a conservative seismic analysis is performed, SRP Section 3.7.2 II.4 should be enhanced to provide guidance on how such analyses should to be performed to ensure that the SM leads to adequate results.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.7.2

3.1 The applicable portion of SRP Section 3.7.2 II.4.B and the included enhancements (highlighted in italics) are shown below.

> B. Modeling of Supporting Soil. The effect of embedment of structure, ground-water effects, *placement of compacted fills,* and the layering effect of soil should be accounted for. For ~~*the half-space*~~ modeling of the *support* soil media, the lumped parameter (soil spring) method and the compliance function methods are acceptable provided that frequency variations and layering effects are incorporated. For the method of modeling soil media with finite boundaries, all boundaries should be properly simulated and the use of types of boundaries should be justified and reviewed on a case-by-case basis. Finite element and finite difference methods are acceptable methods for discretization of a continuum. The properties used in the SSI analysis should be those that are consistent with soil strains developed in free-field site response analyses.
>
> For structures founded on materials having a shear wave velocity of 8,000 feet per second or higher, under the entire surface of the foundation, a fixed base assumption is acceptable.
>
> *In the SSI analysis of embedded structures using the substructure approach, the finite element discretization of the excavated soil volume should have a mesh size in both the horizontal and vertical directions that is appropriate for adequately transmitting seismic motions over the frequency range of interest. Both the horizontal and vertical mesh size for the excavated soil model should satisfy the wave length requirement regardless of whether the model is being analyzed for the horizontal or vertical excitation. The geometric regularity of the mesh (aspect ratio and size) is also an important characteristic of the mesh to ensure the adequacy of the computational capability.*
>
> *In the SSI analysis of surface-supported structures using the substructure approach, or portions of embedded structures that are surface-supported, where there is no associated excavated soil volume, the guidance in the preceding paragraph is applicable to the mesh at the base of the surface supported structures.*

3.2 The criteria for "Half Space or Substructure Solution Technique" in SRP Section 3.7.2 II.4 were enhanced by the following additional statements (highlighted in italics):

In the SSI analysis of embedded structures, some computer implementations of the substructure approach use two alternative methods to model the excavated soil volume:

(1) The direct method (DM), in which the foundation impedance is calculated for the free field at all nodes of the excavated soil volume that is discretized into finite elements. These nodes, termed "interaction nodes," connect the excavated soil volume and the free field soil system to ensure compatible motions. DM is also referred to as the flexible volume method (FVM) in frequency domain solutions.

(2) The subtraction method (SM), in which a simplification is made such that only the nodes on the outer boundary of the excavated soil volume are treated as interaction nodes. This simplification reduces the computational effort needed for solving large problems typically encountered in NPP applications. However, because the interior nodes are not treated as interaction nodes, the compatibility of displacements is no longer imposed at every interaction nodes in the excavated volume. This may lead to limitations in the application of the SM and potential errors in computed foundation compliance functions as well as transfer functions.

In light of the above discussion, the DM should be used to the extent practical to perform the SSI analysis of embedded structures. In cases that require the use of the SM, due to limitations of the DM in handling very large computational models, technical justifications should be provided to demonstrate the adequacy of the SSI analysis based on the SM. These technical justifications should include the following elements:

(1) An assessment of the excavated soil volume should be performed to identify its vibratory frequencies and mode shapes. These frequencies and mode shapes may be spurious in the SM solution, which can lead to unconservative or erroneous results. They can be identified as spikes in the transfer functions computed using the SM, which do not appear in the corresponding transfer functions computed using the DM.

(2) The limitations of the SM can be mitigated by constraining sufficient interior nodes (as interaction nodes) of the excavated soil volume. This approach is known as the modified subtraction method (MSM). The effect of these additional constraints is to shift the frequencies of the spurious vibration modes above the frequency range of interest to the SSI analysis.

(3) A converging trend in the MSM solution may be established by carefully examining the computed transfer functions. The additional interaction nodes should shift the frequencies of the spurious spikes in the transfer functions above the frequencies of interest to the SSI analysis.

(4) An evaluation should be performed to ensure that the frequency content of the ground motion input important to the SSI analysis lies within a range that is minimally affected by the spurious vibration modes of the constrained excavated soil volume.

Computer models of reduced size (e.g., quarter models) can also be used to obtain additional insight into the adequacy of an SSI analysis performed using the SM/MSM. In this case, direct comparisons between the SM/MSM and DM solutions are feasible and may provide valuable information that could be extrapolated to the full size model.

4. Technical Basis and/or Rationale

Given the possible incompatible motions between the excavated soil volume and the portion of the free field being replaced, the SM can potentially affect the SSI response in two aspects. First, the vibration modes of the excavated soil volume in the SM model may be spurious. For ground motion inputs with dominant frequencies higher than the fundamental frequency of the excavated soil volume, these spurious vibration modes are expected to be excited and participate in the dynamic response, which would likely have an impact on the computed SSI response.

Second, the computed SSI response could either be increased or reduced depending on whether the excavated soil volume moves in-phase or out-of-phase with the superstructure. This can be identified in the pattern of highly oscillatory behavior seen in typical SSI response computed using the SM as opposed to the DM.

The limitations of the SM can be alleviated by connecting additional nodes of the excavated soil volume to the part of the free field being replaced. This is accomplished in the so-called Modified SM (MSM); however, as more nodes are added to the MSM, the computational effort is also increased. It should be noted that the MSM can reduce but not eliminate the incompatibility issue associated with the SM. In summary, the issues associated with the SM are strongly dependent on (a) the characteristics of the ground motion input and (b) the particular soil-structure configurations being analyzed.

The rationale for the enhancements to SRP Section 3.7.2 II.4 is as follows:

- The enhancement to the criteria for “Modeling of Supporting Soil” is needed to ensure that the dynamic characteristics of the excavated soil volume are adequately represented in the SSI model. In the review of some past applications, the staff identified that if the finite element mesh in the horizontal dimension was much coarser than the vertical dimension, then this could lead to inadequate transmission of the SSI frequencies.

- The enhancement improves the criteria for “Half Space or Substructure Solution Technique” by providing additional guidance to implement the DM and SM in SSI analyses.

- Specific guidance is provided to ensure that the limitations of the SM are identified and potential errors associated with these limitations are mitigated. If the SM is used, an eigenvalue analysis of the excavated soil volume should be performed to identify its

natural frequencies and modes. The proposed criteria for mesh size of the excavated soil volume (SRP Section 3.7.2 II.4.B "Modeling of Supporting Soil") are also important to ensure that the computed frequencies and modes are reasonably accurate. Such an eigenvalue analysis will provide an indication of the frequencies at which the spurious vibration modes are likely to occur. For example, if it is found that the lowest natural frequency of the excavated soil volume is greater than the ZPA frequency of the ground motion inputs, then the incompatibility issue with the SM is not likely to affect the SSI response.

- Finally, an approach to properly implement the MSM is provided to shift the frequencies of the spurious vibration modes above the frequency range of interest, which should be determined on the basis of the frequency content of the ground input motion important to the SSI analysis.

5. References

[1] Lysmer, J., Tabatabaie, M., Tajirian, F., Vahdani, S., and Ostadan, F., "SASSI: A System for Analysis of Soil-Structure Interaction," Report No. UCB/GT/81-02, Geotechnical Engineering Division, Department of Civil Engineering, University of California, Berkeley, April 1981.

[2] Lysmer, J., Ostadan, F., J., Tabatabaie, M., Tajirian, F., and Vahdani, S., "SASSI2000 Theoretical Manual: A System for Analysis of Soil-Structure Interaction," Revision 1, November 1999.

[3] Mertz, G., Cuesta, I., Maham, A., and Costantino, M., "Seismic Response of Embedded Facilities Using the SASSI Subtraction Method," Report No. LA-UR-10-05302, Los Alamos National Laboratory, August 2010.

[4] Defense Nuclear Facilities Safety Board, Letter to the Deputy Secretary of Energy, April 8, 2011.

[5] US. Department of Energy Report on Technical and Software Quality Assurance Issues Involving the System for Analysis of Soil-Structure Interaction (Response to Defense Nuclear Facilities Safety Board Letter dated April 8, 2011), July 29, 2011.

Technical Issue No. 11

Guidance on Spent Fuel Pool Racks

1. Description of Issue

The regulatory bases for ensuring the structural integrity of spent fuel stored in fuel storage racks are wide ranging and cover requirements related to seismic design, criticality and dose consequence analysis. The "General Design Criteria for Nuclear Power Plants" (GDC) 2 requires that safety-related structures, systems, and components (SSCs) be designed to withstand the most severe natural phenomena including earthquakes without the loss of capability to perform their safety functions. Prior to Revision 4, Appendix D to Standard Review Plan (SRP) Section 3.8.4, "Guidance on Spent Fuel Pool Racks," provided the only guidance that directly addresses fuel assembly integrity in the fuel storage pools during a design basis seismic event. Appendix D stated, "It should be demonstrated that the consequent loads on the fuel assembly do not lead to damage of the fuel." However, Appendix D did not provide any detailed guidance to the definition of "damage" or guidance on how to perform the review.

Appendix D did not provide specific guidance for QA and periodic condition monitoring for spent fuel racks. RG 1.29 identifies spent fuel racks as seismic Category I. As such, Appendix B QA requirements and Maintenance Rule condition monitoring requirements should apply to the spent fuel racks.

Some of the guidance in Appendix D was provided specifically to address re-racking for operating plants to increase the storage capacity of existing spent fuel pools. New plant designs utilize similar high density free-standing spent fuel racks. Appendix D wording needed to be updated to be applicable to both re-racking and new plant designs.

2. Why SRP Section 3.8.4, Appendix D, Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

The statement "It should be demonstrated that the consequent loads on the fuel assembly do not lead to damage of the fuel" was the only guidance provided in Appendix D to SRP 3.8.4 related to the assessment of the structural integrity of the fuel assembly stored in the spent fuel racks.

Specific QA and periodic condition monitoring guidance were not provided for spent fuel racks.

Some of the guidance in Appendix D was directed toward re-racking for operating plants.

2.2 Why Technical Criteria Were Enhanced

Appendix D did not provide sufficient guidance on the definition of "damage" or the detailed process for performing the review. Therefore, this issue was not being addressed by applicants and reviewers on a consistent basis.

The staff's expectations for QA and condition monitoring of fuel racks should be clearly documented to eliminate confusion in the future.

Updating Appendix D was needed to ensure that the guidance is equally applicable to both re-racking and new plant designs.

3. Enhancements Incorporated in Revision 4 to SRP Section 3.8.4, Appendix D

Appendix D and the included enhancements (highlighted in italics) are shown below.

I. INTRODUCTION

Regulatory Guide 1.29, "Seismic Design Classification" classifies spent fuel pool racks as ~~S~~seismic Category I structures. *Spent fuel pool racks should be treated as safety-related components for determining Quality Assurance requirements (10 CFR Part 50, Appendix B) and periodic condition monitoring requirements (10 CFR 50.65 "Maintenance Rule").*

This appendix describes ~~*minimum requirements and*~~ *acceptance* criteria for review of spent fuel pool racks and the associated structures, which would meet the acceptance criteria specified in Subsection II of this SRP section. *A secondary review responsibility would include the review of the material limits associated with the fuel assembly in the fuel storage racks and the effect of rack deformations on the coolability of the fuel assembly.*

1. Description of the Spent Fuel Pool and Racks

The applicant should provide descriptive information including plans and sections showing the spent fuel pool in relation to other plant structures in order to define the primary structural aspects and elements relied on to perform the safety-related functions of the spent fuel pool, pool liner, and racks. The main safety function of the spent fuel pool, including the liner, and the racks is to maintain the spent fuel assemblies in a safe configuration through all environmental and abnormal loadings (such as earthquakes) and impacts from drop of a spent fuel cask, drop of a spent fuel assembly, or drop of any other heavy object during routine spent fuel handling.

The following indicates the major structural elements reviewed and the extent of the descriptive information required:

A. Support of the Spent Fuel Racks - The applicant should describe the general arrangements and principal features of the horizontal and vertical supports to the spent fuel racks and indicate the methods of transferring the loads on the racks to the fuel pool wall and the foundation slab. All gaps (clearance or expansion allowance) and sliding contacts should be indicated. The discussion should cover the extent of interfacing between the ~~*new*~~ rack system and the ~~*old*~~ fuel pool walls and base slab (i.e., interface loads, response spectra, etc.).

If connections of the racks are made to the base and to the side walls of the pool such that the pool liner may be perforated, the applicant should

indicate the provisions for avoiding leakage of radioactive water from the pool.

B. Fuel Handling - The organization responsible for postulation of a drop accident and quantification of the drop parameters reviews the criteria related to fuel handling. The findings of the review are evaluated for the purpose of integrity of the racks and the fuel pool, including the fuel pool liner, in view of a postulated fuel-handling accident. The applicant should provide sketches and sufficient details of the fuel-handling system to facilitate this review.

2. Applicable Codes, Standards, and Specifications

Construction materials should conform to ~~*American Society of Mechanical Engineers, (*~~ASME~~*)*~~, Boiler and Pressure Vessel Code, (Code), Section III, Division 1, Subsection NF. All materials should be selected to be compatible with the fuel pool environment to minimize corrosion and galvanic effects.

Design, fabrication, and installation of spent fuel racks of stainless steel material may be performed based on ASME Code, Section III, Division 1, Subsection NF requirements for Class 3 component supports."

3. Seismic and Impact Loads

~~*For plants where dynamic input data such as floor response spectra or ground response spectra are not available, necessary dynamic analyses may be performed using the criteria described in SRP Section 3.7. The ground response spectra and damping values should correspond to RG 1.60 and 1.61, respectively. For plants where dynamic data are available (e.g., ground response spectra for a fuel pool supported by the ground, floor response spectra for fuel pools supported on soil where soil-structure interaction was considered in the pool design, or a floor response spectra for a fuel pool supported by the reactor building), the design and analysis of the new rack system may be performed by using either the existing input parameters including the old damping values or new parameters in accordance with RG 1.60 and 1.61. The use of existing input with new damping values in RG 1.61 is not acceptable.*~~

~~*Seismic excitation along three orthogonal directions should be imposed simultaneously for the design of the new rack system.*~~

~~*The peak response from each direction should be combined by square root of the sum of the squares in accordance with RG 1.92. If response spectra are available for a vertical and horizontal direction only, the same horizontal response spectra may be applied along the other horizontal direction.*~~

~~*Submergence in water may be taken into account. The effects of submergence are evaluated on case-by-case basis.*~~

For new plants, dynamic input data such as floor response spectra or ground response spectra are developed using the criteria described in SRP Section 3.7.

For operating plants where dynamic data are available (e.g., ground response spectra for a fuel pool supported by the ground, floor response spectra for fuel pools supported on soil where SSI was considered in the pool design, or a floor response spectra for a fuel pool supported by the reactor building), the design and analysis of a replacement rack system may be performed using the existing plant seismic design basis. As an alternate, the seismic analysis of spent fuel pool racks may be conducted using an updated plant seismic design basis developed using the criteria described in SRP Section 3.7. .

For free-standing spent fuel pool racks, which are potentially subject to sliding, uplift, and impact between racks and with the pool walls, time-varying seismic excitation along three orthogonal directions (2 horizontal and vertical) should be imposed simultaneously.

For fully supported spent fuel pool racks, the response spectra analysis method is acceptable. The peak response from each direction is combined in accordance with RG 1.92. If response spectra are available for a vertical and horizontal direction only, the same horizontal response spectra may be applied along the other horizontal direction.

The effects of submergence in water need to be addressed in the spent fuel rack structural analysis. The effects of submergence are evaluated by the staff on case-by-case basis.

Because of gaps between fuel assemblies and the walls of the guide tubes, additional loads will be generated by the impact of fuel assemblies during a postulated seismic excitation. Additional loads resulting from this impact effect may be determined by estimating the kinetic energy of the fuel assembly. The maximum velocity of the fuel assembly may be estimated to be the spectral velocity associated with the natural frequency of the submerged fuel assembly. Loads thus generated should be considered for local as well as overall effects on the walls of the rack and the supporting framework. It should be demonstrated that the consequent loads on the fuel assembly do not lead to damage of the fuel. *Damage of the fuel refers to structural elements of a fuel assembly (including the fuel rod cladding) which are stressed beyond the material allowable limits (established in terms of either strength or strain limits) such that the fuel rods are no longer able to provide confinement for contained radioactive fission materials.*

An evaluation considering pertinent failure modes (such as buckling, etc.) should be performed to demonstrate that when subject to the consequent loads resulting from the various load combinations described in Table 1, the structural elements of the fuel assembly will not exceed appropriate material allowable limits. Irradiation embrittlement effects, as well as pool temperature effects on the material properties, should be adequately accounted for in establishing the material allowable limits. Evaluations based on testing results to demonstrate structural integrity of the fuel assembly may also be acceptable, provided that the testing configurations and parameters are consistent with those for the fuel assembly being evaluated. To this end, the testing results are evaluated on a case-by-case basis in determining the structural integrity of the fuel assembly.

The evaluation should also confirm that any fuel assembly deformation resulting from the applicable load combinations does not degrade the coolable configuration of the fuel assembly to an unacceptable level.

Loads generated from other postulated impact events may be acceptable, if the total mass of the impacting missile, the maximum velocity at the time of impact, and the ductility ratio of the target material used to absorb the kinetic energy are described.

4. Loads and Load Combinations

~~*Any change in the temperature distribution resulting from the proposed modification should be identified. Information pertaining to the applicable design loads and their various combinations should be provided and indicate the thermal load resulting from the maximum temperature distribution through the pool walls and base slab*~~

Information pertaining to the applicable design loads and their various combinations should be provided. If applicable, any change in the temperature distribution resulting from a proposed modification to an existing spent fuel rack configuration should be identified. The temperature gradient across the rack structure that results from the differential heating effect between a full and an empty cell should be indicated and incorporated in the design of the rack structure. Maximum uplift forces available from the crane should be indicated and include consideration of these forces in the design of the racks and the analysis of the existing pool floor, if applicable.

The fuel pool racks and the fuel pool structure, including the pool slab and fuel pool liner, should be evaluated for accident load combinations which include the impact of the spent fuel cask, the heaviest postulated load drop, and/or accidental drop of the fuel assembly from the maximum height.

The review will evaluate the acceptable limits (strain or stress limits) on a case-by-case basis, but in general, the applicant is required to demonstrate that the functional capability and/or the structural integrity of each component is maintained.

The specific loads and load combinations are acceptable if they conform to the applicable portions of ~~*this*~~ SRP *3.8.4*, Subsection II.3, and Table 1 provided in this Appendix.

5. Design and Analysis Procedures

American National Standards Institute, N210-76, “Requirements for Light Water Reactor Spent Fuel Storage Facilities at Nuclear Power Plants, Design,” provides general information regarding design of spent fuel pool racks.

Details of the mathematical model, including a description of how the important parameters are obtained, should be provided. The details should include the methods used to incorporate any gaps between the support systems and gaps between the fuel bundles and the guide tubes; the methods used to lump the

masses of the fuel bundles and the guide tubes; the methods used to account for the effect of sloshing water on the pool walls; and the effect of submergence on the mass, the mass distribution, and the effective damping of the fuel bundle and the fuel racks. Design and analysis procedures in accordance with ~~*this*~~ SRP *3.8.4*, Subsection II, are acceptable. The effect of gaps, sloshing water, and increase of effective mass and damping resulting from submergence in water should be quantified.

If the spent fuel racks are designed to be free standing (i.e., without connections to the pool walls/floor), then their response involves a complex combination of motions that includes sliding, rocking, and twisting and involves impacts between the fuel assemblies and the fuel cell walls, rack-to-rack, and rack-to-wall. In view of this, the seismic analysis of these fuel racks is typically performed using nonlinear dynamic time history analysis methods. *For nonlinear seismic analysis of the racks, multiple time histories should be performed in accordance with the criteria for nonlinear analysis described in SRP 3.7.1, unless otherwise justified. For the free standing rack analyses, the entire range of the coefficient of friction for the rack material in water should be considered between the rack legs and the pool floor as well as the other contact surfaces (e.g., rack-to-rack impacts, rack-to-wall impacts).* NUREG/CR-5912 provides further guidance on the design and analysis of free-standing fuel racks.

The seismic input motion to the racks should consider the spectra at the rack base and the wall of the spent fuel pool that typically is obtained from the overall seismic building SSI analysis. It is acceptable to envelop the seismic motion at these two locations for the input loading to the racks. This approach is also applicable to free standing racks because seismic inertial loading can be transferred from the pool walls through the water in the pool to the racks. Alternative methods that may be used should be provided and reviewed on a case-by-case basis.

When pool walls are used to provide lateral restraint at higher elevations, the applicant should provide a determination of the flexibility of the pool walls and the capability of the walls to sustain such loads. If the pool walls are flexible (having a fundamental frequency less than 33 hertz), the floor response spectra corresponding to the lateral restraint point at the higher elevation are likely to be greater than those at the base of the pool. To use the response spectrum approach in such a case, the following two separate analyses should be performed:

A. A spectrum analysis of the rack system using response spectra corresponding to the highest support elevation provided that there is not significant peak frequency shift between the response spectra at the lower and higher elevations

B. A static analysis of the rack system by subjecting it to the maximum relative support displacement

The resulting stresses from the two analyses above should be combined by the absolute sum method.

To determine the flexibility of the pool wall, it is acceptable for the applicant to use equivalent mass and stiffness properties obtained from calculations similar to those described in "Introduction to Structural Dynamics," McGraw-Hill Book Co., New York, 1964, by Biggs, John M. Should the fundamental frequency of the pool wall model be higher than or equal to 33 hertz, it may be assumed that the response of the pool wall and the corresponding lateral support to the rack system are identical to those of the base slab, for which appropriate floor response spectra or ground response spectra may already exist.

6. Structural Acceptance Criteria

Table 1 of this Appendix provides the structural acceptance criteria, in accordance with ASME Code, Section III, Division 1, Subsection NF. When considering compression loads, Subsection NF, Paragraph 3300, specifies additional criteria that must be satisfied to preclude buckling.

For impact loading, the ductility ratios used to absorb kinetic energy in the tensile, flexural, compressive, and shearing modes should be quantified. In the consideration of the effects of seismic loads, factors of safety against gross sliding and overturning of racks and rack modules under all probable service conditions should be in accordance with SRP Section 3.8.5, Subsection II.5. This position on factors of safety against sliding and tilting need not be met provided that the applicant meets any one of the following conditions:

A. Detailed nonlinear dynamic analyses show that the amplitudes of sliding motion are minimal and impact between adjacent rack modules or between a rack module and the pool walls is prevented provided that the factors of safety against tilting are within the allowable values provided in SRP Section 3.8.5, Subsection II.5.

B. Any sliding and tilting motion will be contained within suitable geometric constraints such as thermal clearances, and any impact resulting from the clearances is incorporated.

The fuel pool structure should be designed for the ~~*increased loads that stem from the new and/or expanded high-density racks*~~ *loads imposed by the racks*. The fuel pool liner leak-tight integrity should be maintained, or the functional capability of the fuel pool should be demonstrated.

7. Materials, Quality Control, and Special Construction Techniques

The applicant should describe materials, quality control procedures, and any special construction techniques; the sequence of installation of the fuel racks; and the precautions to be taken to prevent damage to the stored fuel during ~~*the construction phase*~~ *re-racking at an operating plant*.

