February 19, 2013

MEMORANDUM TO: Mohammed A. Shuaibi, Deputy Director
Division of Engineering
Office of New Reactors

FROM: Jim Xu, Acting Chief
Structural Engineering Branch 2 **/RA/**
Division of Engineering
Office of New Reactors

SUBJECT: TECHNICAL REPORT IN SUPPORT OF STAFF'S ACTIVITY TO ENHANCE GUIDANCE FOR REVIEW OF SEISMIC AND CIVIL STRUCTURAL ISSUES

Transmitted is a report entitled "Technical Rationale for Proposed Enhancements to Seismic and Structural Review Guidance." This report addressed technical issues related to the standard review plan (SRP), Sections 3.7 and 3.8 covering the subject of seismic and civil structural designs. The report was prepared as part of the staff's activity for enhancing guidance to support technical reviews of new reactor applications.

Enclosure:
As stated

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Technical Rationale for Proposed Enhancements to Seismic and Structural Review Guidance

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Enclosure

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 last major revisions to these SRP sections were completed in 2007. Since then, the staff has used these sections to complete the review of several 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.

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 a result of the review of recent DC and COL applications under 10 CFR Part 52, the staff has identified a number of significant issues related to seismic analysis and structural design. To a large extent, the issues are related to the need to develop a standard (generic) plant design that can be located at multiple sites in the United States. These issues have 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 facilitate the review process for future applications.

Objective

The objective of this report is to (1) identify the key design challenges that arose from the seismic and civil structural reviews of new reactor applications under the Part 52 process, (2) to describe the underlying technical issues, (3) to propose enhanced criteria for use in SRP sections related to these issues, and (4) to provide a technical rationale for the proposed enhancements to SRP acceptance criteria. More specifically, the report describes eleven key issues related to seismic analysis and structural designs identified during the staff’s review of recent licensing applications; explains why the SRP should be enhanced in these areas; presents proposed revisions to the SRP; and then provides the technical basis and/or rationale for the proposed revisions. 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 the effort to enhance SRP Sections 3.7 and 3.8 is to address the more significant technical issues associated with the technical challenges identified during the licensing review process of recent DC and COL applications. Eleven technical issues determined to be important to this effort, were identified, and are listed below:

1. Seismic uplift in soil-structure interaction (SSI) analysis
2. Seismic stability evaluation for design of structures
3. Interaction of non-Category I structures with Category I SSCs
4. Seismic soil pressure on embedded walls
5. Ground motion incoherency effect on seismic 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

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: the issue, the reason why the SRP should be enhanced, proposed enhancements, and the technical rationale for proposing them.

Technical Issue No. 1

Seismic Uplift in 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 Current Criteria

SRP Section 3.7.2 II.4 indicates 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 provides guidance on how to define the dead load for uplift evaluations, including the treatment of the stored water in internal water pools inside the structures. Other SRP Section 3.8.5 subsections provide guidance on seismic stability evaluations, foundation design, and maximum toe bearing pressures.

2.2 Why Current Criteria Are Not Adequate

The SRP does not provide specific guidance on permissible levels of foundation uplift to be considered in the SSI analysis and the foundation design.

3. Proposed Revision to SRP Section 3.7.2

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

4. Soil-Structure Interaction A complete SSI analysis should properly account for all effects due to kinematic and inertial interaction 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.

   B. 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 subsection 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 assessed, and if found *important, then it 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 internal water 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 proposed SRP revision is 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.

The proposed SRP revision 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 proposed in the SRP revision 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]).

The proposed SRP revision 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.

The proposed revision 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 a proposed revision 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 current SRP acceptance criteria 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 in SRP Section 3.8.5 should be developed.

2. Why SRP Section 3.8.5 Should Be Revised

2.1 Current Criteria

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 should be 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” state 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 also need 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” provide the factors of safety for the five load combinations defined in SRP Section 3.8.5 II.3.

2.2 Why Current Criteria Are Not Adequate

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 needs to be 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. Proposed Revision 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” should be enhanced to provide additional guidance to the issues associated with seismic stability evaluation. The applicable portion of this SRP section and the proposed enhancements (highlighted in italics) are given below.