If connections between the rack and the pool liner are made by welding, the welder, as well as the welding procedure for the welding assembly, should be qualified in accordance with the applicable code.

For spent fuel pool racks fabricated from aluminum, *ASCE~~American Society of Civil Engineers~~*, Suggested Specification for Structures of Aluminum Alloys 6061-T6 and 6067-T6 and “Specification for Aluminum Structures” (issued by The Aluminum Association) contain the guidance regarding material properties.

Table 1

Load Combination	Acceptance Limit
D + L D+L+T_o D+L+T_o+E	*ASME Code Section III, Subsection NF* Level A service limits *for Class 3*
D+L+T_a+E D+L+T_o+P_f	*ASME Code Section III, Subsection NF* Level B service limits *for Class 3*
D+L+T_a+E'	*ASME Code Section III, Subsection NF* Level D service limits *for Class 3*
D+L+F_d	The functional capability of the fuel racks should be demonstrated
Limit Analysis	
Load Combination	Acceptance Limit
1.7 (D + L) *1.7* ~~*1.3*~~(D + L + T_o) ~~*1.7 (D + L+ E)*~~ *1.7* ~~*1.3*~~ (D + L + E + T_o) *1.7* ~~*1.3*~~ (D + L + E + T_a) *1.7* ~~*1.3*~~ *(*D + L + T_o + P_f) 1.1 (D + L + T_a+ E')	~~*Appendix XVII, Article 4000 of ASME Code, Section III*~~ *ASME Code Section III, Subsection NF, paragraph 3340*

Notes:

1. The abbreviations in the table above are those used in Subsection II.3 of ~~*this*~~ SRP *3.8.4* where each term is defined, except for T_a, *F_d, and P_f. T_a* ~~*which*~~ is defined here as the highest temperature associated with the postulated abnormal design conditions. *F_d is the force caused by the accidental drop of the heaviest load from the maximum possible height. P_f is the upward force on the racks caused by a postulated stuck fuel assembly.*

2. Deformation limits specified by the design specification limits should be satisfied and such deformation limits should preclude damage to the fuel assemblies.

3. ~~*The provisions of ASME Code, Section III, Division 1, Subsection NF 3231.1 shall be amended by the requirements of paragraphs c. 2, 3, and 4 of RG 1.124.*~~ *The provisions of ASME Code, Section III, Division 1, Subsection NF were amended consistent with regulatory positions contained in RG 1.124 "Service Limits and Loading Combinations for Class 1 Linear-Type Component Supports."*

~~4. *Fd is the force caused by the accidental drop of the heaviest load from the maximum possible height, and Pf is the upward force on the racks caused by a postulated struck fuel assembly*.~~

II. REFERENCES

1. *ASCE Suggested Specification for Structures of Aluminum Alloys 6061-T6 and 6067-T6.~~Regulatory Guide1.29, "Seismic Design Classification."~~*

~~2. *Regulatory Guide1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants."*~~

~~3. *Regulatory Guide1.61, "Damping Values for Seismic Design of Nuclear Power Plants."*~~

~~4. *Regulatory Guide1.76, "Design Basis Tornado for Nuclear Power Plants."*~~

~~5.~~2. *ASME Boiler and Pressure Vessel Code, Section III, Division 1.~~Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis."~~*

~~6.~~3. *ANSI, N210-76, "Requirements for Light Water Reactor Spent Fuel Storage Facilities at Nuclear Power Plants, Design."~~Regulatory Guide1.124, "Service Limits and Loading Combinations for Class 1 Linear-Type Component Supports."~~*

~~7.~~4. *Biggs, John M., "Introduction to Structural Dynamics," McGraw-Hill Book Co., New York, 1964.~~American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Division 1.~~*

~~8.~~5. *RG 1.29, "Seismic Design Classification."~~American National Standards Institute, N210-76, "Requirements for Light Water Reactor Spent Fuel Storage Facilities at Nuclear Power Plants, Design."~~*

~~9.~~6. *RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis."~~American Society of Civil Engineers, Suggested Specification for Structures of Aluminum Alloys 6061-T6 and 6067-T6.~~*

~~10.~~7. *RG 1.124, "Service Limits and Loading Combinations for Class 1 Linear-Type Component Supports."~~The Aluminum Association, Specification for Aluminum Structures.~~*

~~11.~~8. *NUREG/CR-5912, "Review of the Technical Basis and Verification of Current Analysis Methods Used to Predict Seismic Response of Spent Fuel Storage Racks," October 1992.~~Biggs, John M., Introduction to Structural Dynamics, McGraw-Hill Book Co., New York, 1964.~~*

~~12.~~9. *The Aluminum Association, "Specification for Aluminum Structures."~~NUREG/CR-5912, "Review of the Technical Basis and Verification of Current Analysis Methods Used to Predict Seismic Response of Spent Fuel Storage Racks," October 1992.~~*

4. Technical Basis and/or Rationale

In accordance with Appendix D to SRP Section 3.8.4, the staff has requested information from DC and COL applicants regarding the capability of spent fuel assemblies to withstand the loads imparted on them during a safe shutdown earthquake (SSE). As indicated in Appendix D to SRP Section 3.8.4, the acceptance criteria related to the design of spent fuel racks requires, in part, that the structural integrity of the spent fuel contained within the racks be maintained during a seismic event. The spent fuel racks are designed as seismic Category I to protect the contained spent fuel assemblies. In addition, it is necessary to evaluate the spent fuel assemblies to ensure that when subject to ASME Service Level D loads, the design material allowable values, when adjusted to the spent fuel pool temperature, will not be exceeded for the spent fuel assembly components. The technical approaches taken by the applicants to demonstrate fuel assembly structural integrity have been diverse, requiring considerable staff effort before a conclusion of structural adequacy could be reached. To address this, the staff formed the Spent Fuel Working Group (SFWG) [1].

Based on its assessment of recent applicant submittals, the SFWG has recommended (1) the addition of secondary review responsibilities assigned to the organization responsible for the review of the fuel system design; and (2) an addendum to Appendix D of SRP Section 3.8.4. The secondary review responsibilities would include the review of the material limits associated with the fuel assembly in the fuel storage racks and the effect of rack deformations on the coolability of the fuel assembly. The addendum to Appendix D would address the apparent deficiency related to the lack of specific guidance related to establishing an acceptable value for the structural capacity of spent fuel cladding. The SFWG arrived at this conclusion based on the fact that it is apparent that licensees and applicants are able to adequately estimate the loading imparted on a spent fuel bundle. However, the challenge lies in determining whether the cladding can withstand these imparted loads. This challenge is exacerbated by the fact that there is no design code applicable to the structural design of spent fuel rods.

In addressing the inconsistency of spent fuel structural integrity reviews, the addendum to Appendix D should afford a greater degree of guidance to reviewers that will facilitate the deliberate and accurate assessment of spent fuel structural integrity.

Spent fuel pool racks are seismic Category I, and Appendix D states “The main safety function of the spent fuel pool, including the liner, and the racks is to maintain the spent fuel assemblies in a safe configuration….” It has come to the attention of staff reviewers that at least some licensees and applicants consider spent fuel pool racks to be non-safety related, do not invoke the Appendix B quality assurance requirements for spent fuel pool racks, and exclude spent fuel pool racks from Maintenance Rule Condition Monitoring programs.

The addition to the first paragraph of Appendix D should clarify the staff’s position.

Appendix D, before the enhancement, provided the staff’s guidance on spent fuel pool re-racking at operating plants, in order to provide significantly increased spent fuel storage. This was necessitated by the lack of available offsite storage facilities for spent nuclear fuel. Consequently, the revised Appendix D provides update to be more generically applicable because new reactors are being designed and constructed.

A change was made in Table 1 of Appendix D, for the acceptance limit under the heading “Limit Analysis.” Since ASME Code Section III, Appendix XVII was deleted, the acceptance limit was replaced with ASME Code Section III, Subsection NF, paragraph 3340, which provides acceptance limits for performing a limit analysis.

5. References

[1] Working Group Recommendations on Spent Fuel Structural Assessment[1]

[1]Note: The spent fuel working group (SFWG) prepared this report. The group includes staff from different offices of the Nuclear Regulatory Commission: Office of New Reactors (NRO) (Jim Xu – Team lead, Christopher VanWert, Brian Thomas), Office of Nuclear Research Reactor (NRR) (William Jessup), Office of Nuclear Material Safety and Safeguards (NMSS) (Bhasker Tripathi), and Office of Nuclear Regulatory Research (RES) (Jose Pires).

Technical Issue No. 12

Guidance on Minimum Power Spectral Density for NUREG/CR-6728 Based Design Spectra or Other Spectra

1. Description of Issue

When an artificial ground motion time history is used in the design of seismic Category I structures, systems, and components (SSCs), the time history should envelop the design response spectra and also contain adequate power typically measured by the power spectral density (PSD). For PSD check, target PSDs can be developed compatible to the design response spectra. When RG 1.60 response spectra are used as design response spectra, Appendix A to SRP Section 3.7.1 provided the target PSD compatible with RG 1.60 design spectra. For design response spectral shapes that are different from RG 1.60 response spectra, the criteria provided in Appendix B to SRP Section 3.7.1 may be used. However, the guidance in Appendix B to SRP 3.7.1, Revision 3 (2007), was intended to be used for design response spectra compatible with spectrum shapes consistent with those in NUREG/CR-6278. Also, the PSDs tabulated in Appendix B were based on the bin average PSDs from NUREG/CR-6728 which may not be compatible with either the bin average response spectra or the NUREG/CR-6278 design response spectra.

2. Why SRP Section 3.7.1, Appendix B, Should Be Revised

2.1 Technical Criteria Prior to SRP Revision 4

Appendix B to SRP Section 3.7.1, Revision 3 (March 2007), indicated that for design response spectra other than RG 1.60 response spectra, a compatible target PSD should be generated. For generation of target PSD in such cases, the guidelines and procedures provided in Appendix B to SRP section 3.7.1 could be used.

SRP 3.7.1, Appendix B provided the minimum PSD levels for response spectrum consistent with the magnitude and distance bin shapes in NUREG/CR-6728. The appendix indicated that the average one-sided PSD of the artificial time history, corresponding to the design response spectra, should exceed 80 percent of the appropriate magnitude and distance bin target PSD as shown in Tables 1 through 4 of the appendix.

2.2 Why Technical Criteria Were Enhanced

Typically, design response spectra used for seismic analysis of plant SSCs are not compatible with the magnitude and distance bin shapes in NUREG/CR-6728. This would prevent the use of SRP 3.7.1, Appendix B in these instances. In addition, the tabulated target PSDs presented in Appendix B to SRP 3.7.1, Revision 3, which were based on the bin average PSDs, are generally not compatible with either the bin average response spectra or the NUREG/CR-6278 design response spectra. Therefore, improvement to the procedure for developing target PSDs using the magnitude and distance bins in NUREG/CR-6728 should be made to address these items and Appendix B to SRP Section 3.7.1 should be revised accordingly.

3. Enhancements Incorporated in Revision 4 to Revision to SRP Section 3.7.1

3.1 Revision to SRP Section 3.7.1 II.1.B

The applicable portions of SRP Section 3.7.1 II.1.B - Design Time Histories, were enhanced to reflect the revision being made in SRP 3.7.1, Appendix B. The incorporated enhancements (highlighted in italics) are shown below:

Option 1: Single Set of Time Histories. To be considered acceptable, the …

i. Approach 1. For Approach 1, the spectrum

When RG 1.60 response spectra are used as design response spectra, the criteria for a compatible target PSD are contained in Appendix A to this SRP section. Target PSD functions other than those given in Appendix A can be used if justified. For design response spectra other than RG 1.60 response spectra, a compatible target PSD should be generated. For generation of target PSD in such cases *(i.e., spectra based on NUREG/CR-6728 or other spectra)*, the guidelines and procedures provided in Appendix B to this SRP section can be used. *These guidelines and procedures are consistent with the approach described in NUREG/CR-5347, "Recommendations for Resolution of Public Comments, Seismic Design Criteria," dated June 1989. Alternative methods for developing*~~*Procedures used to generate the*~~ target PSD *can*~~*will*~~ be *used and are* reviewed on a case-by-case basis.

Regardless of the approach used, the development of the target PSD and the range of frequency for the PSD check are reviewed on a case-by-case basis. The PSD criteria are included as secondary check to prevent potential deficiency of power over the frequency range of interest. It should be noted that the ground motion is still primarily defined by the design response spectrum. The use of PSD criteria alone can yield time histories that may not envelop the design response spectrum.

3.2 Revision to SRP Section 3.7.1, Appendix B

The guidance and procedures presented in SRP Section 3.7.1, Appendix B were revised to ensure that the target PSDs are compatible with the response spectra based on NUREG/CR-6728 and also to enable its use when the design response spectra are not compatible with NUREG/CR-6728 spectra. In view of the substantial revision made to SRP 3.7.1, Appendix B, the entire revised SRP 3.7.1, Appendix B is presented below (without highlighted italics), and the Reference section of SRP 3.7.1, Appendix B is reproduced as Section 5 in this Technical Issue.

APPENDIX B TO SRP SECTION 3.7.1

GUIDANCE ON MINIMUM POWER SPECTRAL DENSITY FOR NUREG/CR-6728 BASED DESIGN SPECTRA OR OTHER SPECTRA

While Appendix A to SRP Section 3.7.1 provides guidance on the minimum power spectral density (PSD) for a RG 1.60 horizontal response spectrum, this appendix presents guidance on developing minimum PSD for a response spectrum with a shape consistent with the magnitude and distance bins in NUREG/CR-6728, as well as guidance for other spectral shapes.

The one-sided PSD for an acceleration time history $a(t)$, is related to its Fourier amplitude $|F(\omega)|$ by the following equation:

$$S_o(\omega) = \frac{2|F(\omega)|^2}{2\pi T_D} \tag{1}$$

In which T_D is the strong motion duration over which $F(\omega)$ is evaluated; ω represents the circular frequency (in rad/s) and is defined as: $\omega = 2\pi f$, where f is the natural frequency (in Hz). The duration T_D represents the duration of near maximum and nearly stationary power of an acceleration time history record. Additional guidance on estimating T_D for artificial time history or actual earthquake time history is provided in Appendix B of NUREG/CR-5347.

Fourier amplitude $|F(\omega)|$ in Equation (1) (also for Appendix A) is defined at each circular frequency ω_n as:

$$|F(\omega_n)| = \Delta t \left| \sum_{j=0}^{N-1} a(t_j) e^{-2\pi i \left(\frac{nj}{N}\right)} \right| \tag{2}$$

where, $a(t_j)$ is the strong motion portion of the acceleration time history (after proper tapering at both ends), $t_j = j\,\Delta t,\ j = 0, 1,\ \ldots N-1$, and $n = 0, 1,\ \ldots N/2$. For $N/2 < n \leq N-1$, ω_n represents the negative frequencies and does not appear in the one-sided PSD calculation.

At any frequency f (where, $f = \omega/2\pi$), the average one-sided PSD is computed over a frequency band width of ±20%, centered on the frequency f (e.g., 4 Hz to 6 Hz band width for f = 5 Hz).

In NUREG/CR-6728, the horizontal design response spectra for rock site conditions appropriate for the Central and Eastern United States (CEUS) are defined as a function of the earthquake moment magnitude M and the fault distance R (in km) by the following equation:

$$\ln\left(\frac{SA(f)}{PGA}\right) = \frac{C_1}{\cosh\left(C_2 f^{C_3}\right)} + C_4 \left(\frac{\exp(C_5 f)}{f^{C_6}} + \frac{C_7 \exp(C_8 f)}{f^{C_9}}\right)^{1/2} \tag{3}$$

where SA and PGA represent the spectral amplitude and peak ground acceleration, respectively, and the parameters C_1 through C_9 are defined as (for the single corner frequency model):

$$C_1 = 0.88657$$

$$C_2 = \exp(-10.411)$$

$$C_3 = 2.5099$$

$$C_4 = -7.4408 + M[1.5220 - 0.088588M + 0.0073069\ln(0.12639R + 1)]$$

$$C_5 = -0.34965$$

$$c_6 = -0.31162 + 0.0019646R$$

$$c_7 = 3.7841$$

$$C_8 = -0.89019$$

$$c_9 = 0.39806 + 0.058832M$$

Similarly, the horizontal design spectra for rock site conditions appropriate for the Western Unites States (WUS) are defined as a function of the earthquake moment magnitude M and the fault distance R (in km) by the following equation:

$$\ln\left(\frac{SA(f)}{PGA}\right) = \frac{C_1}{\cosh(C_2 f^{C_3})} + C_4\left(\frac{\exp(C_5 f)}{f^{C_6}}\right) \tag{4}$$

where the parameters C_1 through C_6 are defined as:

$$C_1 = 1.8197$$

$$C_2 = 0.30163$$

$$C_3 = 0.47498 + 0.034356M + 0.0057204\ln(R + 1)$$

$$C_4 = -12.650 + M[2.4796 - 0.14732M + 0.034605\ln(0.040762R + 1)]$$

$$C_5 = -0.25746$$

$$c_6 = 0.29784 + 0.010723M - 0.0000133R$$

For each magnitude-distance bin in the NUREG/CR-6728 database, a bin representative design response spectrum is defined in this appendix by Equations (3) or (4) with the moment magnitude M and fault distance R equal to the midpoint bin values. For example, for bin M6-7 D010-050, the midpoint bin values are M=6.5 and R=30 km.

For a NUREG/CR-6728 bin representative horizontal response spectrum as defined above and anchored to 1.0 g, the following guidance on developing minimum PSD can be used in conjunction with the target PSD tabulated in Tables 1 and 2 of Appendix B. For other peak accelerations, the target PSD in Tables 1 and 2 of Appendix B should be scaled by the square of the peak acceleration. For response spectrum shapes different from the NUREG/CR-6728 bin representative shapes (and different from RG 1.60 horizontal design spectra), the target

PSD may be developed based on the guidance in Section B.2 of this appendix and are reviewed and accepted by the NRC staff on a case-by-case basis.

The averaged one-sided PSD defined by Equation (1) should exceed 70% of the target PSD in Tables 1 and 2 or developed using the guidelines described in this appendix. When the target PSD in Tables 1 and 2 are used, linear interpolation of the data should be performed in the log-log scale to determine PSD values for frequencies other than the tabulated frequencies. The minimum check is set at 70% of the target PSD so as to be sufficiently high to prevent a deficiency of power over any broad frequency band, but not so high that it introduces additional conservatism over that already embodied in the specified design response spectrum. The use of 70% in Appendix B is comparable to the effect of the 80% criterion in Appendix A, because the response spectra compatible with the target PSD in Tables 1 and 2 of Appendix B match more closely to the design response spectra (Equations (3) and (4)) than do the response spectra compatible with the target PSD in Appendix A, which were generally lower than the design response spectra based on RG 1.60.

In general, power below 0.3 Hz has no influence on stiff nuclear plant structures, so that checks below 0.3 Hz are unnecessary. The upper bound frequency (cutoff frequency) should be consistent with the design response spectrum.

Table 1 – Horizontal Target PSD for CEUS Rock Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12		0.001	0.66	1.14	0.95	1.93	4.77	9.26	7.68	18.95
0.15	0.014	0.021	1.01	1.65	1.73	2.73	7.39	12.34	12.94	24.05
0.19	0.077	0.10	1.61	2.49	3.48	4.01	10.92	15.88	20.41	31.26
0.24	0.23	0.30	2.83	4.34	4.95	5.97	15.76	20.80	31.58	39.95
0.30	0.48	0.58	4.80	6.58	8.42	8.76	19.55	27.10	33.57	41.39
0.37	0.86	1.04	7.01	8.03	11.57	11.57	27.83	34.55	47.16	54.62
0.46	1.48	1.67	9.86	12.14	15.58	14.74	34.72	47.01	56.42	63.05
0.58	2.73	2.88	14.14	15.56	19.05	20.05	46.53	55.40	62.90	69.53
0.72	3.46	4.18	18.84	20.60	24.20	25.99	48.63	61.05	68.64	81.44
0.90	5.65	6.14	22.49	25.98	27.78	31.93	56.20	68.92	74.30	87.68
1.13	8.25	8.78	28.77	31.14	36.31	38.49	59.64	71.64	81.34	85.52
1.41	10.34	13.10	31.40	35.99	40.41	43.09	62.46	75.11	76.48	84.61
1.76	13.87	15.10	34.29	38.25	43.02	50.25	62.29	71.23	78.79	89.74
2.20	15.35	18.68	34.05	41.26	42.64	50.49	58.66	67.58	75.49	82.68
2.75	18.36	20.72	35.25	40.12	45.65	51.56	55.01	60.20	68.46	77.70
3.44	19.76	23.38	33.45	38.60	43.92	49.74	50.80	55.17	64.06	71.89
4.29	21.36	24.50	31.64	36.26	40.24	47.31	44.20	49.06	54.53	62.36
5.37	22.70	26.64	33.04	35.67	39.27	45.30	38.67	43.75	50.93	55.75
6.71	23.91	27.31	29.46	33.57	37.70	42.29	36.44	42.00	43.49	49.83
8.39	25.21	27.57	29.42	31.91	35.50	37.93	35.37	36.33	38.28	43.08
10.49	25.74	28.15	28.81	30.87	32.70	35.26	31.31	33.74	35.08	37.34
13.11	25.59	26.75	27.31	28.04	29.55	29.96	28.85	29.18	29.06	30.57
16.38	23.82	24.73	25.18	24.77	25.65	25.70	24.67	25.02	25.59	26.81
20.48	21.15	21.26	21.10	20.95	21.81	20.97	21.30	21.16	20.88	20.59
25.60	17.31	17.57	17.73	17.50	17.52	16.82	17.62	17.13	16.71	16.50
32.00	14.55	13.97	14.06	14.17	13.59	13.72	13.48	13.10	13.37	13.20
40.00	11.32	11.43	11.36	10.71	10.60	10.83	10.47	10.82	10.23	10.57
50.00	8.13	8.05	7.98	7.98	7.54	7.64	7.47	7.34	7.43	7.08
62.50	4.59	4.61	4.50	4.06	4.35	3.99	4.10	4.00	4.01	3.77
78.13	1.46	1.39	1.28	1.34	1.26	1.11	0.98	1.04	0.92	0.83
97.66	0.25	0.28	0.22	0.069	0.14	0.13	0.14	0.088	0.090	0.048

Table 2 - Horizontal Target PSD for Western US Rock Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.011	0.036	1.62	2.60	7.03	13.91	10.39	27.21	41.16	84.42
0.15	0.091	0.29	2.98	5.32	10.26	18.82	15.42	34.46	57.15	113.14
0.19	0.52	0.97	5.02	9.12	15.13	27.98	24.62	42.32	80.09	142.53
0.24	1.26	2.08	10.32	15.13	24.05	40.44	36.80	65.03	108.97	178.48
0.30	2.73	4.03	18.05	21.31	36.89	61.09	51.89	88.08	142.38	202.52
0.37	4.45	6.67	24.34	35.26	55.35	73.73	66.45	108.84	172.28	238.08
0.46	7.30	10.17	36.53	49.28	69.60	90.03	88.85	130.01	190.94	269.65
0.58	12.75	16.86	45.61	63.48	83.75	116.47	121.27	156.99	224.31	296.91
0.72	18.93	24.71	63.85	82.15	102.17	130.81	134.19	182.10	254.66	367.93
0.90	28.68	36.36	77.89	105.84	126.30	156.03	168.53	221.88	294.40	343.14
1.13	39.34	48.86	100.63	127.33	149.64	188.77	185.48	261.31	283.18	348.70
1.41	59.43	65.96	122.18	148.15	164.28	204.65	198.30	248.06	290.78	349.42
1.76	75.52	83.58	137.06	171.39	180.50	222.19	225.85	258.78	293.88	346.20
2.20	92.43	102.53	158.13	188.43	193.80	219.11	219.67	261.12	278.94	307.60
2.75	105.40	113.84	158.75	183.68	192.08	199.43	210.84	219.22	248.01	254.38
3.44	115.92	118.97	146.51	155.31	166.61	177.36	181.93	180.92	196.89	200.95
4.29	111.71	118.38	130.04	131.13	138.05	138.47	133.54	139.00	135.38	148.16
5.37	89.70	93.98	95.28	96.46	96.80	97.77	92.25	89.51	93.30	87.18
6.71	69.26	69.94	67.56	65.69	62.30	63.24	57.72	57.31	55.97	51.61
8.39	46.51	45.71	40.22	39.74	39.06	37.26	33.71	30.94	30.31	27.09
10.49	29.62	27.60	23.38	21.57	21.21	19.78	17.89	16.00	15.18	13.39
13.11	15.77	15.40	12.18	11.30	10.62	10.00	8.64	8.07	6.90	6.08
16.38	8.45	8.02	6.39	5.69	5.11	4.91	4.51	3.86	3.13	2.57
20.48	4.37	4.30	3.16	2.82	2.54	2.32	1.90	1.65	1.09	0.94
25.60	2.39	2.21	1.63	1.44	1.27	1.10	0.96	0.67	0.45	0.40
32.00	1.37	1.25	0.90	0.76	0.66	0.56	0.54	0.39	0.23	0.20
40.00	0.85	0.80	0.57	0.47	0.41	0.33	0.33	0.24	0.16	0.12
50.00	0.60	0.56	0.37	0.36	0.30	0.24	0.25	0.19	0.11	0.090
62.50	0.51	0.48	0.32	0.31	0.24	0.20	0.23	0.16	0.10	0.078
78.13	0.50	0.46	0.32	0.29	0.24	0.20	0.23	0.16	0.10	0.077
97.66	0.50	0.46	0.32	0.29	0.24	0.20	0.23	0.16	0.10	0.077

B.1 Development of Target PSD for Bin Representative Response Spectra

To develop each target PSD in Tables 1 and 2 for the NUREG/CR-6728 bin representative design response spectra (RS_{rep}), as defined by Equations (2) and (3), an iterative frequency-by-frequency scaling approach was applied. The initial PSD value was taken as the bin average PSD for the NUREG/CR-6728 time history database (Tables 3 through 6 of Appendix B show the bin average PSD and Tables 7 through 10 show the bin average response spectra [RS]). Ten iterations were found to be adequate to produce a target PSD compatible with the bin representative RS. The iterative process consists of the following four steps:

(1) Generate *M* number of synthetic time histories from the current PSD function, where *M* took a value of 10, 20, … 100 for iteration 1, 2, …10, respectively. To generate synthetic time histories from a PSD function, (a) Fourier spectra were constructed by utilizing random phase angles and Fourier magnitudes computed following Equation (1), and (b) time histories were then generated through inverse FFT. The synthetic time histories were assumed to have 4096 data points and a time step of 0.005 s. Since the synthetic time histories are stationary at this point (the envelope function is applied in step (2) below), the entire duration of 20.48 s was used to generate the Fourier spectrum using Equation (1). The PSD function was linearly interpolated in the log-log scale to fill all frequency points for the Fourier coefficients. This method produces the same acceleration time histories as the method in Appendix B of NUREG/CR-5347.

(2) Apply a trapezoidal envelope function to the synthetic time histories (rise time = 1.4 s, strong motion duration = 10.24 s, and decay time = 7.0 s), which is Function B in Appendix B of NUREG/CR-5347.

(3) Calculate the 5% damped absolute acceleration response spectra for the synthetic time histories and obtain the arithmetic average RS_{avg}.

(4) Multiply the PSD frequency-by-frequency by $(RS_{rep}/RS_{avg})^2$, and use this adjusted PSD in the next iteration.

Convergence on the bin representative RS can be quickly achieved in the dominant frequency range of interest to structural response. However, in some cases, at the very low and/or very high frequencies, successive iterations could lead to increase or decrease of the PSD values without noticeable improvement to the RS match. This behavior may be due to the inadmissibility of the bin representative RS; they were developed by statistically fitting to the bin average RS shapes and may not necessarily be physical at these extreme frequencies. Therefore, in those cases, the PSD values at a few very low frequencies (close to 0.1 Hz) or very high frequencies (close to 100 Hz) were manually adjusted. The tabulated target PSD's in Tables 1 and 2 are values after the manual adjustment.

The converged target PSD's were smoothed using cubic splines at the frequency points as shown in the first column of Tables 1 and 2, and the tabulated target PSD's represent the smoothed PSD's.

B.2 Development of Target PSD for Other Response Spectral Shapes

For a design response spectrum different from the NUREG/CR-6728 bin representative RS, the iterative frequency-by-frequency scaling procedure described in Section B.1 can still be applied to generate the target PSD for a given design RS. The development of the target PSD in these cases is reviewed and accepted on a case-by-case basis.

It is noted that a proper choice of the initial PSD can accelerate the convergence process; for this purpose, it is recommended to use the following guidance to select the initial PSD:

(1) For a NUREG/CR-6728 design RS other than the bin representative RS (i.e., the moment magnitude and the fault distance take values other than bin midpoint values), the corresponding target PSD in Table 1 or 2 can be used;

(2) For a RS enveloping several NUREG/CR-6728 bin representative RS, the envelope of the corresponding target PSD (with proper scaling using PGA) can be used;

(3) For other RS (e.g., the vertical NUREG/CR-6728 RS or site specific RS not developed as the envelope of bin representative RS), an initial PSD can be determined by enveloping several tabulated target PSD, provided the envelope of the corresponding RS can reasonably resemble the subject RS. Note that the initial PSD does not need to be very close to the target PSD obtained after convergence; a proper initial PSD only speeds up the convergence process.

Table 3 – Horizontal Bin Average PSD for CEUS Rock Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.29	0.004	1.27	0.14	0.028	0.36	4.83	2.03	0.92	1.48
0.15	0.35	0.006	1.65	0.22	0.043	0.52	8.68	3.01	1.45	2.47
0.19	0.40	0.008	2.49	0.41	0.072	0.79	14.17	4.66	2.63	4.52
0.24	0.50	0.017	3.46	0.74	0.12	1.31	17.82	6.75	3.67	7.68
0.30	0.62	0.029	4.09	1.35	0.24	2.20	21.57	7.40	5.92	11.62
0.37	0.89	0.054	4.70	2.53	0.51	3.21	23.61	8.35	7.23	16.43
0.46	1.40	0.093	6.56	3.91	1.09	5.77	25.27	12.73	7.70	28.68
0.58	2.41	0.22	10.82	6.00	2.32	10.34	33.00	16.93	10.92	35.96
0.72	3.69	0.56	13.72	7.80	3.62	13.08	33.46	16.48	12.26	38.11
0.90	4.50	1.29	18.26	9.55	5.97	15.25	32.46	13.25	13.25	31.62
1.13	6.26	2.65	28.08	8.82	11.26	18.52	32.59	12.93	16.67	28.52
1.41	8.76	4.24	33.83	13.28	23.17	26.99	36.67	16.74	21.29	32.03
1.76	12.19	6.26	36.53	15.26	31.78	32.41	31.64	19.74	21.59	26.13
2.20	21.20	7.29	31.06	14.20	34.01	36.00	21.37	19.26	17.54	19.43
2.75	29.74	10.00	29.40	13.96	34.43	39.38	19.84	22.38	16.93	18.29
3.44	30.01	16.15	26.15	13.25	33.45	40.01	19.39	18.93	20.76	15.21
4.29	29.10	27.35	20.38	15.14	35.84	39.67	16.07	17.00	21.31	12.34
5.37	40.68	32.84	22.11	18.01	36.88	39.59	15.09	19.32	19.75	11.66
6.71	33.51	30.88	18.84	19.99	30.37	30.64	15.47	17.82	17.93	12.40
8.39	21.72	29.09	13.12	16.40	20.76	21.98	12.88	13.82	15.25	10.39
10.49	14.81	26.09	10.81	13.12	13.26	15.69	12.04	9.20	11.33	8.19
13.11	12.39	23.69	10.66	12.30	10.17	11.86	12.33	8.38	9.25	8.16
16.38	13.89	19.49	9.00	11.42	9.84	10.78	10.72	8.74	9.06	8.03
20.48	12.54	13.94	6.42	9.55	8.87	9.14	8.54	7.59	7.34	6.30
25.60	10.02	6.58	4.23	7.70	7.62	6.68	5.83	5.46	5.37	4.80
32.00	8.99	3.35	2.98	6.55	7.34	5.50	3.85	4.51	4.64	4.26
40.00	6.73	0.83	1.94	4.79	6.31	4.38	2.11	3.36	3.93	3.60
50.00	3.40	0.31	0.71	2.30	3.26	2.15	0.76	1.71	2.26	2.17
62.50	0.89	0.16	0.13	0.71	1.27	0.75	0.13	0.58	0.83	0.85
78.13	0.15	0.056	0.018	0.18	0.46	0.22	0.015	0.19	0.27	0.27
97.66	0.018	0.003		0.010	0.14	0.11	0.001	0.059	0.10	0.073

Table 4 – Horizontal Bin Average PSD for CEUS Soil Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.47	0.13	24.10	0.67	0.29	0.14	58.13	13.24	5.25	1.81
0.15	0.54	0.13	35.59	0.83	0.38	0.20	100.59	19.46	11.33	3.56
0.19	0.71	0.14	54.41	1.18	0.60	0.33	141.53	29.34	19.02	8.45
0.24	0.92	0.15	73.83	1.35	0.79	0.46	158.47	43.37	20.78	16.11
0.30	1.18	0.21	102.74	2.33	1.93	0.93	162.91	62.81	27.14	21.01
0.37	1.55	0.41	141.32	4.42	4.39	2.30	199.00	73.94	45.49	26.95
0.46	2.36	0.69	160.64	6.61	7.17	4.59	212.60	93.40	72.00	40.24
0.58	3.78	1.27	154.05	12.01	9.02	8.07	232.39	156.39	122.74	50.52
0.72	5.90	2.02	194.19	19.30	15.65	14.61	240.37	199.06	172.60	72.00
0.90	11.52	3.59	224.78	29.42	32.33	25.36	262.77	229.19	202.97	115.59
1.13	25.22	8.45	230.37	54.30	56.38	52.08	289.45	301.99	236.88	141.11
1.41	50.68	16.91	235.67	78.67	86.12	90.95	251.90	294.83	263.90	130.16
1.76	69.50	25.01	208.75	95.70	108.51	106.53	190.52	224.94	181.17	98.69
2.20	64.34	47.78	176.61	120.63	128.96	88.67	159.27	180.68	112.75	66.46
2.75	80.20	87.75	146.97	127.13	104.08	91.40	112.55	136.87	85.14	59.53
3.44	85.23	100.35	133.79	111.14	66.98	76.28	82.69	98.84	59.91	52.06
4.29	88.84	90.26	108.70	108.46	60.86	62.10	54.84	70.98	46.58	46.95
5.37	94.64	75.60	64.96	91.93	50.91	50.02	31.48	37.41	29.46	39.14
6.71	80.44	55.30	40.88	58.96	31.71	31.58	19.35	20.66	19.82	24.41
8.39	62.11	45.04	23.32	37.86	20.80	22.30	10.09	13.67	13.76	15.47
10.49	42.54	34.02	10.37	21.82	14.91	15.43	4.43	7.67	9.58	10.45
13.11	21.76	19.01	5.11	10.62	9.72	10.08	1.95	4.18	6.19	6.68
16.38	10.65	10.55	2.77	5.42	6.06	6.81	0.83	2.27	3.75	4.18
20.48	5.16	5.74	1.29	2.80	3.45	4.06	0.30	1.22	2.22	2.29
25.60	2.37	2.65	0.54	1.26	2.06	2.11	0.082	0.62	1.17	1.14
32.00	0.85	1.11	0.24	0.66	1.13	1.08	0.017	0.33	0.59	0.54
40.00	0.24	0.44	0.082	0.40	0.62	0.54	0.003	0.19	0.34	0.27
50.00	0.057	0.16	0.026	0.21	0.35	0.25	0.001	0.11	0.21	0.14
62.50	0.015	0.061	0.009	0.12	0.21	0.12		0.060	0.13	0.066
78.13	0.004	0.026	0.004	0.072	0.14	0.062		0.035	0.093	0.035
97.66	0.001	0.009	0.002	0.022	0.068	0.069		0.016	0.068	0.030

Table 5 - Horizontal Bin Average PSD for Western US Rock Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	5.34	0.26	19.26	4.63	0.88	4.36	49.94	19.56	15.75	23.74
0.15	5.77	0.31	19.82	5.99	1.10	5.79	62.82	23.14	18.14	40.83
0.19	6.24	0.40	23.58	8.53	1.74	8.92	77.30	25.72	23.07	73.14
0.24	6.94	0.52	30.66	9.85	2.67	14.72	93.18	26.34	32.28	114.01
0.30	9.33	0.77	35.89	12.92	4.16	21.56	101.53	23.92	44.49	166.64
0.37	12.36	1.15	38.59	16.39	6.24	28.11	108.57	24.87	49.62	193.93
0.46	14.75	1.68	50.12	22.50	8.19	41.47	107.82	35.08	46.02	262.41
0.58	18.36	2.53	77.44	37.27	14.69	65.87	151.07	47.35	63.48	322.30
0.72	28.92	5.11	103.27	56.46	26.96	90.64	190.71	58.33	82.73	378.85
0.90	42.77	12.01	134.78	83.37	48.25	120.30	227.50	70.85	99.80	364.34
1.13	55.94	24.79	217.25	99.21	88.43	149.73	264.19	85.99	141.88	351.00
1.41	65.72	40.02	261.26	137.27	141.15	163.39	266.24	98.44	143.41	291.86
1.76	96.91	54.34	254.95	159.16	195.11	188.23	239.18	112.96	135.72	211.43
2.20	147.57	52.94	222.22	149.38	227.58	202.77	181.65	123.33	122.11	157.62
2.75	207.46	81.55	197.55	141.96	227.39	211.92	141.92	127.26	115.58	121.42
3.44	243.51	105.39	156.80	114.99	202.70	192.41	105.93	105.38	132.19	87.53
4.29	181.17	126.56	92.89	96.90	182.66	143.36	60.19	73.92	110.17	49.56
5.37	170.63	118.07	66.59	75.89	134.19	103.07	34.22	60.05	70.02	23.99
6.71	103.04	68.41	40.06	48.89	66.64	45.20	22.61	35.71	35.41	10.28
8.39	42.20	38.44	19.66	24.16	26.27	16.37	12.66	19.41	18.16	3.52
10.49	18.77	19.78	9.31	10.48	9.11	6.40	5.59	7.74	7.88	1.03
13.11	7.48	8.08	4.05	4.02	2.33	1.93	2.67	3.30	3.16	0.37
16.38	2.51	2.86	1.57	1.26	0.56	0.45	1.11	1.28	0.88	0.11
20.48	0.69	0.82	0.49	0.38	0.13	0.12	0.37	0.51	0.45	0.030
25.60	0.17	0.18	0.048	0.10	0.038	0.024	0.069	0.046	0.011	0.007
32.00	0.035	0.044	0.011	0.021	0.010	0.005	0.018	0.016	0.002	0.002
40.00	0.006	0.009	0.003	0.005	0.005	0.002	0.004	0.006		0.001
50.00	0.001	0.002					0.001	0.001		
62.50										
78.13										
97.66										

Table 6 - Horizontal Bin Average PSD for Western US Soil Sites ($\times 10^{-3}$ m^2/s^3)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	3.46	0.97	63.25	3.65	2.77	2.32	167.95	62.09	32.99	27.11
0.15	4.02	1.01	81.58	5.35	3.33	3.05	220.35	77.75	50.64	35.44
0.19	4.89	1.33	113.06	8.24	4.35	5.04	240.35	92.37	78.28	63.40
0.24	5.55	1.86	149.47	9.66	5.48	8.17	244.32	121.75	82.10	98.24
0.30	7.82	2.50	210.85	10.82	12.53	13.70	263.47	153.14	85.99	141.16
0.37	11.03	3.21	257.74	15.43	25.78	22.68	319.90	164.73	165.22	179.40
0.46	14.80	4.81	276.50	20.41	39.23	39.54	346.60	193.37	270.84	236.50
0.58	20.90	12.09	243.59	36.13	51.51	62.45	323.45	284.68	407.42	300.99
0.72	28.38	21.28	269.68	56.53	74.68	101.32	324.60	323.25	463.50	340.78
0.90	49.73	27.48	309.37	70.16	116.62	148.45	340.55	384.31	489.71	444.15
1.13	89.82	46.84	311.57	100.26	185.88	220.90	334.61	460.99	489.05	422.95
1.41	147.65	88.19	323.60	137.83	218.18	292.94	294.47	413.49	426.22	287.97
1.76	188.00	132.11	281.45	181.46	247.19	317.76	240.99	315.90	284.80	205.64
2.20	177.54	212.95	230.91	221.95	293.99	237.48	198.79	257.99	179.55	136.21
2.75	187.23	292.94	206.71	222.37	202.90	177.13	137.35	205.42	118.53	95.09
3.44	178.03	266.10	171.91	188.12	110.60	133.29	92.05	132.37	70.22	69.62
4.29	158.85	165.70	121.19	153.58	74.01	78.80	56.27	76.39	37.14	46.76
5.37	135.92	105.08	69.94	105.78	44.38	42.18	28.77	41.37	16.19	26.27
6.71	98.09	60.78	40.88	62.23	19.37	18.72	15.99	21.01	7.37	10.76
8.39	58.51	32.31	21.44	30.97	7.60	7.44	7.69	10.76	2.86	3.23
10.49	29.26	13.13	7.97	13.93	2.52	2.41	3.26	4.80	0.95	0.98
13.11	11.95	3.50	3.05	4.76	0.73	0.79	1.41	2.06	0.26	0.22
16.38	3.62	0.75	1.09	1.38	0.22	0.20	0.56	0.67	0.050	0.059
20.48	0.89	0.17	0.36	0.41	0.076	0.047	0.20	0.20	0.012	0.019
25.60	0.18	0.026	0.073	0.054	0.029	0.010	0.027	0.030	0.002	0.005
32.00	0.052	0.005	0.017	0.015	0.010	0.002	0.007	0.007	32.99	0.002
40.00	0.010	0.001	0.004	0.006	0.005	0.001	0.002	0.003	50.64	0.001
50.00	0.002		0.001	0.001	0.001					
62.50										
78.13										
97.66										

Table 7 – Horizontal Bin Average RS for CEUS Rock Sites (g)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.002	0.002	0.006	0.007	0.005	0.012	0.026	0.035	0.040	0.042
0.15	0.004	0.003	0.010	0.011	0.009	0.021	0.044	0.058	0.059	0.066
0.19	0.005	0.004	0.017	0.018	0.014	0.032	0.074	0.078	0.092	0.11
0.24	0.008	0.007	0.030	0.030	0.024	0.050	0.12	0.10	0.12	0.16
0.30	0.013	0.010	0.047	0.044	0.036	0.073	0.16	0.14	0.14	0.23
0.37	0.019	0.015	0.065	0.067	0.053	0.10	0.21	0.16	0.18	0.28
0.46	0.029	0.024	0.090	0.10	0.075	0.14	0.24	0.21	0.22	0.42
0.58	0.046	0.037	0.15	0.14	0.12	0.20	0.33	0.27	0.27	0.52
0.72	0.073	0.060	0.21	0.21	0.17	0.25	0.42	0.32	0.34	0.62
0.90	0.11	0.10	0.28	0.27	0.25	0.34	0.49	0.37	0.39	0.67
1.13	0.16	0.16	0.39	0.30	0.36	0.42	0.59	0.43	0.49	0.75
1.41	0.26	0.24	0.52	0.41	0.56	0.59	0.67	0.52	0.63	0.91
1.76	0.37	0.33	0.63	0.50	0.78	0.78	0.74	0.66	0.73	0.92
2.20	0.52	0.43	0.72	0.60	0.88	0.88	0.75	0.78	0.74	0.90
2.75	0.73	0.61	0.82	0.69	1.07	1.11	0.83	1.04	0.91	1.05
3.44	0.93	0.85	0.88	0.79	1.23	1.28	0.99	1.13	1.09	1.02
4.29	1.05	1.28	0.92	1.02	1.48	1.43	1.05	1.24	1.27	1.10
5.37	1.40	1.61	1.16	1.22	1.69	1.74	1.12	1.48	1.44	1.13
6.71	1.52	1.73	1.25	1.51	1.70	1.68	1.32	1.56	1.54	1.24
8.39	1.50	1.98	1.29	1.59	1.66	1.67	1.38	1.71	1.62	1.28
10.49	1.58	2.20	1.38	1.67	1.55	1.63	1.57	1.60	1.71	1.35
13.11	1.54	2.26	1.64	1.81	1.56	1.69	1.92	1.73	1.77	1.59
16.38	1.82	2.32	1.79	2.09	1.75	1.80	2.08	1.94	1.97	1.75
20.48	1.94	2.19	1.84	2.10	1.88	1.91	2.26	2.00	1.99	1.87
25.60	1.97	1.98	1.85	2.14	1.97	1.99	2.20	2.02	2.06	1.89
32.00	2.09	1.81	1.92	2.15	2.12	2.07	2.28	2.17	2.12	1.99
40.00	2.16	1.65	1.82	2.07	2.14	2.06	2.11	2.22	2.12	2.00
50.00	1.94	1.52	1.56	1.90	1.94	1.93	1.78	1.99	1.80	1.68
62.50	1.59	1.36	1.30	1.47	1.50	1.48	1.38	1.58	1.41	1.39
78.13	1.26	1.19	1.16	1.22	1.25	1.24	1.19	1.21	1.21	1.18
97.66	1.10	1.10	1.07	1.09	1.13	1.12	1.08	1.12	1.11	1.09