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

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. 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*~~ *of* the analysis *results* ~~adequately~~ *to* account~~*s*~~ for potential foundation mat liftoff effects.~~, *if appropriate?*~~ The ~~*method to*~~ 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, *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. 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, the pair of each horizontal force with the 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:

1. 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.

2. To demonstrate the 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.

3. 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.

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

5. If the input motion applied at the foundation of the mathematical model is developed from the response of the linear SSI analysis, justification is needed to demonstrate that any minimal sliding or uplift that might occur would not affect the assumed seismic input motion which was 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.

6. 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.

7. 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 Proposed 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 Proposed 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 does 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 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 1:

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 SRP Section 3.7.1 already provides guidance in this area and the proposed revisions to SRP Section 3.7.1 further enhance this guidance, a reference to SRP Section 3.7.1 addresses these items.

4.4 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 2:

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 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 3:

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

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 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 5:

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 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 6:

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 Proposed Enhancement to SRP Section 3.8.5 II.4.B, Item 7:

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-Category I Structures with Category I SSCs

1. Description of Issue

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

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

One of the methods – (approach C) states: “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.” has been recognized to be overly conservative for the design of non-Category I structures, since it apparently invokes the same design criteria as is applicable to Category I structures. While adherence to Category I design criteria is acceptable, the criteria associated with approach C should be enhanced to reduce the inherent conservatism.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Current Criteria

SRP Section 3.7.2.8, approach C, states, “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 Current Criteria Are Not Adequate

The seismic II/I criteria are intended to ensure that seismic interaction between the non-Category I structures and 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, is overly restrictive because it imposes the margin of safety for the non-category structures to be equivalent to that of Category I structures. Therefore, the current criterion is overly conservative and should be enhanced.

3. Proposed Revision to SRP Sections 3.7.2 and 3.7.3

3.1 Proposed Revision to SRP Section 3.7.2

The applicable portion of SRP Section 3.7.2 II.8.C and the proposed enhancements (highlighted in italics) are shown below.

C. 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 disposition of each non-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-Category I structure is acceptable. This should include a description of any additional loads imposed on the 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 Category I SSCs should be described.

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

A conservative way to address criterion C is to apply a linear elastic analysis to the non-Category I structure, similar to Category I structures. However, depending on the magnitude of the gap between the non-Category I structure and the adjacent Category I SSCs, a limited inelastic response may be permissible for the non-Category I structure, provided the structural integrity can be demonstrated, to ensure no physical interaction between the non-Category I structure and all adjacent Category I SSCs. In the assessment, the effect of structure-soil-structure interaction (SSSI) 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-Category I structures. Consequently, DC applicants should provide sufficient analysis and design information concerning interaction of the non-Category I Structures with 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-Category I Structures with 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-Category I structures) should be identified.

3.2 Proposed Revision to SRP Section 3.7.3

The applicable portion of SRP Section 3.7.3 II.8 and the proposed enhancements (highlighted in italics) are shown below.

8. Interaction of Other Systems With Seismic Category I Systems. To be acceptable, each non-seismic Category I system should be designed to be isolated from any seismic Category I system by either a constraint or barrier, or should be remotely located with regard to the seismic Category I system. If *~~it~~ this* is not feasible or practical*, ~~to isolate the seismic Category I system,~~ then* adjacent non-seismic Category I systems should be analyzed according to the same seismic criteria as applicable to the seismic Category I system. For non-seismic Category I systems attached to seismic Category I systems, the dynamic effects of the non-seismic Category I systems should be simulated in the modeling of the seismic Category I system. The attached non-seismic Category I systems, 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 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-Category I structure to Category I SSCs.

For example, if a non-Category I structure is in proximity to a Category I structure, the absolute sum of the seismic displacements of the two structures needs to be less than the as-designed gap between the structures for the entire height of the structures to satisfy criterion C.

If the structures are very close, it may be necessary to apply Category I structural design criteria to the non-Category I structure to satisfy criterion C.

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

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 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-Category I 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. 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 current guidance in the Standard Review Plan (SRP) should be enhanced 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 should also be provided 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 Current Criteria

SRP Sections 3.8.1 II.4.E and 3.8.4 II.4.H indicate 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 Current Criteria Are Not Adequate

The two approaches for computing seismically induced lateral soil pressures that are currently described as acceptable in SRP Sections 3.8.1 II.4.E and 3.8.4 II.4.H have limitations that need to be more explicit.