Table 8 – Horizontal Bin Average RS for CEUS Soil Sites (g)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.004	0.003	0.027	0.010	0.009	0.011	0.11	0.077	0.063	0.041
0.15	0.005	0.004	0.046	0.016	0.015	0.016	0.17	0.13	0.10	0.065
0.19	0.008	0.006	0.079	0.025	0.025	0.025	0.24	0.20	0.16	0.11
0.24	0.011	0.009	0.14	0.040	0.041	0.041	0.32	0.29	0.19	0.18
0.30	0.017	0.014	0.22	0.059	0.064	0.061	0.42	0.39	0.24	0.25
0.37	0.028	0.021	0.32	0.094	0.12	0.094	0.54	0.50	0.38	0.35
0.46	0.044	0.035	0.43	0.14	0.17	0.15	0.66	0.57	0.59	0.44
0.58	0.072	0.064	0.53	0.20	0.25	0.24	0.85	0.77	0.84	0.61
0.72	0.12	0.10	0.72	0.29	0.36	0.35	1.08	1.02	1.15	0.82
0.90	0.19	0.15	0.93	0.42	0.52	0.54	1.33	1.26	1.42	1.13
1.13	0.33	0.25	1.19	0.59	0.76	0.74	1.60	1.61	1.71	1.36
1.41	0.56	0.42	1.41	0.88	0.98	1.12	1.79	1.91	2.06	1.56
1.76	0.72	0.58	1.54	1.06	1.28	1.39	1.86	2.02	2.02	1.58
2.20	0.89	0.86	1.66	1.39	1.71	1.49	1.97	2.05	1.91	1.47
2.75	1.13	1.29	1.83	1.70	1.91	1.82	2.03	2.13	1.92	1.65
3.44	1.36	1.71	2.04	1.84	1.76	1.94	2.09	2.22	1.86	1.79
4.29	1.71	1.89	2.18	2.20	2.01	2.00	2.13	2.20	1.94	1.94
5.37	2.02	2.12	2.06	2.31	2.07	2.14	2.00	1.93	1.77	1.93
6.71	2.18	2.12	1.99	2.26	1.95	2.08	1.81	1.85	1.75	1.74
8.39	2.39	2.21	1.88	2.27	1.84	1.95	1.65	1.70	1.66	1.64
10.49	2.28	2.29	1.64	2.12	1.77	1.99	1.48	1.59	1.61	1.55
13.11	2.13	2.08	1.50	1.77	1.67	1.82	1.37	1.46	1.51	1.49
16.38	1.87	1.88	1.41	1.57	1.65	1.74	1.28	1.42	1.46	1.48
20.48	1.65	1.84	1.30	1.45	1.49	1.62	1.22	1.30	1.36	1.38
25.60	1.48	1.60	1.24	1.31	1.40	1.53	1.18	1.25	1.34	1.35
32.00	1.33	1.45	1.20	1.23	1.28	1.40	1.14	1.18	1.28	1.24
40.00	1.23	1.32	1.15	1.20	1.23	1.33	1.10	1.17	1.22	1.20
50.00	1.18	1.22	1.14	1.16	1.18	1.26	1.08	1.13	1.15	1.18
62.50	1.12	1.15	1.09	1.14	1.12	1.21	1.05	1.08	1.11	1.13
78.13	1.07	1.10	1.06	1.07	1.10	1.13	1.03	1.06	1.07	1.07
97.66	1.05	1.06	1.04	1.06	1.10	1.11	1.01	1.04	1.08	1.10

Table 9 - Horizontal Bin Average RS for Western US Rock Sites (g)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.005	0.005	0.015	0.015	0.012	0.036	0.060	0.066	0.11	0.12
0.15	0.007	0.007	0.025	0.024	0.020	0.060	0.095	0.10	0.14	0.18
0.19	0.011	0.012	0.041	0.038	0.033	0.093	0.15	0.11	0.20	0.27
0.24	0.017	0.018	0.070	0.063	0.054	0.14	0.23	0.14	0.25	0.39
0.30	0.026	0.027	0.11	0.093	0.080	0.20	0.32	0.19	0.30	0.54
0.37	0.039	0.039	0.15	0.14	0.12	0.26	0.39	0.21	0.36	0.66
0.46	0.059	0.061	0.21	0.20	0.16	0.34	0.46	0.29	0.43	0.94
0.58	0.10	0.10	0.34	0.30	0.25	0.48	0.65	0.40	0.55	1.20
0.72	0.15	0.15	0.53	0.44	0.36	0.62	0.88	0.53	0.73	1.49
0.90	0.24	0.25	0.73	0.64	0.56	0.87	1.12	0.69	0.86	1.68
1.13	0.38	0.43	1.01	0.82	0.86	1.08	1.45	0.88	1.14	1.92
1.41	0.55	0.59	1.29	1.08	1.19	1.36	1.63	1.06	1.36	2.11
1.76	0.79	0.81	1.47	1.36	1.57	1.71	1.74	1.33	1.53	2.01
2.20	1.17	0.93	1.83	1.56	1.91	1.99	1.91	1.70	1.64	2.06
2.75	1.54	1.34	1.98	1.72	2.19	2.36	1.96	2.04	1.92	2.16
3.44	2.02	1.77	2.02	1.91	2.51	2.63	2.13	2.22	2.33	2.14
4.29	2.09	2.19	1.91	2.11	2.76	2.67	1.94	2.25	2.49	2.01
5.37	2.52	2.62	1.98	2.18	2.78	2.74	1.80	2.39	2.39	1.78
6.71	2.37	2.38	1.94	2.14	2.36	2.20	1.82	2.22	2.14	1.60
8.39	2.06	2.23	1.76	1.92	1.95	1.83	1.69	2.09	1.94	1.38
10.49	1.89	2.03	1.62	1.73	1.61	1.53	1.59	1.72	1.75	1.23
13.11	1.60	1.78	1.54	1.52	1.36	1.31	1.51	1.52	1.50	1.21
16.38	1.45	1.58	1.39	1.33	1.23	1.17	1.38	1.37	1.33	1.11
20.48	1.31	1.41	1.26	1.20	1.13	1.10	1.30	1.22	1.17	1.06
25.60	1.16	1.28	1.14	1.13	1.09	1.05	1.18	1.11	1.10	1.03
32.00	1.10	1.18	1.10	1.07	1.07	1.03	1.13	1.08	1.06	1.03
40.00	1.06	1.09	1.06	1.05	1.05	1.02	1.08	1.06	1.05	1.02
50.00	1.04	1.04	1.03	1.03	1.02	1.01	1.03	1.03	1.02	1.01
62.50	1.02	1.02	1.02	1.02	1.01	1.01	1.02	1.02	1.02	1.01
78.13	1.01	1.01	1.02	1.01	1.01	1.00	1.01	1.01	1.01	1.00
97.66	1.01	1.01	1.01	1.01	1.01	1.01	1.00	1.01	1.01	1.00

Table 10 - Horizontal Bin Average RS for Western US Soil Sites (g)

Frequency (Hz)	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 km	10-50 km	50-100 km	100-200 km
0.12	0.006	0.005	0.044	0.018	0.020	0.027	0.15	0.13	0.12	0.10
0.15	0.009	0.007	0.072	0.030	0.034	0.046	0.22	0.20	0.18	0.16
0.19	0.013	0.011	0.12	0.045	0.056	0.072	0.31	0.27	0.28	0.24
0.24	0.021	0.018	0.20	0.072	0.094	0.11	0.40	0.39	0.32	0.40
0.30	0.033	0.031	0.30	0.11	0.14	0.16	0.51	0.51	0.41	0.53
0.37	0.057	0.051	0.43	0.16	0.25	0.24	0.65	0.65	0.61	0.70
0.46	0.092	0.086	0.55	0.22	0.35	0.35	0.81	0.72	0.91	0.86
0.58	0.14	0.16	0.67	0.33	0.50	0.54	0.98	0.96	1.26	1.14
0.72	0.23	0.26	0.88	0.45	0.67	0.74	1.21	1.21	1.56	1.44
0.90	0.34	0.35	1.10	0.60	0.90	1.07	1.46	1.49	1.87	1.86
1.13	0.53	0.55	1.35	0.82	1.25	1.35	1.70	1.89	2.18	2.11
1.41	0.84	0.83	1.60	1.09	1.47	1.81	1.90	2.08	2.39	2.14
1.76	1.10	1.17	1.82	1.41	1.84	2.16	2.04	2.29	2.35	2.16
2.20	1.39	1.61	1.88	1.78	2.44	2.26	2.13	2.31	2.24	1.97
2.75	1.64	2.18	2.10	2.12	2.41	2.39	2.16	2.41	2.11	2.02
3.44	1.89	2.53	2.23	2.31	2.21	2.43	2.13	2.37	2.01	2.07
4.29	2.15	2.43	2.34	2.51	2.28	2.21	2.11	2.20	1.89	2.02
5.37	2.35	2.44	2.18	2.47	2.17	2.12	1.95	1.98	1.60	1.86
6.71	2.39	2.25	2.06	2.35	1.89	1.83	1.75	1.77	1.51	1.54
8.39	2.35	2.02	1.85	2.15	1.59	1.58	1.58	1.60	1.35	1.32
10.49	2.04	1.81	1.60	1.92	1.42	1.39	1.41	1.43	1.23	1.17
13.11	1.81	1.52	1.40	1.54	1.25	1.22	1.30	1.29	1.14	1.09
16.38	1.50	1.27	1.30	1.33	1.14	1.13	1.22	1.21	1.07	1.05
20.48	1.30	1.17	1.19	1.19	1.07	1.07	1.16	1.11	1.03	1.03
25.60	1.19	1.08	1.12	1.08	1.06	1.05	1.09	1.06	1.02	1.01
32.00	1.12	1.04	1.09	1.05	1.03	1.03	1.06	1.03	1.01	1.01
40.00	1.07	1.03	1.05	1.04	1.02	1.01	1.03	1.03	1.01	1.01
50.00	1.03	1.02	1.03	1.02	1.01	1.01	1.01	1.01	1.00	1.01
62.50	1.02	1.01	1.01	1.02	1.01	1.01	1.01	1.01	1.01	1.01
78.13	1.01	1.00	1.01	1.01	1.01	1.00	1.00	1.00	1.00	1.00
97.66	1.01	1.00	1.00	1.01	1.01	1.01	1.00	1.00	1.00	1.01

4. Technical Basis and/or Rationale

The purpose of SRP 3.7.1, Appendix B is to provide guidance for determining the minimum PSD compatible to a design response spectrum with spectral shape consistent with the magnitude and distance bins in NUREG/CR-6728, as well as other spectra. Developing the appropriate target PSD is important in order to ensure that the artificial time history that adequately envelops the design response spectra has sufficient power throughout the frequency range of interest. This check is performed in addition to verifying that the spectra developed from the artificial time history envelops the design response spectra.

The average one-sided PSD defined by Equation (1), in SRP 3.7.1, Appendix B, should exceed 70 percent of the target PSD appropriate for the corresponding magnitude and distance bin. The approach used to calculate the bin target PSD values has been further improved to ensure that the bin target PSDs are compatible with the bin representative NUREG/CR-6728 design response spectra. The NUREG/CR-6728 design spectra are functions of earthquake moment magnitude M and fault distance R (km) and the bin representative design response spectra are defined as those with M and R taking the midpoint bin values. The use of the midpoint bin values for M and R in the definition of bin representative design spectra facilitates the application of Appendix B because each bin has a single pair of target PSD and the corresponding bin representative RS. In addition, guidance is provided to explain how to obtain target PSDs when the design response spectra do not have the same shapes as the bin representative NUREG/CR-6728 design response spectra.

In light of different ways for normalization in discrete Fourier transforms, Equation (2) of SRP 3.7.1 Appendix B provides a definition of the Fourier amplitude that is consistent with the target PSDs in Appendices A and B. More details on PSD formulation can be found in Subsection 5.1 of Appendix to this Technical Issue.

The target PSDs for the bin representative NUREG/CR-6728 design spectra are presented in Tables 1 and 2 of SRP 3.7.1 Appendix B. These PSDs were determined following the approach presented in Subsection B.1. Subsection B.2 provides guidance for developing target PSDs for response spectral shapes different from the bin representative NUREG/CR-6728 design spectra.

The lower bound frequency for PSD check was determined to be 0.3 Hz because (1) power below 0.3 Hz has no influence on stiff nuclear plant structures, and (2) adjustment of tabulated PSD values below 0.3 Hz does not noticeably affect the compatibility (as shown by the figures with the term “Adjusted” in the captions in Subsection 5.3 of Appendix to this Technical Issue).

Section 4 of Appendix to this Technical Issue provides sensitivity studies to confirm that the upper bound frequencies for PSD checks should be consistent with the frequency range of interest for the design response spectra. The objective is to ensure that removing all tabulated PSD points above the upper bound frequency does not significantly affect the compatibility of the PSD with the RS.

In SRP 3.7.1 Appendix B, 70% of the target PSD is used for PSD check, which is different from the 80% criterion in SRP 3.7.1, Appendix A. This is because the response spectra compatible with the target PSDs in Tables 1 and 2 of SRP 3.7.1 Appendix B are very close to the design response spectra but the response spectrum compatible with the target PSD in SRP 3.7.1

Appendix A is generally lower than the RG 1.60 response spectrum. More details on the choice of the 70% factor can be found in Section 2 of Appendix to this Technical Issue.

SRP 3.7.1 Appendix B also describes the horizontal spectral shapes for rock site conditions appropriate for the CEUS and the WUS as a function of the earthquake moment magnitude M and the fault distance R. The equations provided, which are taken from NUREG/CR-6728, were developed based on the NUREG/CR-6728 acceleration time history databases and five empirical attenuation models.

SRP 3.7.1 Subsection B.1 presents an iterative frequency-by-frequency scaling approach to the development of the target PSD for each NUREG/CR-6728 bin representative design response spectrum, using the bin average PSD as the initial value for the target PSD. The application of this approach in the development of the target PSDs in SRP 3.7.1 Appendix B indicates that 10 iterations were adequate to reach the converged target PSD compatible with the bin representative RS. It should be noted that a general numerical convergence criterion did not exist during this effort. In addition, in some cases the converged target PSD may need to be manually adjusted at a few non-dominating frequencies, in the very low and/or very high frequency range(s). This adjustment is made because the iterative frequency-by-frequency scaling method can sometimes, only at the very low frequencies and/or very high frequencies, continue to increase or decrease the PSD values without noticeable improvement to the response spectral match. This behavior may be due to the inadmissibility of the bin representative RS; they were developed by statistically fitting to the bin average RS shapes and may not necessarily be physical at these extreme frequencies. The adjusted target PSDs were confirmed to maintain their compatibility with the bin representative design spectra, as can be seen in the figures with the term “Adjusted” in their caption in Section 5 of Appendix to this Technical Issue. The figures for rock sites show both the bin average PSD and bin target PSD, as well as the corresponding RS, while the figures for soil sites show bin average PSDs and bin average RS.

SRP 3.7.1 Subsection B.2 provides guidance on the application of the iterative frequency-by-frequency scaling approach for developing target PSD for response spectra with shapes different from the bin representative NUREG/CR-6728 design response spectra. In order to accelerate the convergence process, it includes recommendations on how to determine proper initial PSDs for three different cases, which are general enough for the iterative approach to be essentially applicable to many different types of response spectral shapes. However, the development of target PSD is reviewed and accepted on a case-by-case basis.

Additional detailed technical basis, regarding the development of the target PSD for NUREG/CR-6728-based design spectra or other spectra, is provided in Appendix to Technical Issue 12 that follows Section 5 below.

5. References

[1] NUREG/CR-6728, “Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines,” October 2001.

[2] NUREG/CR-5347,"Recommendations for Resolution of Public Comments on USI A-40, Seismic Design Criteria," June 1989.

[3] NUREG/CR-3509, "Power Spectral Density Functions Compatible with NRC Regulatory Guide 1.60 Response Spectra," June 1988.

[4] Regulatory Guide 1.60, Rev. 1, “Design Response Spectra for Seismic Design of Nuclear Power Plants”. U.S. Nuclear Regulatory Commission, 1971.

[5] Wirsching, Paez, Ortiz, “Random Vibrations: Theory and Practice”, John Wiley and Sons, Inc., 1995.

[6] Mario Paz, “Structural Dynamics: Theory and Computation” Van Nostrand Rein¬hold Company, 3nd Edition, 1989.

[7] Trifunic, M. D. and Brady A. G., "A Study on the Duration of Strong. Earthquake Ground Motion," Bulletin, Seismological Society of America, vol. 65, no. 3, pp 581-626, June 1975.

Appendix to Technical Issue 12

This Appendix provides supporting analyses and results for the tabulated target power spectral density (PSDs) and the associated procedure in SRP 3.7.1 Appendix B.

1. Development of Target PSD for Multiple Consistent Response Spectra

The guidelines and procedures described in SRP 3.7.1 Appendix B are consistent with the approach described in NUREG/CR-5347, which was also used to develop the target PSD for RG 1.60 RS in SRP 3.7.1 Appendix A. The target PSD functions presented in Tables 1 and 2 of SRP 3.7.1 Appendix B were developed for the NUREG/CR-6728 bin representative design acceleration response spectra, which are based on a damping ratio of 5%. On the other hand, the target PSD for SRP 3.7.1 Appendix A was developed based on 2% damped pseudo relative velocity response spectra. It is noted that the development of target PSD following the SRP 3.7.1 Appendix B procedure should not be sensitive to the selection of a particular damping value because the calculation of PSD is independent of damping, which is confirmed by the study described below.

To demonstrate that the tabulated target PSD values in SRP 3.7.1 Appendix B are not sensitive to damping ratios, two representative bins from the NUREG/CR-6728 database, CEUS_ROCK_M75D000.010 and WUS_ROCK_M75D000.010, were selected to represent the CEUS rock sites and WUS rock sites, respectively. These two bins are those with the highest earthquake magnitude and shortest fault distance in the NUREG/CR-6728 database. For each of these two bins, the following steps were performed to demonstrate the insensitivity of target PSD to damping values associated with RS:

(1) use the target PSD of SRP 3.7.1 Appendix B to generate 1,000 synthetic acceleration time histories, which should be sufficient to produce smooth average response spectra;
(2) compute the response spectra for these synthetic acceleration time histories at 2% and 10% damping values and obtain the average response spectra for the respective damping values;
(3) use cubic splines to obtain the smoothed average response spectra;
(4) apply the procedure in SRP 3.7.1 Appendix B to generate two target PSDs from the 2%- and 10%-damped, smoothed average response spectra;
(5) compare these two target PSDs to the tabulated target PSD that was computed from the 5% damped NUREG/CR-6728 bin representative design response spectrum.

Figure 1 and Figure 2 show the average response spectra for 2%, 5%, and 10% damping ratios and their smoothed versions. These figures confirmed that the use of 1,000 synthetic acceleration time histories produced sufficiently smooth average response spectra as expected, and the cubic spline smoothing further removed the minor irregularity from these spectral curves. It should be pointed out that these average response spectra generated based on the tabulated target PSDs have in general the same shapes as but are not as smooth as the design spectra prescribed by the NUREG/CR-6728 formula. This is due to the fact that the tabulated target PSDs are represented at only a limited number of frequency points. Nevertheless, the minor deviation in response spectral shapes from the formula does not affect the sensitivity study in this section to investigate whether the development of target PSD is sensitive to the damping ratio associated with the response spectra, because the smoothed average response spectra shown in Figure 1 and Figure 2 are compatible with the same tabulated target PSDs.

Figure 3 and Figure 4 compare the target PSDs computed based on the RS with damping ratios 2% and 10% to the tabulated target PSDs in SRP 3.7.1 Appendix B, which were computed based on the NUREG/CR-6728 design response spectra at a damping ratio of 5%. These target PSDs were calculated by assuming an initial PSD value of a constant 0.001 $g \cdot g \cdot s$ (0.096 m^2/s^3) for all frequencies and the iterative procedure was performed for 10 iterations as described in SRP 3.7.1, Appendix B.1. As shown by the two figures, the target PSDs based on the 2% and 10% damped RS agree very well with the tabulated target PSD over almost the entire frequency range, except for the case of 10% damped RS where the target PSDs are slight higher than the other two cases at higher frequencies. This is due to the choice of a constant initial PSD; 20 more iterations for the case of 10% damped RS greatly improved the agreement, as shown in Figure 5 and Figure 6. This exercise confirms that a proper choice of initial PSD can accelerate the convergence process, as indicated in SRP 3.7.1 Appendix B.

In particular, a good choice of initial PSD for this sensitivity study is the tabulated target PSD, because in theory the PSD calculation does not involve damping and it should be the same regardless what damping value is used for RS. Based on the tabulated target PSD (as the initial PSD), Figure 7 and Figure 8 show much better comparisons than Figure 3 and Figure 4. The use of tabulated target PSD as the initial PSD in the iterative procedure produced excellent agreement among the target PSDs developed based on RS associated with different damping ratios.

In conclusion, the development of target PSD is not sensitive to the damping ratio associated with the response spectrum. The minor difference between the target PSDs developed based on differently damped response spectra does not represent any significant problem in using the target PSD as a secondary check of the synthetic acceleration time histories to detect any potential deficiency of power.

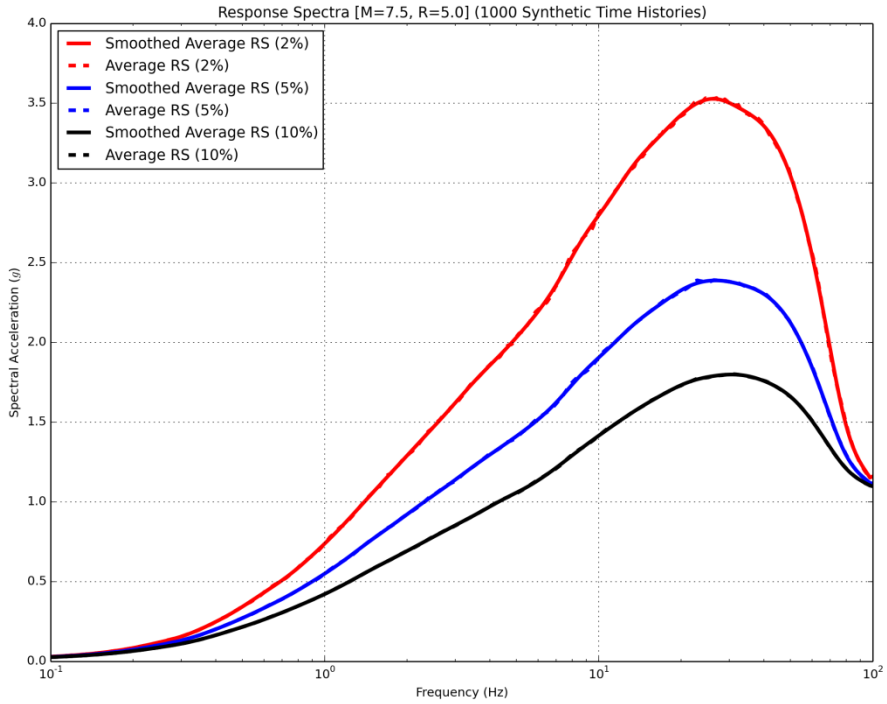

Figure 1 Response Spectra with Different Damping Ratios for CEUS ROCK M75D000.010

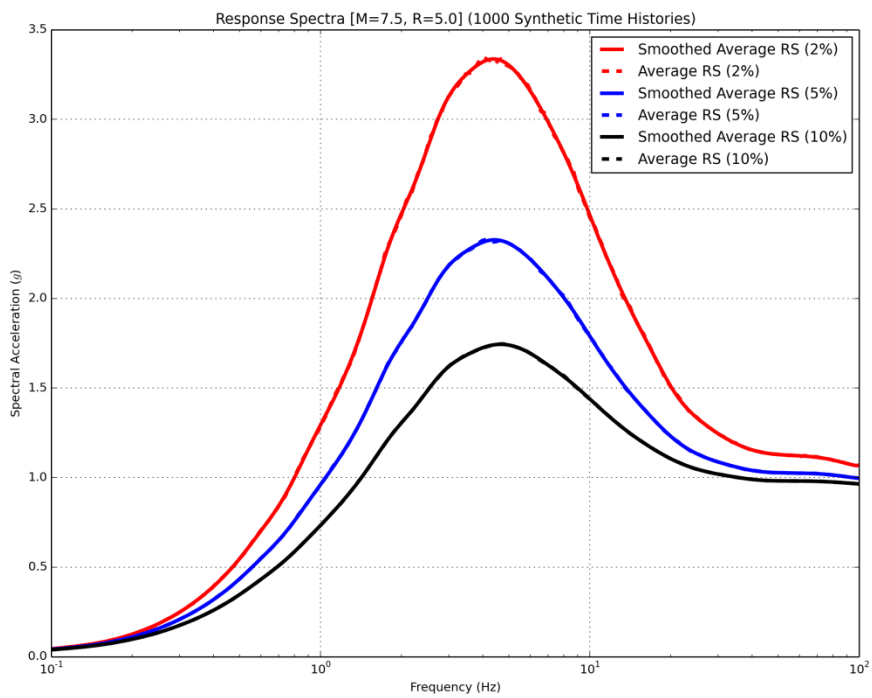

Figure 2 Response Spectra with Different Damping Ratios for WUS ROCK M75D000.010

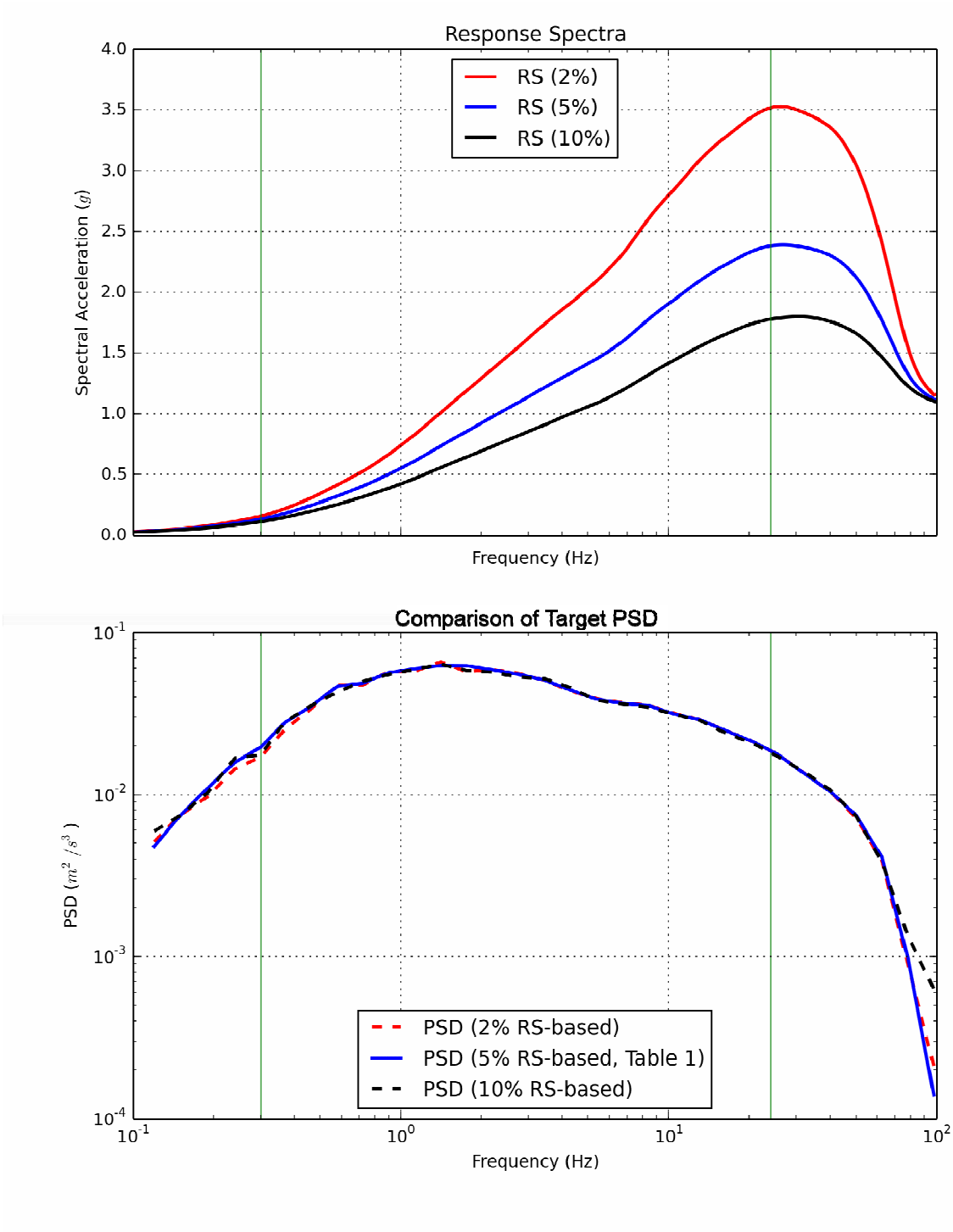

Figure 3 Comparison of PSDs Generated from Differently Damped RS (CEUS ROCK M75D000.010)

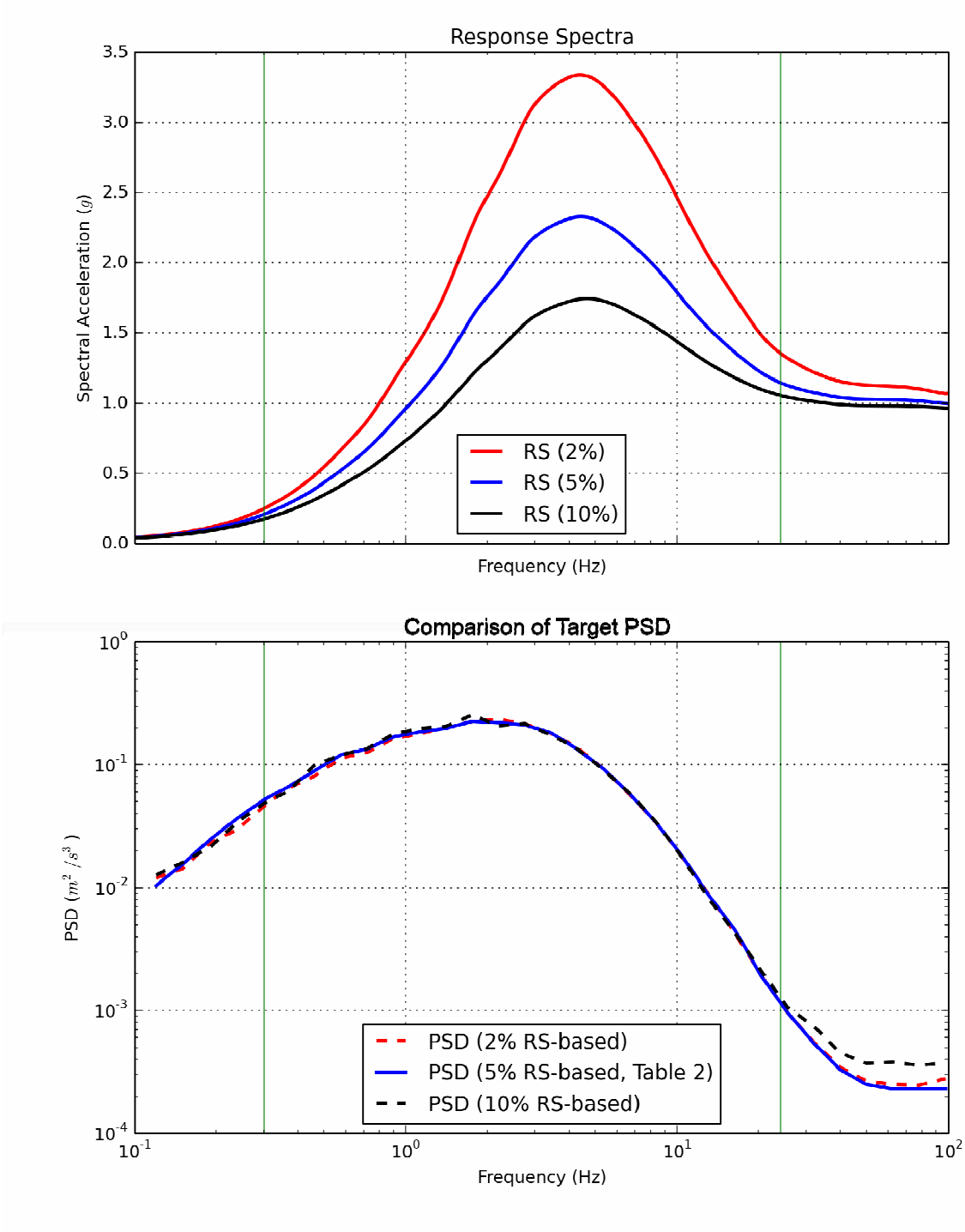

Figure 4 Comparison of PSDs Generated from Differently Damped RS (WUS ROCK M75D000.010)

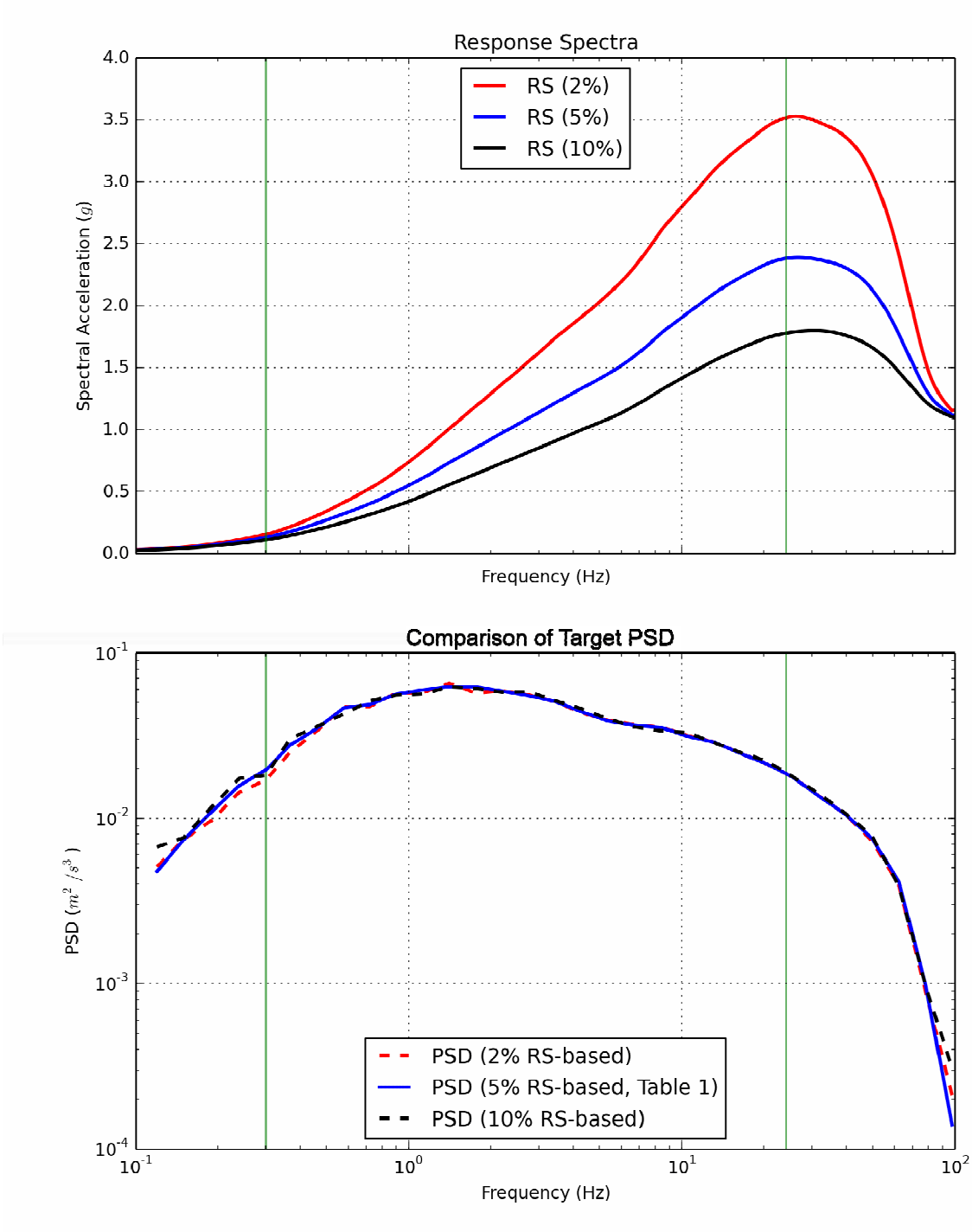

Figure 5 Comparison of PSDs Generated from Differently Damped RS, 30 Iterations for Damped RS (CEUS ROCK M75D000.010)

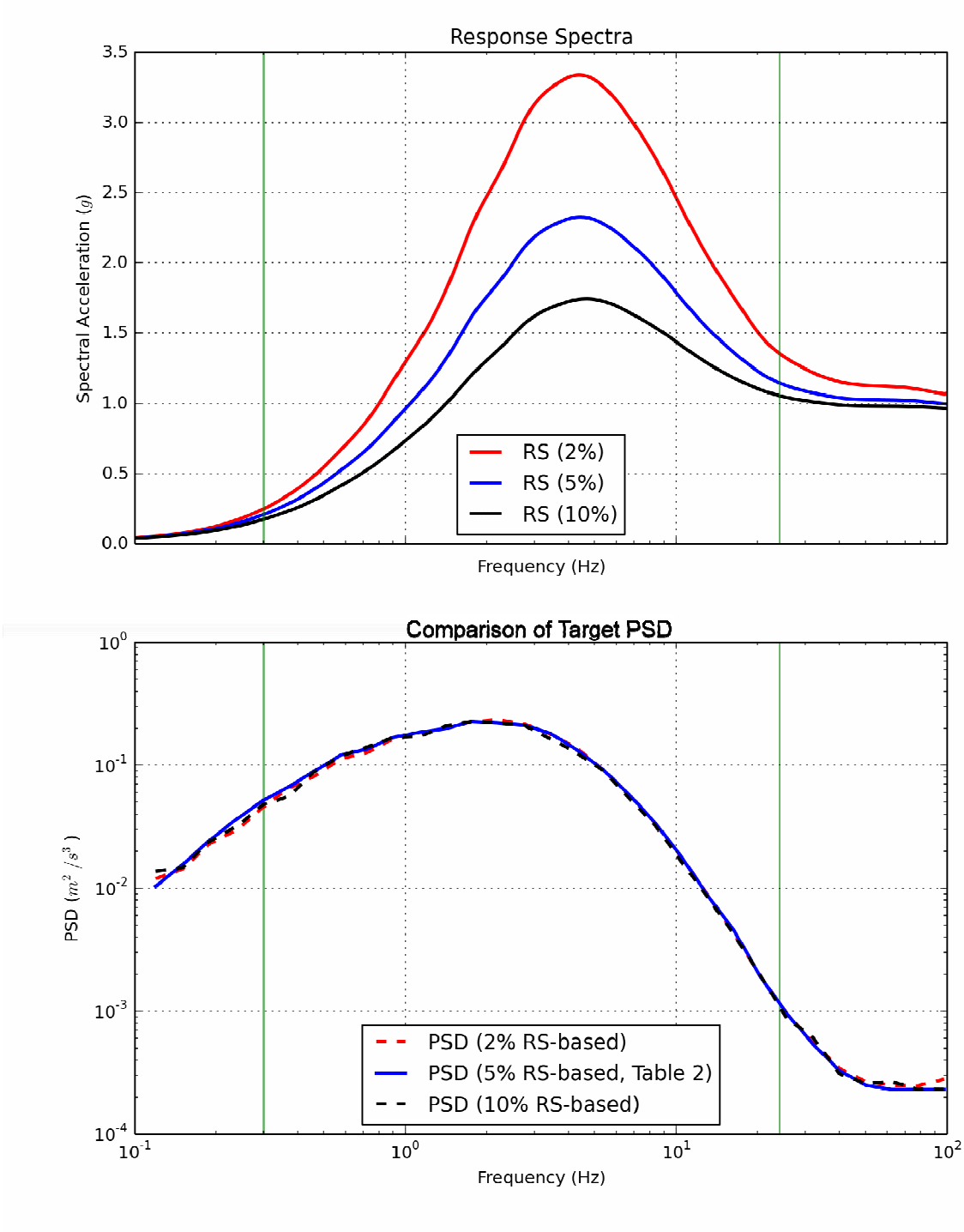

Figure 6 Comparison of PSDs Generated from Differently Damped RS, 30 Iterations for Damped RS (WUS ROCK M75D000.010)

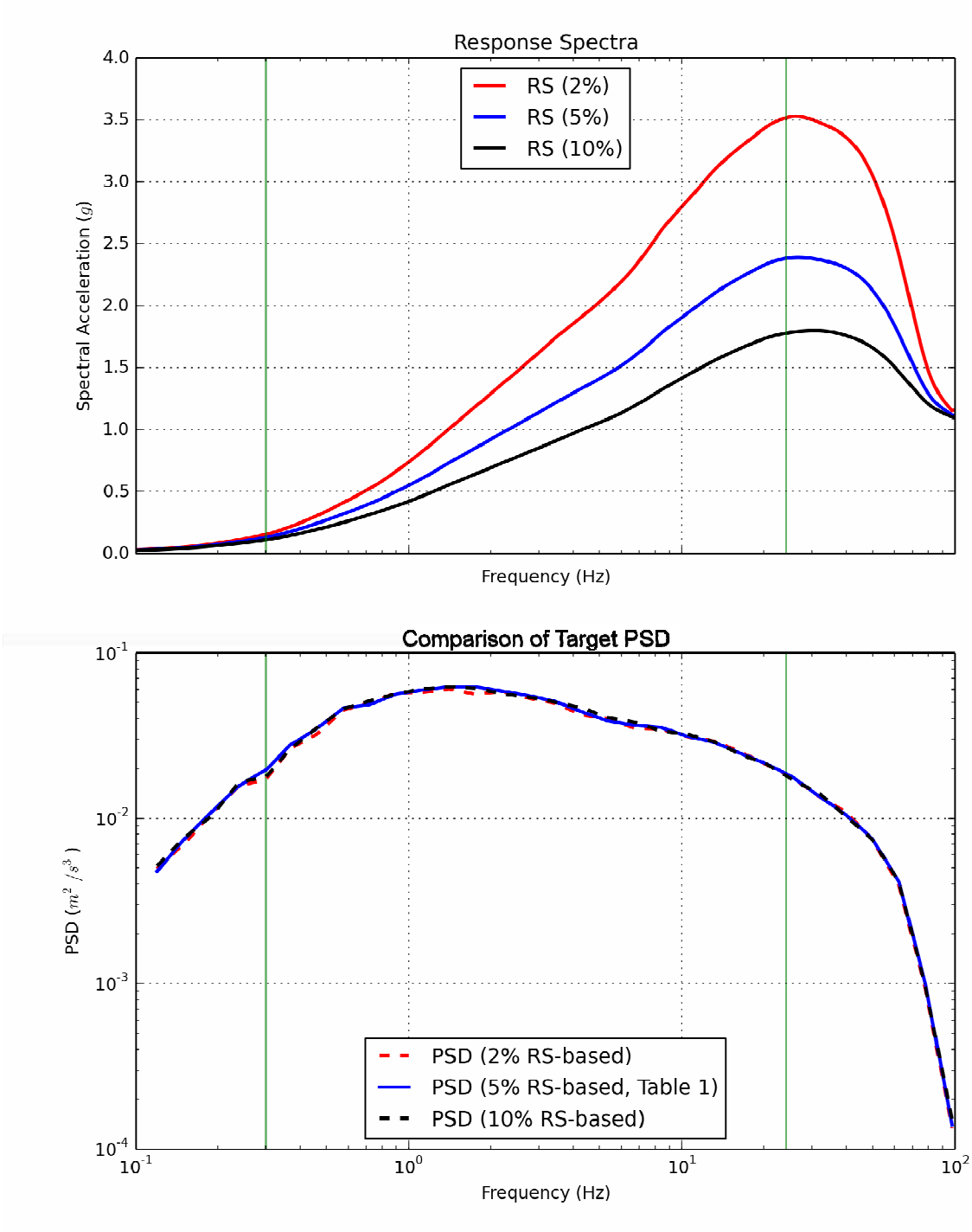

Figure 7 Comparison of PSDs Generated from Differently Damped RS, Initialized with Tabulated Target PSD (CEUS ROCK M75D000.010)

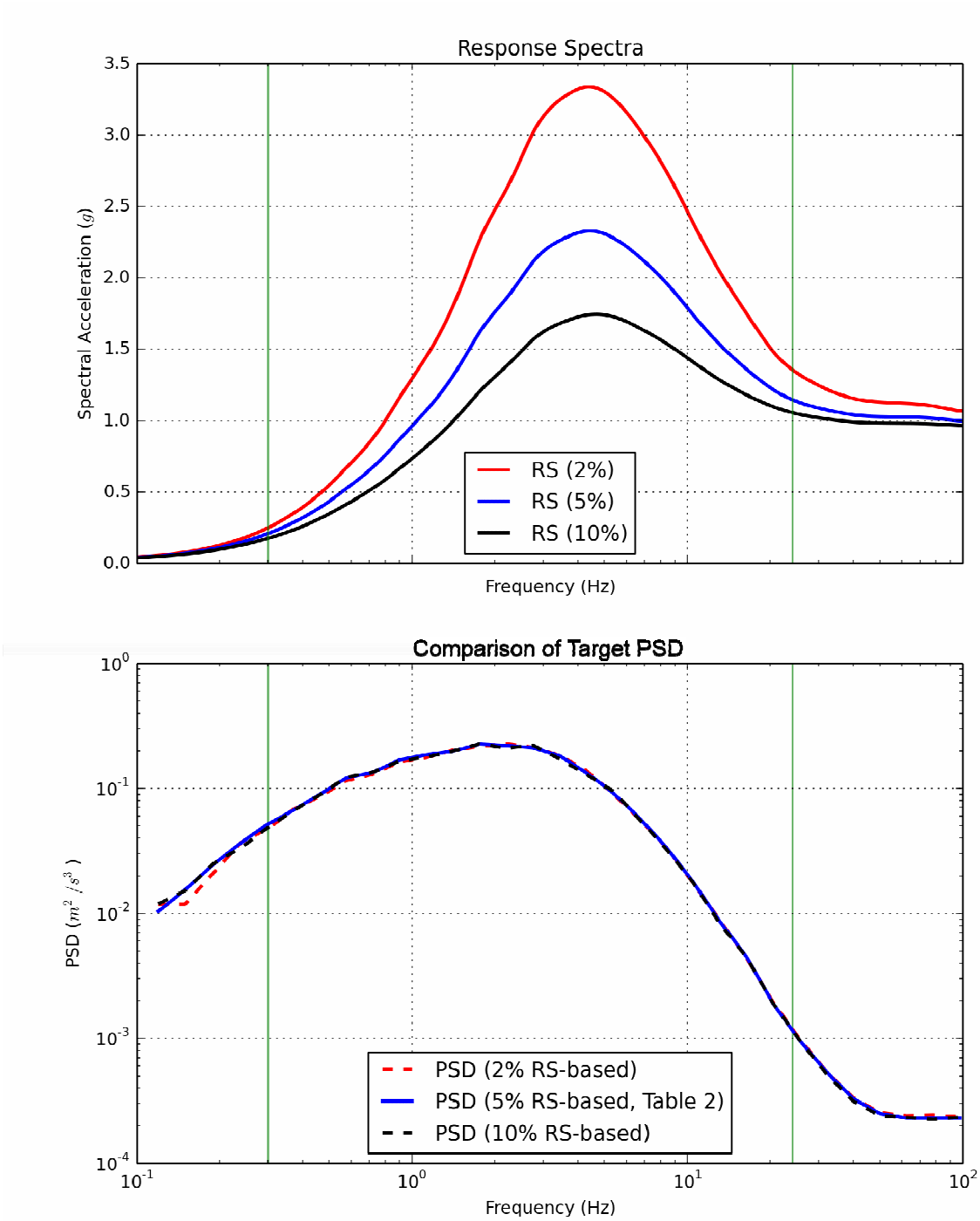

Figure 8 Comparison of PSDs Generated from Differently Damped RS, Initialized with Tabulated Target PSD (WUS ROCK M75D000.010)

2. Sufficiency of Power in Synthetic Time History Based on 70 percent Target PSD

As described in Subsections B.1 and B.2 of SRP 3.7.1 Appendix B, the iterative procedure to generate the target PSD for a given design response spectrum converges to the target PSD, so that the average response spectrum of the synthetic acceleration time histories generated from this PSD is very close to the given design response spectrum. This is different from the criterion used in the development of the target PSD for SRP 3.7.1 Appendix A, as described in NUREG/CR-5347, in that the response spectrum compatible with the target PSD is generally lower than the RG 1.60 response spectrum. To achieve a PSD check consistent with SRP 3.7.1 Appendix A, the computed PSD from the synthetic time history is expected to be above 70 percent of the target PSD developed based on SRP 3.7.1 Appendix B, as opposed to the 80 percent factor used in SRP 3.7.1 Appendix A.