The limitations of Wood's method described in ASCE 4-98 Section 3.5.3.2 need to 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 may need to 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 should also provide guidance on the review of the analysis assumptions for the methods that the staff considers acceptable.

<u>3. Proposed Revision to SRP Sections 3.8.1 and 3.8.4</u>

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 proposed enhancements (highlighted in italics) are shown below.

SRP Section 3.8.1 II.4.E

E. <u>Dynamic Soil Pressure</u>. 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 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) lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated using an embedded SSI/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 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 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 also be considered in the application of 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 – sinϕ, where ϕ 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 proposed 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 proposed 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

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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 notes 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 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 staff-accepted methodology (per DC/COL-ISG-01) to incorporate incoherency effects in seismic response analyses of structures.

The proposed enhancement to SRP Section 3.7.2 incorporates 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 proposed 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 Current Criteria

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 Current Criteria Are Not Adequate

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 does 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. Proposed Revision to SRP Section 3.7.2.

SRP Section 2.7.2 II.4, under "Specific Guidelines for SSI Analysis," ninth bullet, which is given below, should be revised with 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 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. 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 NRC issued Interim Staff Guidance (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 percent reduction
30 Hz and above – 30 percent reduction
10 to 30 Hz – reduction based on linear variation between 0% at 10 Hz and 30 percent 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 Design Certification 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 proposed 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, the Standard Review Plan (SRP) guidance should be enhanced 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 Current Criteria

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 The acceptance criteria in SRP Section 3.7.2 II.9 "Effects of Parameter Variations on Floor Response Spectra" indicate 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 are presented in subsection II.5 of SRP Section 3.7.2. The SRP criteria also indicate that for concrete structures, the effect of potential concrete cracking on the structural stiffness should be specifically addressed.

2.1.2 The review procedures in SRP Section 3.7.2 III.9 "Effects of Parameter Variations on Floor Response Spectra" have similar information as SRP Section 3.7.2 II.9 which indicates that among the various structural parameters analyzed, the effect of potential concrete cracking on structural stiffness should be 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 Current Criteria Are Not Adequate

The SRP is intended to develop 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 current criteria in SRP Section 3.7.2 only identify the need to consider the effects of concrete cracking on the structural stiffness with respect to development of floor response spectra. The criteria do not provide specific guidance on how to accomplish that. In addition, the criteria do 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 current 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,” indicate that cracking of concrete for containments is expected and discuss the effects of concrete cracking on stiffness, frequencies, and member forces. Therefore, the criteria conclude that the effects of concrete cracking need to be considered if significant. The criteria indicate 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 is 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,” refer 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 address the effects of cracking with respect to design of concrete structures. They do not address how to treat cracking when seismic soil structure interaction analyses are performed.

3. Proposed Revision to SRP Section 3.7.2

3.1 The criteria for considering concrete cracking in the seismic analysis of concrete structures in SRP Section 3.7.2 II.3.C “Modeling of Structures” should be 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., in-structure response spectra (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 *stiffnesses 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 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 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 and the associated SSE damping values in Table 1 of Regulatory Guide (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 best estimates 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 needs to be provided to demonstrate that the best estimate stiffness properties used for concrete are appropriate and that uncertainty associated with the best estimate 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 reflects 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.E and J indicate 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.

Further enhancement of SRP Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5 is necessary 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 Current Criteria

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 describe the loads and load combinations applicable to the design of seismic Category I foundations.

2.1.2 SRP Section 3.8.5 II.4.E indicates 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.J indicates 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 provides criteria for reviewing testing and in-service surveillance provisions for seismic Category I foundations.

2.2 Why Current Criteria Are Not Adequate

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 do 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 do 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 does not fully specify the construction sequence. In addition, the acceptance criteria do 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 does 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.

<u>3. Proposed Revision to SRP Sections 3.8.1, 3.8.3, 3.8.4, and 3.8.5</u>

3.1 The applicable portions of SRP Sections 3.8.1 I.3, 3.8.3 I.3, 3.8.4 I.3, and 3.8.5 I.3, shown below, should be 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 proposed 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 proposed 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 proposed 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 proposed 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" should be 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.

The specific acceptance criteria and review procedures are contained in the referenced SRP sections.