To derive the 70 percent factor for use with SRP 3.7.1 Appendix B, the iterative procedure described in SRP 3.7.1 Appendix B was applied to the RG 1.60 response spectrum and a target PSD was determined accordingly. The frequency-by-frequency ratios of the target PSD defined by Equation (2) of SRP 3.7.1 Appendix A over the target PSD developed based on the SRP 3.7.1 Appendix B procedure were calculated for the frequency range of 0.3 Hz to 24 Hz and the geometric mean of these ratios was found to be 0.89. Therefore, the adjusted factor for use with SRP 3.7.1 Appendix B target PSDs can be determined as 0.89 x 80% ≈ 70%.

3. The Role of Seed Selection in PSD Check

As described in SRP 3.7.1, the seed recorded time histories should have a similar response spectral shape to the target spectra across the frequency range of interest to the analysis and the phasing characteristics of the earthquake records should not change significantly. In addition, the staff's experience also indicates that seed records can play an important role in achieving a satisfactory PSD check as well, when they do not exhibit sufficient frequency-stationarity in the strong-motion duration. In SRP 3.7.1 Appendices A & B, the strong motion duration of an acceleration time history is used to calculate the PSD because it represents the duration of "near maximum and nearly stationary power of the acceleration time history." In general, the PSD estimate is sensitive to how the strong motion duration is selected for typical earthquake ground motions.

Stationarity is manifested in both amplitude stationarity and frequency stationarity, the former of which can be fairly easily represented by a straight line in the Husid plot but the latter cannot as easily be represented. Since a PSD function describes power distribution over frequency, a frequency non-stationarity in the strong motion usually leads to underestimating the true power that a structure experiences for those frequencies that do not exist for the entire strong motion duration. For many acceleration time history records, stationarity is well demonstrated in the strong motion portion and thus the use of the strong motion duration is sufficient for the PSD check. However, there are cases where the stationarity cannot be easily identified for a proper determination of the strong motion duration, and therefore, can lead to an unsatisfactory PSD check. Common techniques to identify the strong motion duration, such as a duration corresponding to the 5%-to-75% rise of the cumulative Arias energy or a nearly linear portion of it, could underestimate the PSD if some bands of frequencies do not exist for the entire identified duration.

An un-satisfactory PSD check usually indicates that the strong motion portion of the seed recorded ground motion is not frequency-stationary. In such cases, a different seed should be pursued. Seeds of shorter strong motion durations often show higher level of stationarity (frequency stationarity in particular) and can make the PSD check easier to satisfy.

4. Effect of High-Frequency Power on PSD-RS Compatibility

In Appendices A and B of SRP 3.7.1, the PSD check has been provisioned to exclude a check below 0.3 Hz, because power below 0.3 Hz has no influence on stiff nuclear plant structures. SRP 3.7.1 Appendix A also states that the power above 24 Hz for the target PSD is so low as to be inconsequential so that checks above 24 Hz are unnecessary. This frequency is somewhat lower than the ZPA frequency of 33 Hz for the referenced RG 1.60 design response spectrum. For SRP 3.7.1 Appendix B, since it is applicable to the NUREG/CR-6728 bin representative response spectra and other design response spectra, the upper frequency limit for PSD check cannot be explicitly set to a fixed value and the guidance in SRP 3.7.1 Appendix B states that it should be consistent with the design response spectrum.

Section 4.1 below attempts to investigate whether upper frequency limits can be determined by using a power level equal to what the 24 Hz frequency limit implies in SRP 3.7.1 Appendix A. Section 4.2 describes a study to show the effect of the PSD values at higher frequencies on the compatibility between the target PSD and the corresponding NUREG/CR-6728 bin representative response spectra.

4.1 Approaches based on Cumulative PSD or Cumulative Square Root of PSD

A plateau in the cumulative power with respect to frequency could indicate that the power beyond a certain frequency (e.g., 24 Hz in SRP 3.7.1 Appendix A) is very small and that frequency, together with other considerations such as the ZPA frequency of the design response spectra and the dynamic characteristics of the soil-structure-equipment system, can be used to develop an upper frequency limit for PSD check. This subsection presents the results of an effort for determining upper frequency limits using the cumulative PSD or cumulative square root of PSD (effectively equivalent to cumulative Fourier amplitude spectra) as a measure to signify negligible power.

Using the target PSD in SRP 3.7.1 Appendix A, the level of cumulative target PSD that corresponds to the upper bound frequency 24 Hz was determined to be 0.9955 of the maximum cumulative target PSD. Similarly, the level of cumulative square root of target PSD at 24 Hz was calculated to be 0.97. These two levels, together with three other levels 0.90, 0.95, and 0.99, were used to estimate the cutoff frequencies for all 20 bins associated with Tables 1 and 2 of SRP 3.7.1 Appendix B.

As shown in the following table, even though both the level of 0.9955 for cumulative PSD and the level of 0.97 for cumulative square root of PSD correspond to the same cutoff frequency of 24 Hz in SRP 3.7.1 Appendix A, they do not lead to the same or close cutoff frequencies for the PSDs in Tables 1 and 2 of SRP 3.7.1 Appendix B. Small difference in these cumulative measures (e.g., 0.99 versus 0.9955) can lead to large differences in cutoff frequency estimates because the cumulative curves are very flat at higher frequencies. For the target PSDs associated with Western US spectral shapes, the levels of 0.9955 for cumulative PSD and 0.97 for cumulative square root of PSD led to generally small upper limit frequencies (smaller than 33

Hz). As extreme cases, both approaches led to cutoff frequencies less than 10 Hz for the bin WUS M75D100-200, which are considered too low. Most importantly, many of these estimated cutoff frequencies can lead to significant incompatibility between the truncated PSD and the RS, as can be seen by comparing these cutoff frequencies to the figures presented in Subsection 4.2 below.

Cutoff Frequencies Calculated based on Cumulative Measures

Level	M 5-6		M 6-7				M 7+			
	0-50 km	50-100 km	0-10 km	10-50 km	50-100 km	100-200 km	0-10 Km	10-50 km	50-100 km	100-200 km
Calculated Cutoff Frequency based on Cumulative CEUS PSD										
0.90	27.89	26.29	20.68	19.16	17.81	16.41	13.92	12.25	11.09	9.99
0.95	37.75	36.51	30.61	28.71	26.96	25.57	22.52	20.30	18.85	17.01
0.99	57.37	56.37	50.92	48.97	48.38	46.80	44.19	42.00	40.41	38.27
0.9955	*63.89*	*62.37*	*59.38*	*57.92*	*57.31*	*55.47*	*52.82*	*50.67*	*49.36*	*47.59*
Calculated Cutoff Frequency based on Cumulative Square Root of CEUS PSD										
0.90	34.60	33.67	28.25	26.61	25.55	24.66	22.18	20.61	19.61	18.19
0.95	47.73	46.97	41.74	39.82	39.13	38.01	35.48	33.88	32.62	30.92
0.97	*56.57*	*55.86*	*50.61*	*48.61*	*48.37*	*47.08*	*44.87*	*43.23*	*42.16*	*40.01*
0.99	72.72	72.42	67.62	64.40	64.60	62.86	61.08	59.92	58.97	57.16
Calculated Cutoff Frequency based on Cumulative WUS PSD										
0.90	7.28	6.87	5.52	5.09	4.77	4.35	4.22	3.82	3.41	3.08
0.95	9.59	9.12	7.30	6.66	6.34	5.86	5.64	5.09	4.64	4.11
0.99	17.80	16.61	12.89	11.88	10.96	10.13	9.84	8.77	7.94	7.13
0.9955	*25.89*	*24.09*	*17.43*	*15.69*	*14.43*	*13.00*	*12.73*	*11.33*	*9.98*	*9.02*
Calculated Cutoff Frequency based on Cumulative Square Root of WUS PSD										
0.90	11.99	11.32	8.72	7.99	7.41	6.7	6.59	5.87	5.22	4.67
0.95	20.41	19.19	14.38	12.95	11.87	10.69	10.49	9.25	8.04	7.23
0.97	*31.29*	*29.26*	*21.05*	*19.04*	*16.95*	*15.15*	*15.07*	*12.78*	*10.80*	*9.72*
0.99	64.50	62.01	49.77	46.18	40.58	35.10	37.06	29.20	21.80	18.73

4.2 Assessment of Compatibility of Truncated Target PSD and Response Spectra

For each bin shown in Tables 1 and 2 of Appendix B, a sensitivity study was performed by progressively removing the PSD value at the highest frequency from the target PSD and generating the corresponding response spectra through averaging the response spectra of 100 time histories generated from the truncated target PSD curve. Figure 9 and Figure 10 show two representative comparisons of the resultant response spectra (dashed lines) and the NUREG/CR-6728 bin representative design response spectra (solid blue lines). The vertical

lines in these figures indicate the frequencies at which the PSD curves were truncated from higher frequencies. These figures can be used for the determination of upper bound (cutoff) frequencies for PSD check, together with other considerations such as the ZPA frequency of the design response spectra and the dynamic characteristics of the soil-structure-equipment system. The criterion is that removing all tabulated PSD values above the cutoff frequencies does not significantly affect the compatibility between the PSD and RS.

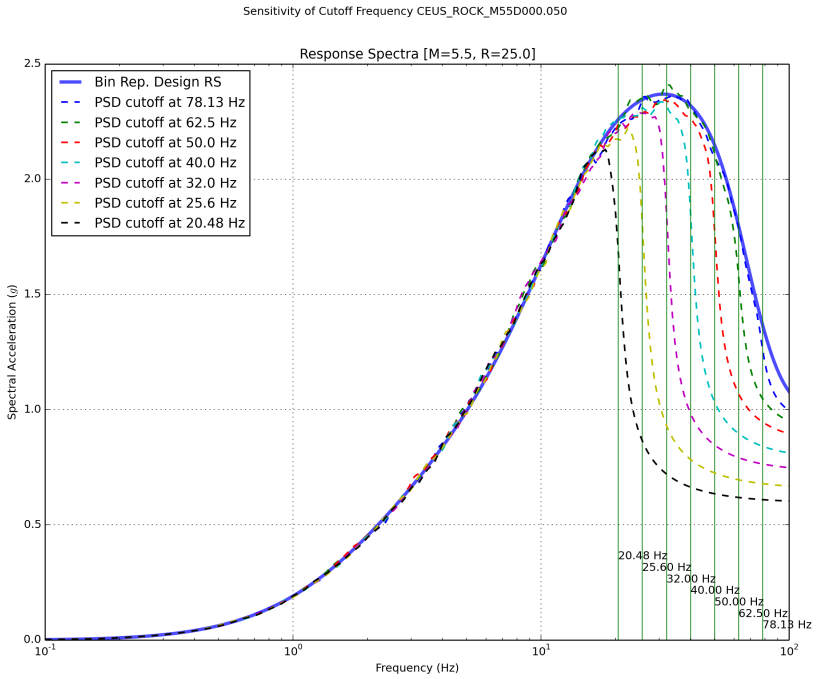

Figure 9 Sensitivity of Target PSD Cutoff Frequency for Table 1 CEUS ROCK M55D000.050

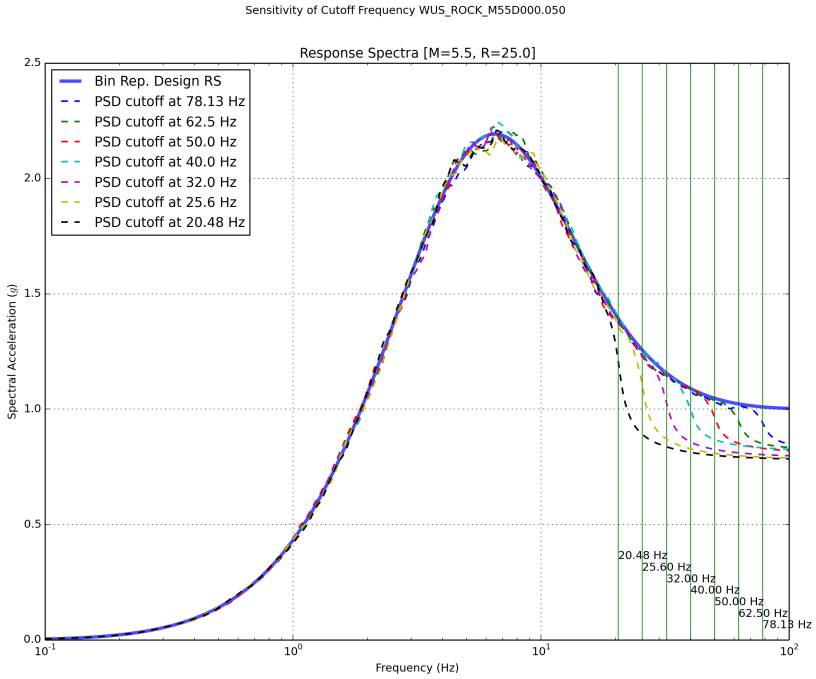

Figure 10 Sensitivity of Target PSD Cutoff Frequency for Table 2 WUS ROCK M55D000.050

5 RS and PSD Figures for the Development of SRP 3.7.1 Appendix B

5.1 PSD Formulation

This subsection describes the relationship between the PSD and the discrete Fourier transform of a random process, for use in the context of SRP 3.7.1 [e.g., Wirsching, et al, 1995, and Paz, 1989].

The two-sided power spectral density function $S_X(\omega)$ of a stationary, ergodic random process $X(t)$ is equal to the Fourier transform of the autocorrelation function, $R_X(\tau)$, which is related to the random process $X(t)$ as follows:

$$R_X(\tau) = \lim_{T\to\infty} \frac{1}{T}\int_0^T X(t+\tau)X(t)dt \tag{1}$$

The Fourier transform pair for the spectral density function is:

$$S_X(\omega) = \frac{1}{2\pi}\int_{-\infty}^{\infty} R_X(\tau)e^{-i\omega\tau}\, d\tau \tag{2}$$

$$R_X(\tau) = \int_{-\infty}^{\infty} S_X(\omega)e^{i\omega\tau}d\omega \tag{3}$$

For this transform pair shown in Equations (2) and (3) to exist, the autocorrelation function needs to be absolutely integrable: $\int_{-\infty}^{\infty}|R_X(\tau)|\, d\tau < \infty$. The limit of $R_X(\tau)$ as $\tau \to \infty$ is equal to the square of the mean of $X(t)$, μ_x^2 , and when the mean is nonzero, the integral does not converge. Therefore, the mean value of the process should be removed before the spectral analysis is performed. The spectral density function is real, symmetric and positive, and its area is equal to the mean-square value σ_x^2 of the random process.

$$E[X^2(t)] = \sigma_x^2 = \int_{-\infty}^{\infty} S_X(\omega)d\omega \tag{4}$$

where $E[\ ]$ denotes the mathematical expectation operator.

A more direct approach to computing the power spectral density can be accomplished by using the relationship with the Fourier transform of $X(t)$. It is shown by Wirsching, et al [1995], that the spectral density function corresponds to the ensemble average of the squared moduli of the Fourier transforms of $X(t)$, evaluated over a finite time duration *T* and scaled by an appropriate factor. Thus, the spectral density function may be estimated from a sufficient number of adequately long sample realizations of $X(t)$ as follows:

1. Obtain a number *m* of sample realizations of $X(t)$ having length T,
2. For each realization, compute its discrete Fourier transform and the square of its modulus,

3. Compute the average squared modulus frequency by frequency, where the averaging is done with respect to the number of sample realizations, *m*,
4. Divide by $2\pi T$ to obtain proper normalization and units.

In practical implementation, cycles per second (Hz) are often used in place of radians per second (rad/s), and negative frequencies are discarded. In order to maintain the same area for the integral shown in Equation (4), the spectral density function must be multiplied by 2π if the frequency unit is changed from rad/s to Hz, i.e., the two-sided spectral density function $S_X(f)$ in terms of f (Hz) is defined as:

$$S_X(f) = 2\pi S_X(\omega) \tag{5}$$

Similarly, the one-sided spectral density function $G_X(\omega)$, which discards the negative frequencies, is defined as twice the two-sided spectral density function $S_X(\omega)$ for positive frequencies, i.e.:

$$G_X(\omega) = \begin{cases} 2S_X(\omega) & \omega \geq 0 \\ 0 & \omega < 0 \end{cases} \tag{6}$$

The above Equation is the one-sided power spectral density function corresponding to Equation (1) of SRP 3.7.1 Appendix B. Combining Equations (5) and (6), the one-sided spectral density function $G_X(f)$ in terms of f (Hz) becomes:

$$G_X(f) = \begin{cases} 4\pi S_X(\omega) & \omega \geq 0 \\ 0 & \omega < 0 \end{cases} \tag{7}$$

To develop Equation (1) of SRP 3.7.1 Appendix B (as well as Appendix A), the strong motion portion of an acceleration time history, $a(t_j)$, where $t_j = j\,\Delta t$ and $j = 0, 1, \ldots N-1$, is used to calculate its discrete Fourier coefficients, as expressed by the following exponential form of the Fourier series:

$$\hat{F}(\omega_n) = \sum_{j=0}^{N-1} a(t_j) e^{-2\pi i\left(\frac{nj}{N}\right)} \tag{8}$$

and its inverse discrete Fourier transform is given by:

$$a(t_j) = \frac{1}{N} \sum_{n=0}^{N-1} \hat{F}(\omega_n) e^{2\pi i\left(\frac{nj}{N}\right)} \tag{9}$$

Where, for $0 \leq n \leq N/2$, the frequency ω_n increases with index n; for $N/2 < n \leq N-1$, ω_n represents the negative frequencies and $\hat{F}(\omega_n)$ is the complex-conjugate of $\hat{F}(\omega_{N-n})$.

The discrete Fourier coefficients $\hat{F}(\omega_n)$ need to be multiplied by Δt (the time increment in the acceleration time history) to get the Fourier amplitudes $|F(\omega)|$:

$$|F(\omega)| = |\hat{F}(\omega_n)\Delta t| \tag{10}$$

which is used to define the averaged, one-sided PSD $S_0(\omega)$, as shown in Equation (1) of SRP 3.7.1 Appendix B (as well as Appendix A):

$$S_0(\omega) = \frac{2|F(\omega)|^2}{2\pi T_D} \tag{11}$$

The frequency window used to compute the averaged one-sided PSD $S_0(\omega)$ at any frequency $f = \omega/2\pi$, is ±20% of f (e.g., 4 Hz to 6 Hz band width for f = 5 Hz).

To compute the PSD, the strong motion duration T_D in Equation (11) is taken herein as the time it takes the Arias energy to accumulate between 5% and 75% of the total energy in the record. The Arias energy is defined as:

$$E(t_1) = \frac{\pi}{2g}\int_0^{t_1} a^2(t)dt \tag{12}$$

where $a(t)$ is the acceleration time history, which in this case includes the entire record (i.e., rise, strong motion, and decay portions).

The T_D calculated with the help of Equation (12) should be confirmed to represent the duration of near maximum and nearly stationary power of an acceleration time history record, to be consistent with the guidance provided in Appendix B of NUREG/CR-5347.

5.2 Procedure for Development of Bin Average Response Spectra and PSD

NUREG/CR-6728 includes a time history database that provides a suite of time histories for structural and soil column analyses. Empirical time history records have been catalogued into magnitude and fault distance bins. These distance and magnitude bins are shown in the headers of Tables 1 and 2 of this SRP 3.7.1 Appendix B. The three moment magnitude bins are designated herein as “M55” (M 5-6), “M65” (M 6-7), and “M75” (M 7+). The four distance bins used for magnitudes “M65” and “M75” are designated as “D1” (0-10 km), “D2” (10-50 km), “D3” (50-100 km) and “D4” (100-200 km). The two distance bins associated with magnitude bin “M55” are designated as “D1” (0-50 km) and “D2” (50-100 km).

The procedure used to compute the bin average PSD and bin average response spectra (RS) is described in this subsection, using the acceleration time histories in the NUREG/CR-6728 database.

5.2.1 Calculation of PSD and RS

The PSD of an acceleration time history can be computed in the follow steps:

1. First, remove the mean from the time history;

2. Calculate the cumulative Arias energy and find the time points t_5 and t_{75} corresponding to the 5% and 75% of the cumulative Arias energy and compute the strong motion duration as the difference between these two time points, i.e., $T_D = t_{75} - t_5$;

3. Set the acceleration amplitudes outside the 5% and 75% range to 0;

4. Apply a Tukey window (also known as tapered cosine window) with a taper of 15% (7.5% each for the rise and the decay) on the strong motion;

5. Remove the mean from tapered strong motion;

6. Compute the Fourier coefficients for the strong motion using Equation (8) in Subsection 5.1;

7. Adjust the discrete Fourier coefficients to the Fourier amplitudes using Equation (10) in Subsection 5.1;

8. Compute PSD using Equation (11) in Subsection 5.1;

9. Smooth the PSD using a frequency window of ±20%.

The RS of an acceleration time history can be calculated using the entire record after removing the mean at Step 1 above.

5.2.2 Calculation of Bin Average PSD and RS

The record PSD and RS for each bin were normalized by utilizing the average PGA_{avg}, which was calculated as the geometric mean of the PGA's of all records in the subject bin. All PSDs were scaled by $(1/PGA_{avg})^2$ and all RS were scaled by $(1/PGA_{avg})$.

Additionally, many of the records in some bins have differing values for the time step and the record length; therefore, the PSDs may not be represented with the same frequency points and with the same upper frequency limit. In order to compute the bin average PSD, each record PSD calculated in Subsection 5.2.1 was interpolated at common frequency points (301 points evenly spaced in log scale from 0.1 Hz to 100 Hz).

The bin average PSD was calculated as the geometric mean of record PSDs. Similarly, the bin average RS was calculated as the geometric mean of the record RS. The bin average PSD and the bin average RS were then smoothed using cubic splines at the frequency points as shown in the first columns of Tables 1 and 2 of Appendix B, and the tabulated bin average PSD and bin average RS represent the smoothed values.

Tables 3 through 6 of SRP 3.7.1 Appendix B show the bin average PSD and Tables 7 through 10 present the bin average RS, for the CEUS and WUS rock site and soil site conditions in the NUREG/CR-6728 ground motion database. These tables can be used for assessing the site-specific ground motions developed for early site permit applications or combined license applications.

It should be noted that the bin average PSDs are generally not compatible with the bin average RS, as exemplified in Figure 11 through Figure 21 by the large difference between the bin average RS and the RS generated based on the bin average PSD, which were computed by averaging the RS of 100 synthetic acceleration time histories developed from the bin average PSD.

5.3 RS and PSD Figures

This subsection includes figures for some selected bins, which were used in the development of the tables for SRP 3.7.1 Appendix B. The four subsections below include RS and PSD figures for CEUS rock sites, CEUS soil sites, WUS rock sites, and WUS soil sites, respectively. Each figure corresponds to one bin in the NUREG/CR-6728 database, and the label at the top of the figure indicates the region, site condition, name of the bin (i.e., moment magnitude and fault distance), and number of events for that bin. Each figure has two plots: the top plot shows various RS curves and the bottom plot shows the PSD curves, which are explained below for the soil sites and rock sites separately.

For soil sites, the figures show the bin average PSDs and bin average RS. Each PSD plot includes three curves: (1) bin average PSD, (2) smoothed PSD using cubic spline with knots (the specified frequencies as shown in the tables in Appendix B, hereinafter referred to as tabulated frequencies), and (3) tabulated PSD that are points on the smoothed PSD at the tabulated frequencies. Each RS plot includes four curves: (1) bin average RS, (2) smoothed RS using cubic spline with knots at the tabulated frequencies, (3) tabulated RS that are points on the smoothed RS at the tabulated frequencies, and (4) RS which is generated from the bin average PSD using 100 artificial time histories. It can be seen that the bin average PSD based RS are generally not sufficiently close to the bin average RS, so the bin average PSD and bin average RS are generally not compatible for soil sites.

For rock sites, the figures show the bin average PSDs, the target PSDs, bin average RS, and bin representative NUREG/CR-6728 design RS. Each PSD plot includes four curves: (1) bin average PSD, (2) iterated target PSD, (3) smoothed PSD, and (4) tabulated PSD. Each RS plot also includes four curves: (1) bin average RS, (2) bin representative RS, (3) bin average PSD based RS, and (4) tabulated target PSD based RS. Similar to the case of soil sites, it can be seen that the bin average PSD based RS and bin average RS are generally not compatible for rock sites. The critical message in these figures is that the RS generated based on the tabulated target PSDs closely match the bin representative NUREG/CR-6728 design RS, demonstrating their compatibility. For tabulated target PSDs that require minor manual adjustments at a few very low frequencies and/or very high frequencies, the corresponding figures (as identified in their caption by “Adjusted”) show that the adjustments do not have noticeable effect on the level of agreement between the tabulated PSD based RS and the bin representative RS.

5.3.1 RS and PSD for CEUS Rock Sites

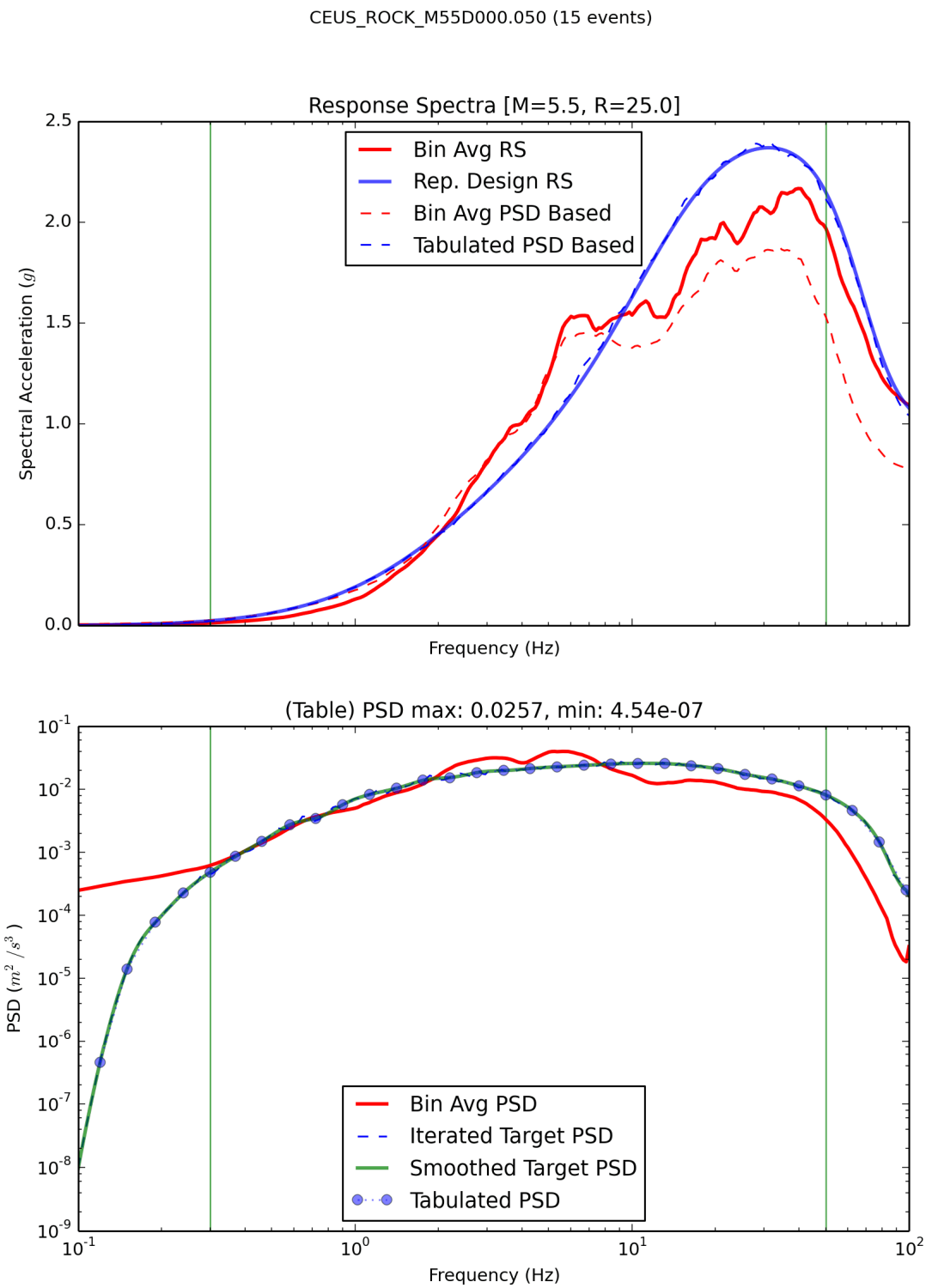

Figure 11 RS and PSD for CEUS Rock M55D000.050

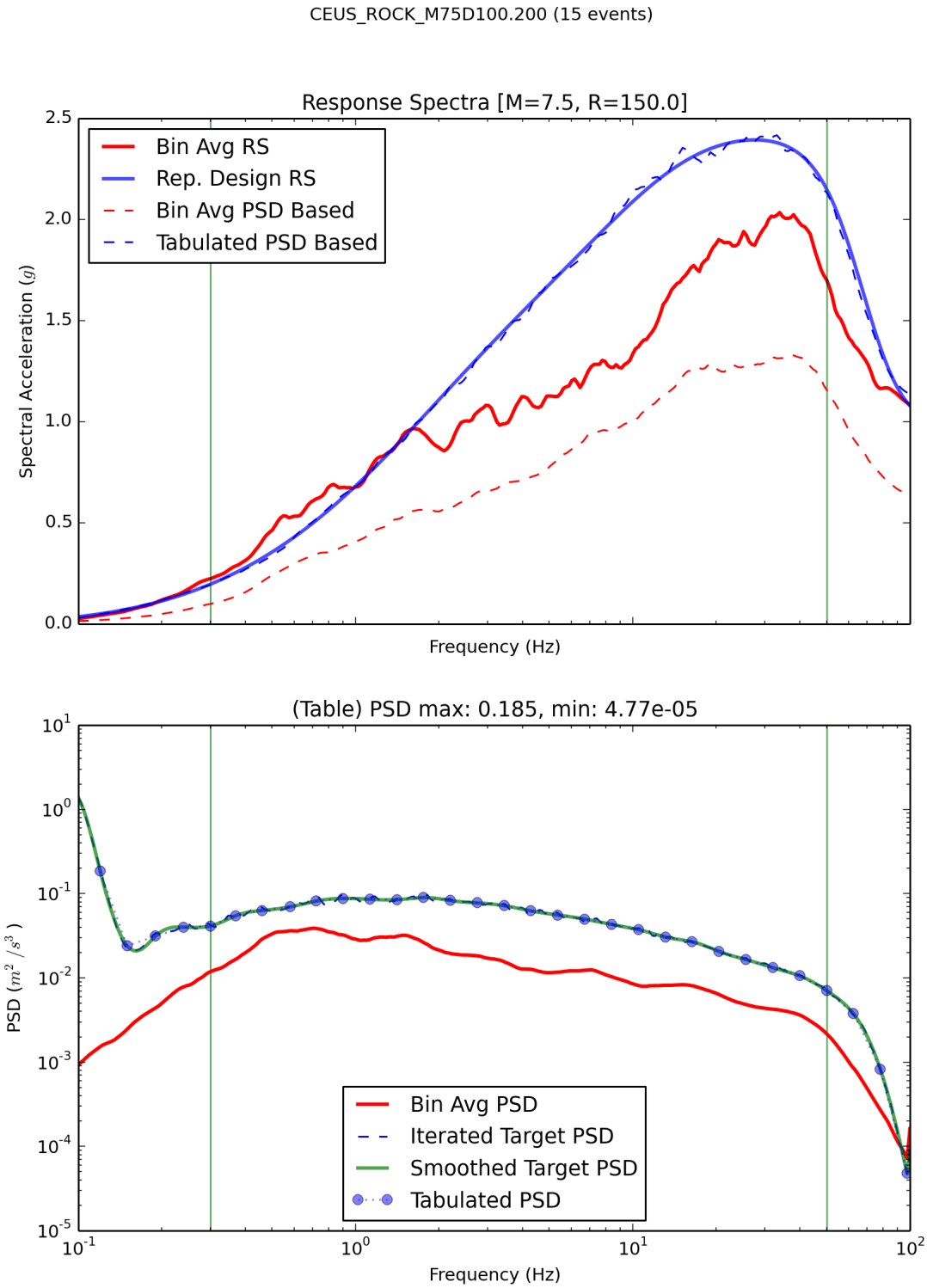

Figure 12 RS and PSD for CEUS Rock M75D100.200

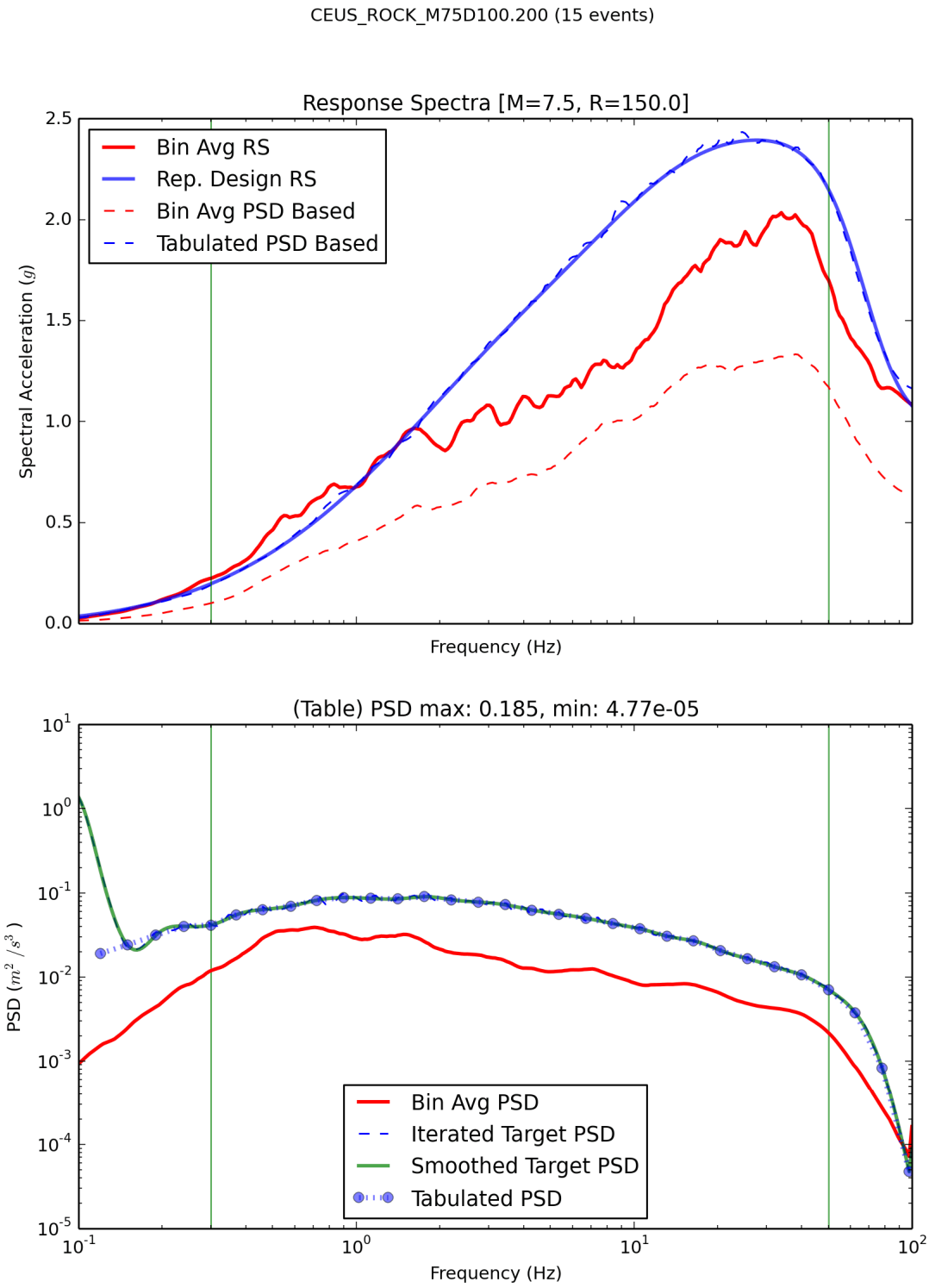

Figure 13 RS and PSD for CEUS Rock M75D100.200 (Adjusted)