3.3 SRP Section 3.8.5 II.4 should be 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 base~~-~~mat 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 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.

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.*

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

- *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 such as a Nuclear Island basemat. 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 should be enhanced by the following additional statements (highlighted in italics):

For Category I foundations, it is important to accommodate inservice inspection of critical areas. The staff considers monitoring and maintaining the condition of 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 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 proposed SRP revisions 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 proposed 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 proposed 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 proposed 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 proposed 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

The Standard Review Plan (SRP) guidance for developing design time histories for soil structure interaction (SSI) analyses should be clarified and enhanced 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 Current Criteria

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

2.1.1 The criteria indicate 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 are 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 states:

> "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 states:

> "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, states:

> "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 are reviewed on a case-by-case basis."

2.2 Why Current Criteria Are Not Adequate

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. 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 [1]. 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 provides two alternative options for demonstrating that there are no significant gaps in energy at any frequency. The process involves 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 current criteria in the SRP do 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 do 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 is needed.

3. Proposed Revision to SRP Section 3.7.1

3.1 The applicable portion of SRP Section 3.7.1 II.1.B and the proposed 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 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 real 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 real earthquake records should be preserved. 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 real 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.*

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

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

ii. Approach 2. For Approach 2, the artificial ground motion time histories …

(d) ~~*In lieu of the power spectrum density requirement of Approach 1, the*~~ 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. *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 needs to be computed and shown to not have significant gaps in energy at any frequency over this frequency range.

~~*Artificial*~~*If the artificial* ground motion time *history,*~~*histories*~~ defined *in Approach 2*~~*as described*~~ *above, is intended to be compatible to a site-specific GMRS, it* shall 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, should be enhanced with the following additional guidance:

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. 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 need 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 requirements 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 need to 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 overprediction 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.

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 proposed 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.

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 the 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, the 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. To address this enhancement, proposed updates to SRP Section 2.0 and SRP Section 3.7.2 have been developed.

2.1 Why SRP Section 2.0 Should Be Revised

2.1.1 Current Criteria

Appendix A, Table 1, of SRP Section 2.0, Revision 0 (March 2007,) identifies site parameters that should be provided in a DC or combined license (COL) application.

2.1.2 Why Current Criteria Are Not Adequate

Based on the experience gained during DC application reviews, there are several important site parameters that are not identified in Appendix A, Table 1: minimum dynamic bearing capacity, lateral soil variability, soil angle of internal friction. These parameters are geotechnical properties inherent in site material characteristics critical to SSI analysis, foundation design and stability analysis.

2.2 Why SRP Section 3.7.2 Should Be Revised

2.2.1 Current Criteria

There is currently no guidance in the 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.

2.2.2 Why current criteria are not adequate

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.

3.1 Proposed Revision to SRP Section 2.0

Augment Appendix A, Table 1, to include the following additional row entries (highlighted in italics):

APPENDIX A

TABLE 1: EXAMPLES OF SITE CHARACTERISTICS AND SITE PARAMETERS

Site Characteristic / Parameter	SRP Location	ESP/COL Site Characteristic	DC Site Parameter
Minimum Dynamic Bearing Capacity	2.5.4	✔	✔
Lateral Soil Variability	2.5.4	✔	✔
Soil Angle of Internal Friction	2.5.4	✔	✔

3.2 Proposed Revision to SRP Section 3.7.2

Insert the following statement 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 the selected profiles are adequate for the postulated site conditions.*

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

- ~~*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 GMRS fall below the standard plant CSDRS, 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.*

Insert 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.

SRP 2.0, Appendix A, Table 1, provides a list of site characteristics and site parameters to be defined in a DC application. However, during the staff's DC application reviews, additional site parameters were identified that are important to controlling facility response; namely (1) lateral soil variability, (2) minimum dynamic bearing capacity, and (3) soil angle of internal friction.

The generic SSI analyses performed to establish the seismic demands for a standard plant design typically assume the site strata have uniform properties in the horizontal (lateral) direction. To ensure that this underlying assumption in the generic SSI analyses is valid for a specific site, the soil lateral variability should be determined for the site. This is done by carrying out geotechnical investigation of the site such that the actual lateral variability of the site strata is characterized. It is noted that lateral soil variability can be significant for soil sites and for soil strata overlaying rock sites. When the site structures are directly supported on hard rock where little SSI effects occur, the site can be considered effectively uniform for determining the seismic response. For intermediate or soft rock bearing conditions, the issue of lateral variability needs to be evaluated on a case-by-case basis.