5.3.2 RS and PSD for CEUS Soil Sites

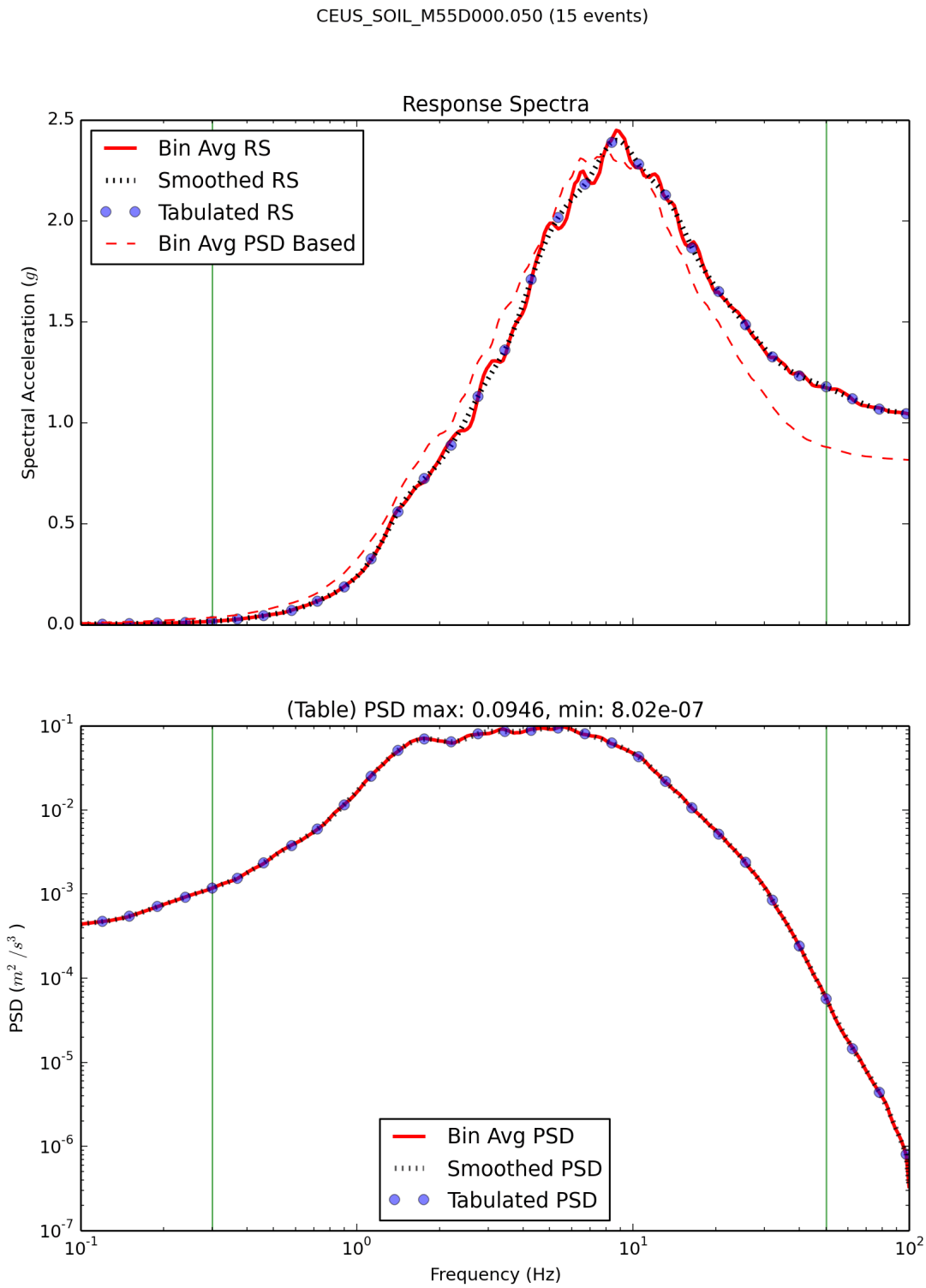

Figure 14 RS and PSD for CEUS Soil M55D000.050

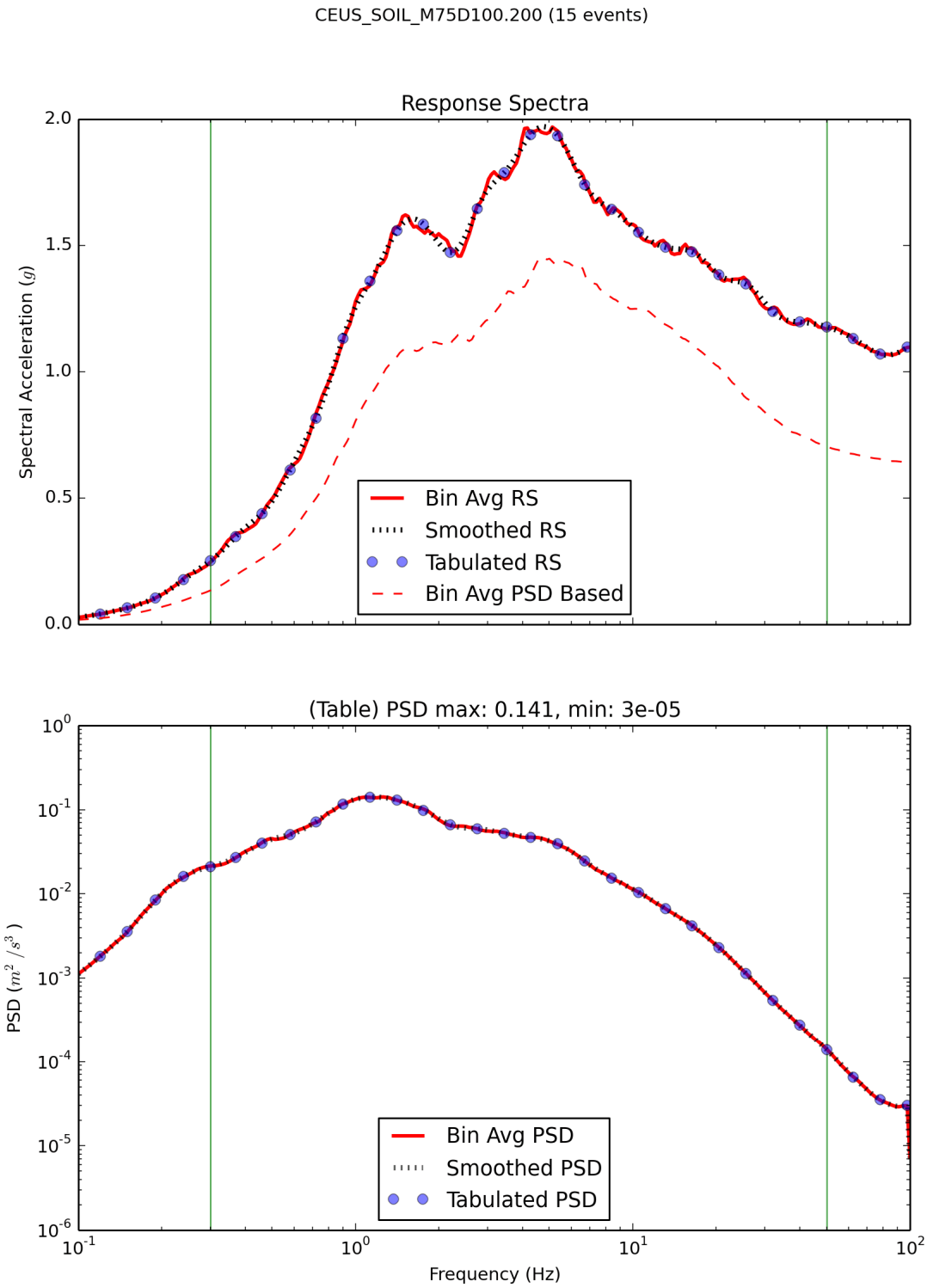

Figure 15 RS and PSD for CEUS Soil M75D100.200

5.3.3 RS and PSD for WUS Rock Sites

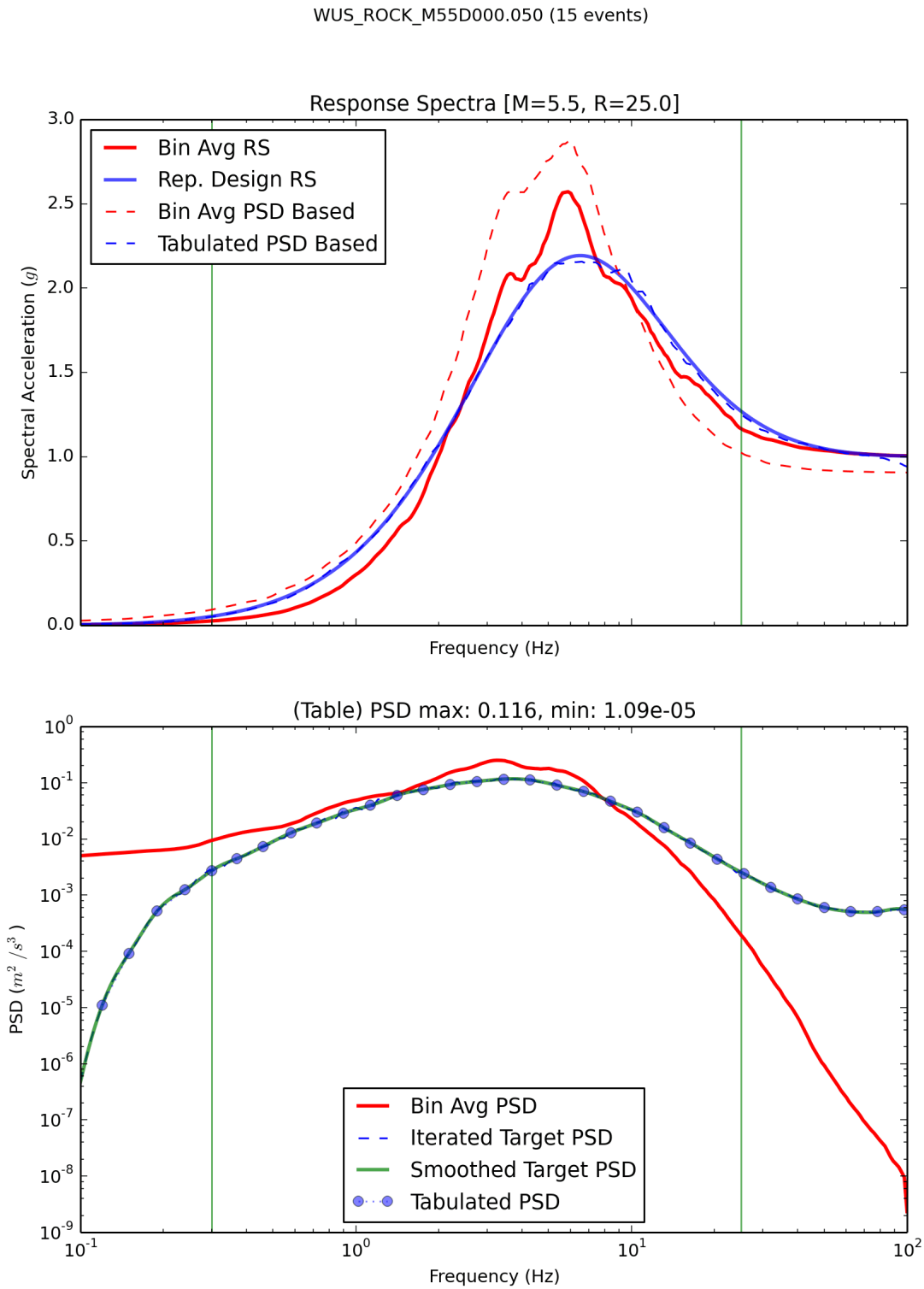

Figure 16 RS and PSD for WUS Rock M55D000.050

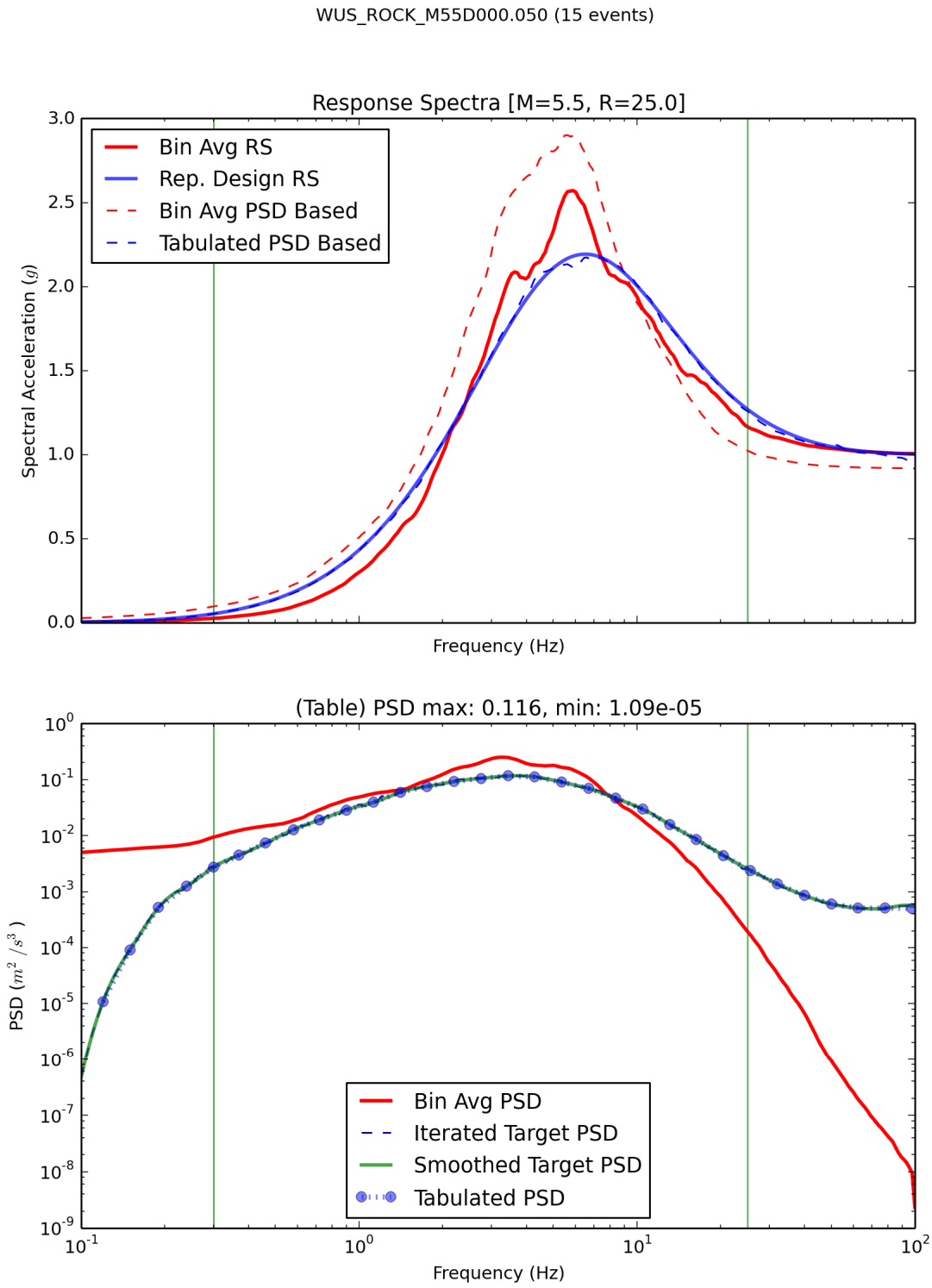

Figure 17 RS and PSD for WUS Rock M55D000.050 (Adjusted)

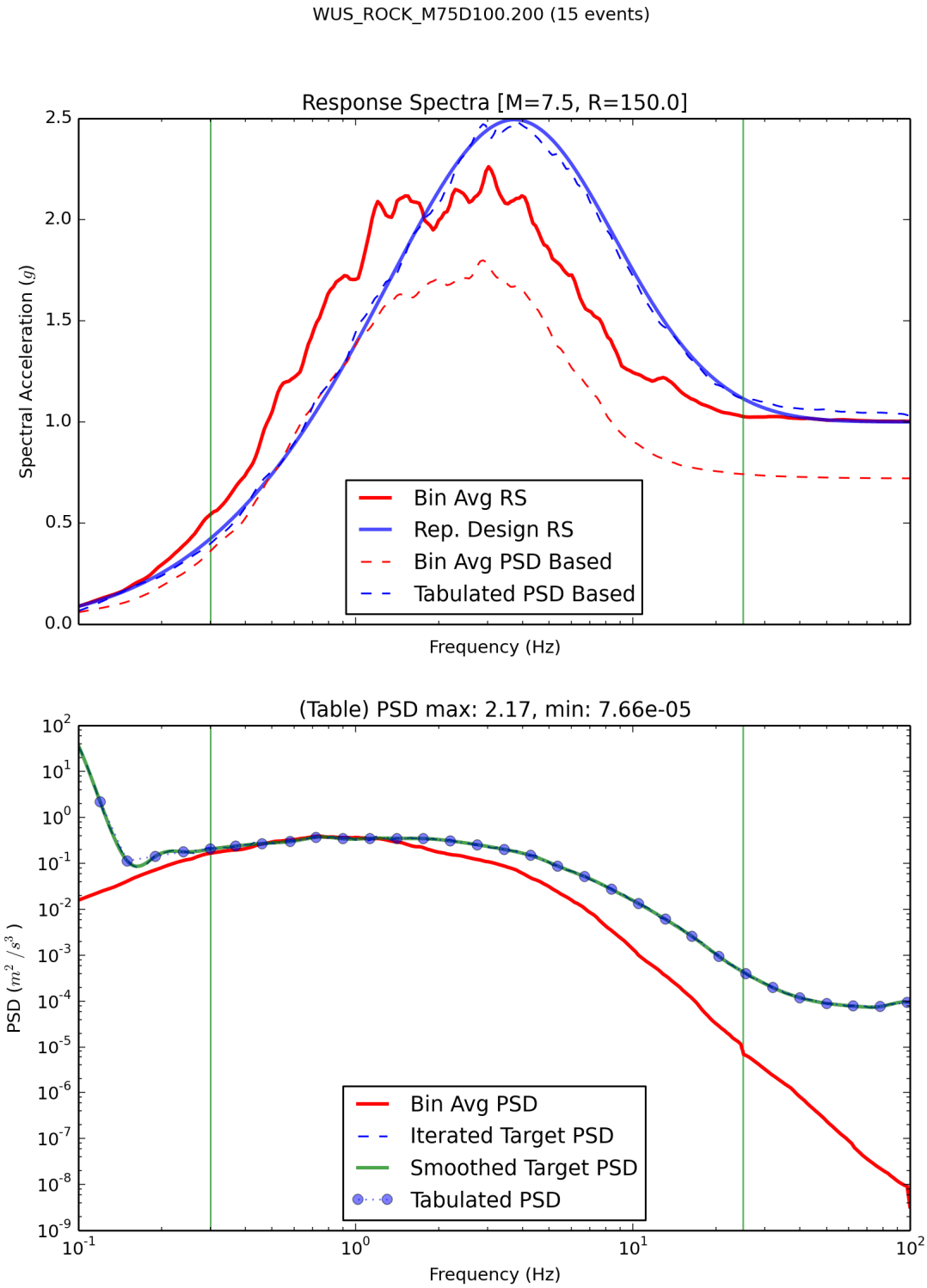

Figure 18 RS and PSD for WUS Rock M75D100.200

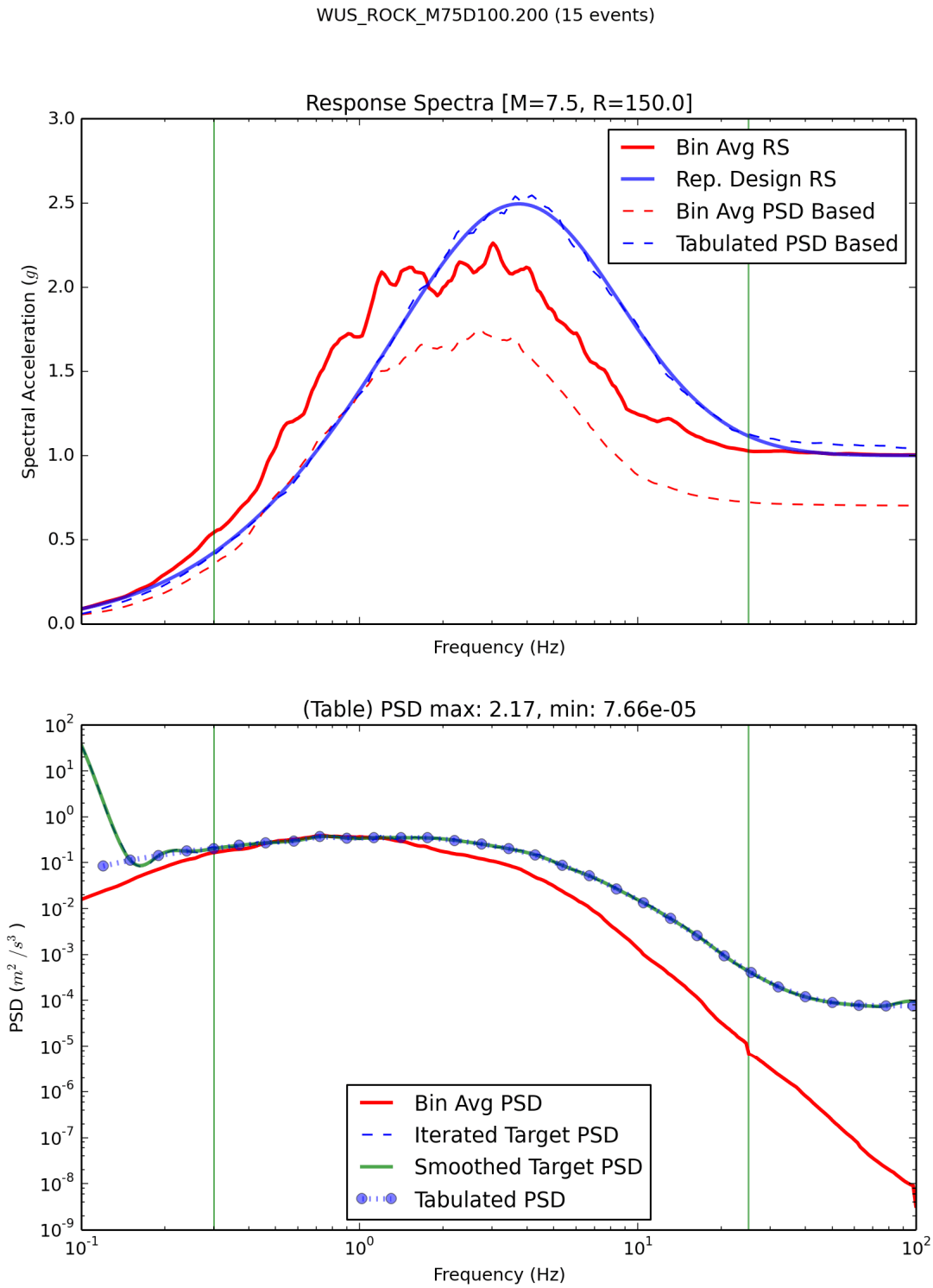

Figure 19 RS and PSD for WUS Rock M75D100.200 (Adjusted)

5.3.4 RS and PSD for WUS Soil Sites

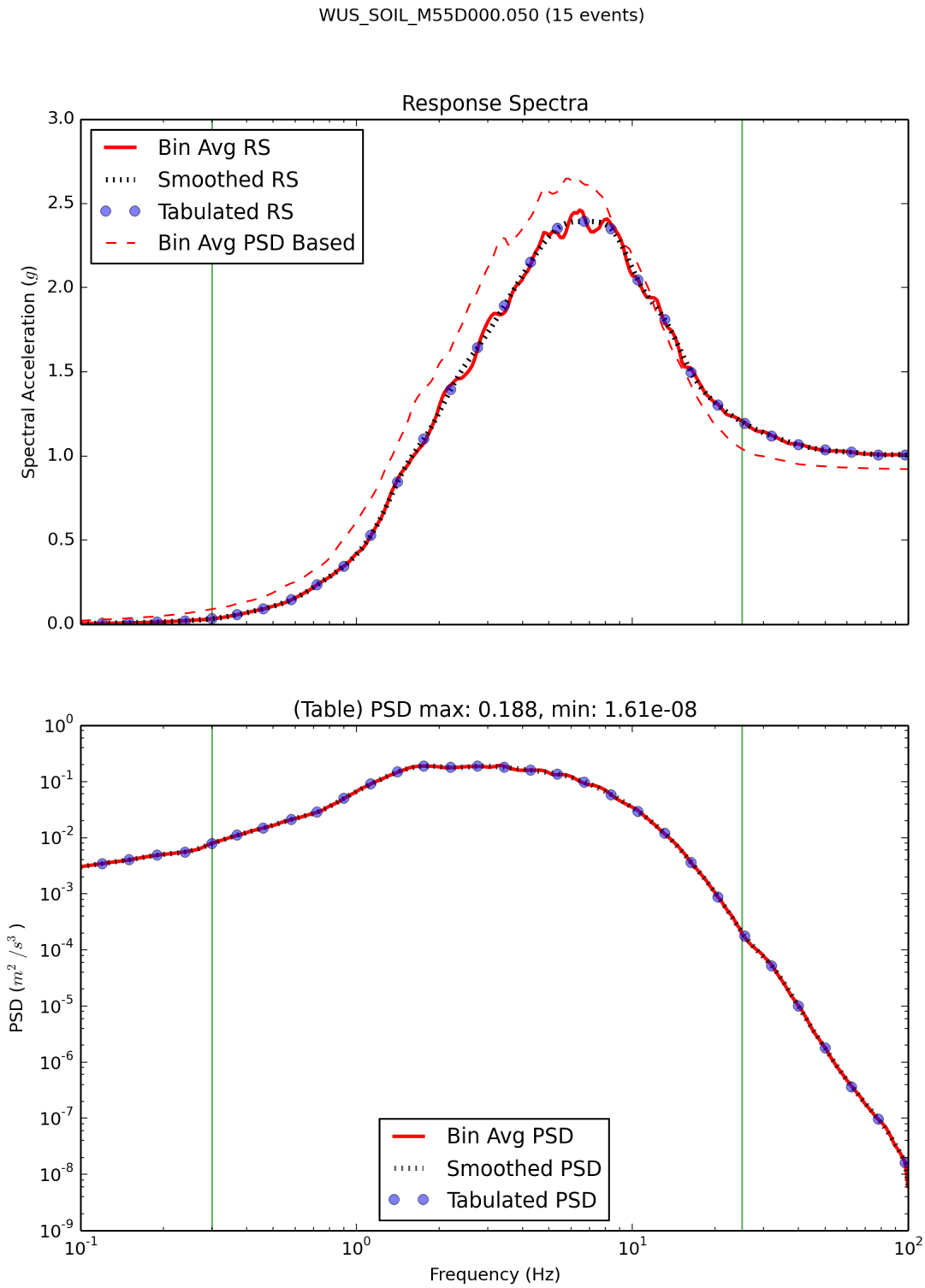

Figure 20 RS and PSD for WUS Soil M55D000.050

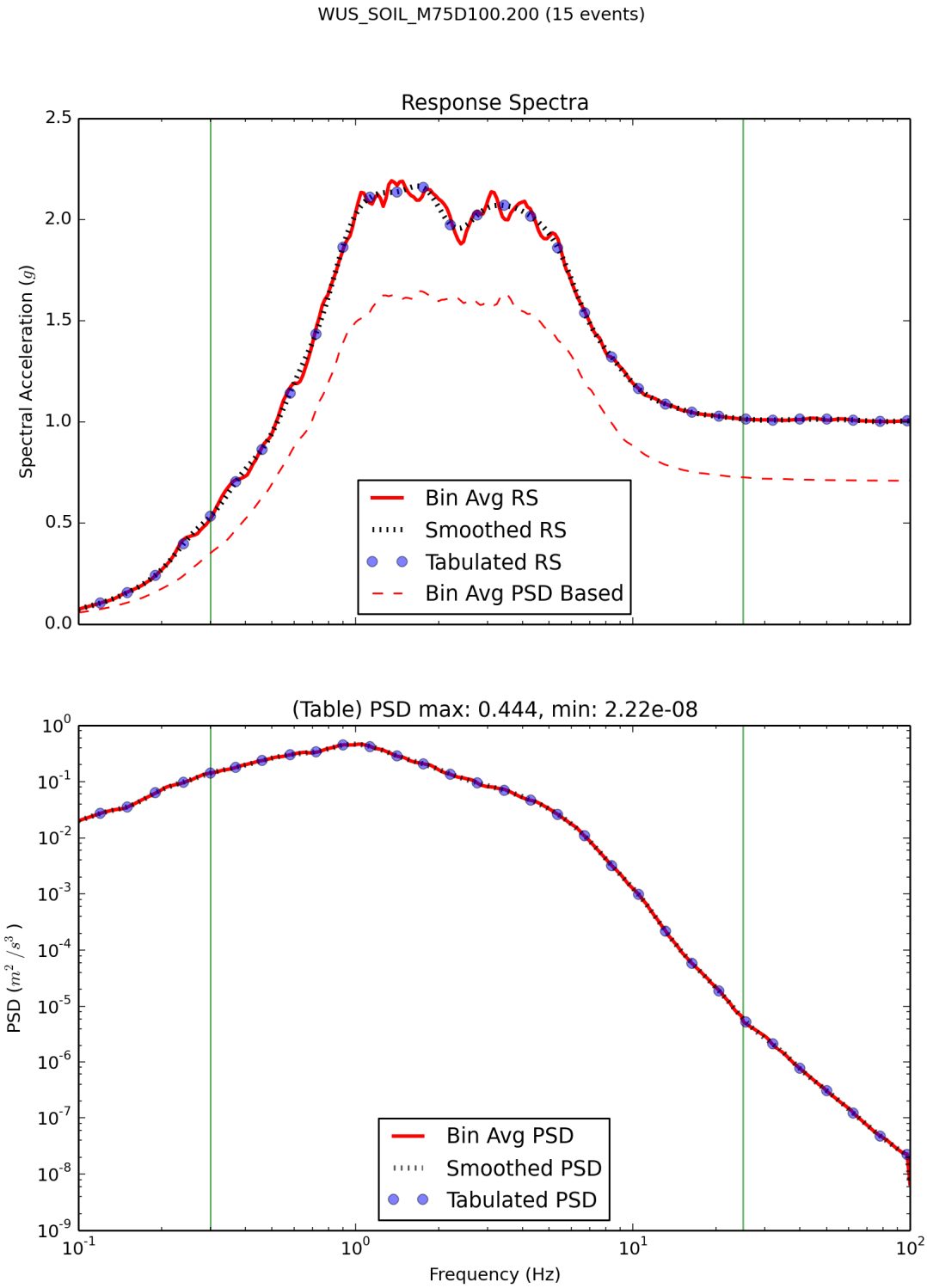

Figure 21 RS and PSD for WUS Soil M75D100.200