The dynamic bearing capacity and soil angle of internal friction are intrinsic geotechnical properties of the soils that are important to foundation design and stability evaluation. For a standard design based on a generic site, the minimum dynamic bearing capacity and soil angle of internal friction are postulated for the design and analysis of the foundations. Therefore, specifying these site parameters in Table 1 facilitates the staff's reviews of DC and COL applications.

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 GMRS 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, the Standard Review Plan (SRP) Section 3.7.2 guidance on SSI analysis should be enhanced to provide additional criteria for reviewing SSI analysis of embedded structures performed using the SM.

2. Why SRP Section 3.7.2 Should Be Revised

2.1 Current Criteria

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

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

> “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,” provide a brief description of the half space or substructure approach, and also states 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 Current Criteria Are Not Adequate

The criteria in SRP Section 3.7.2 II.4 do 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 need to be performed to ensure that the SM leads to adequate results.

3. Proposed Revision to SRP Section 3.7.2

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

B. Modeling of Supporting Soil. 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, 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. 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.

3.2 The criteria for "Half Space or Substructure Solution Technique" in SRP Section 3.7.2 II.4 should be 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.

(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 connected to the free-field system, the excavated soil volume may not have compatible motions with the part of the free-field being replaced, especially at frequencies higher than the fundamental frequency of the excavated soil volume. This may lead to limitations in the application of the SM and potential errors if the method is not implemented appropriately.

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 is unaffected 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 proposed enhancements to SRP Section 3.7.2 II.4 is as follows:

- The proposed 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 proposed 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. Appendix D to Standard Review Plan (SRP) Section 3.8.4, "Guidance on Spent Fuel Pool Racks," provides the only guidance that directly addresses fuel assembly integrity in the fuel storage pools during a design basis seismic event. Appendix D states, "It should be demonstrated that the consequent loads on the fuel assembly do not lead to damage of the fuel." However, Appendix D does not provide any detailed guidance to the definition of "damage" or guidance on how to perform the review.

Appendix D does 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 is 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 needs 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 Current Criteria

The statement "It should be demonstrated that the consequent loads on the fuel assembly do not lead to damage of the fuel" is the only guidance provided in Appendix D 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 are not provided for spent fuel racks.

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

2.2 Why Current Criteria Are Not Adequate

Appendix D does not provide sufficient guidance on the definition of "damage" or the detailed process for performing the review. Therefore, this issue is 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 is needed to ensure that the guidance is equally applicable to both re-racking and new plant designs.

3. Proposed Revision to SRP Section 3.8.4, Appendix D

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

I. INTRODUCTION

Regulatory Guide 1.29, "Seismic Design Classification" classifies spent fuel pool racks as 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. <u>Applicable Codes, Standards, and Specifications</u>

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. <u>Seismic and Impact Loads</u>

~~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 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 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 (RSA) 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 will be 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. NUREG/CR-5912 provides further guidance on the design and analysis of free-standing fuel racks.

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

~~6.~~*3.* Regulatory Guide1.124, "Service Limits and Loading Combinations for Class 1 Linear-Type Component Supports."

~~7.~~*4*. American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section III, Division 1.

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

~~9.~~*6.* American Society of Civil Engineers, Suggested Specification for Structures of Aluminum Alloys 6061-T6 and 6067-T6.

~~10.~~*7.* The Aluminum Association, Specification for Aluminum Structures.

~~11.~~*8.* Biggs, John M., Introduction to Structural Dynamics, McGraw-Hill Book Co., New York, 1964.

~~12.~~*9.* 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 in Appendix D, which is 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 proposed addition to the first paragraph of Appendix D should clarify the staff’s position.

Appendix D, as currently written, provides 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 wording of Appendix D needs updating 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-lead, Christopher VanWert, Brian Thomas), Office of Naval Research Reactor (NRR) (William Jessup), Office of Nuclear Material Safety and Safeguards (NMSS) (Bhasker Tripathi), and Office of Nuclear Regulatory Research (RES) (Jose Pires).