ArevaEPRDCPEm Resource

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Subject: Draft - U.S. EPR Design Certification Application RAI No. 376 (4355,4367,4377), FSAR Ch. 3
Attachments: Draft RAI_376_SEB2_4355_4367_4377.doc

Attached please find draft RAI No. 376 regarding your application for standard design certification of the U.S. EPR. If you have any question or need clarifications regarding this RAI, please let me know as soon as possible, I will have our technical Staff available to discuss them with you.

Please also review the RAI to ensure that we have not inadvertently included proprietary information. If there are any proprietary information, please let me know within the next ten days. If I do not hear from you within the next ten days, I will assume there are none and will make the draft RAI publicly available.

Thanks,
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Request for Additional Information No. 376 (4355, 4367, 4377), Revision 0

3/11/2010

U. S. EPR Standard Design Certification
AREVA NP Inc.
Docket No. 52-020
SRP Section: 03.08.01 - Concrete Containment
SRP Section: 03.08.03 - Concrete and Steel Internal Structures of Steel or Concrete Containments
SRP Section: 03.08.05 - Foundations
Application Section: 3.8

QUESTIONS for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

03.08.01-47

Follow-up to RAI 155, Question 3.8.1-7

The response to this RAI provided information related to load combinations described in FSAR Section 3.8.1.3.2. However, the staff has determined that the applicant's response has not fully addressed the two issues indentified below. To determine if the load combinations utilized for the design of the U.S. EPR meet the acceptance criteria in SRP 3.8.1.II.3, the staff requests that the applicant address the following items:

1. There appears to be an inconsistency in Table 03.08.01-7-1 included in the RAI response. Explain why the load factor for soil/lateral earth pressure (H) in the fifth revised load combination listed in Table 03.08.01-7-1 is 1.0 and not 1.3 to match the load factor used for the live load, as indicated in Item 2 of the RAI response.

2. The RAI response indicated that for live load (L), multiple cases are considered where L is varied between zero and its maximum value. The varying live load, however, was not considered in the static model of the NI but was considered in local analysis and design. Since the static model of the NI is used to obtain member forces for design, explain why full live load and no live load were not considered in the static model.

03.08.01-48

Follow-up to RAI 190, Question 3.8.1-28

The response to this RAI has provided a discussion on the methodology used to determine seismic modification factors, which are then used in the equivalent static seismic analysis of the Nuclear Island (NI) Common Basemat Structure. Also included in the discussion is a limited comparison of results between SSI and equivalent-static analyses. The response to RAI 03.08.01-28 does not provide the requested information in sufficient detail for the staff to conclude that the methodology meets the seismic analysis procedures presented in SRP 3.7.2.II.1 and 3.8.1.II.4. Therefore, the staff requests AREVA to submit the following information:

1. The RAI response indicated that forces and moments were obtained from “application of maximum accelerations (determined from the SSI analysis) on a detailed finite element static model,” presumably as factors multiplying the mass associated with each degree of freedom in the static FE model, and that “accelerations are then adjusted by a modification factor until the SSI-derived results are in effective agreement with the statically calculated results.” The RAI response did not demonstrate that the equivalent static approach using the modification factors is conservative with respect to the results obtained using the SASSI analyses. To demonstrate that the seismic demands in the static FE model are adequately represented, (1) provide the specific steps used in developing the seismic modification factors and (2) justify that the use of the seismic modification factors leads to conservative estimates of the seismic demands for the equivalent static FE model in accordance with the request in Items 2 and 3 below.

2. The results provided in the RAI response, for the comparisons between the SSI analysis and static analysis, make no reference to the direction of the seismic ground motion. Clarify if the “base” forces and moments provided correspond to a specific direction of ground motion or if they already represent the combined results from the three directions of ground motion. The staff notes that a technically acceptable comparison of results should be made using an independent comparison in each of the three orthogonal directions of ground motion.

3. The results provided in the RAI response refer only to “base” forces and moments in the NI Common Basemat Structure, and to a single soil case. To verify the adequacy of the proposed methodology, provide numerical results to confirm that the force resultants (integrated moments and shears) from the equivalent static FE model analyses are equal to or more conservative than the force resultants from the SSI analyses. This should be done for an adequate number of elevations representative of the vertical distribution of the structure, for soil cases that represent the range of soil properties, and for all Seismic Category I structures.

4. Since the SASSI stick model used for SSI analysis is a simple representation of the NI Common Basemat Structure, explain how the proposed equivalent static methodology including the use of the modification factors account for localized flexibilities of structural elements such as floor slabs and walls that may experience accelerations higher than ZPA values, as well as other higher mode effects that cannot be captured by the SASSI stick model.

5. As a result of discussions on this topic held during the meeting on December 14-15, 2009 at AREVA offices, AREVA indicated that they understood the concerns raised by the staff and they may revise their analysis approach to eliminate the use of modification factors. AREVA is requested to explain whether the use of modification factors will be eliminated, in which case Items 1 through 4 above would no longer apply.

03.08.03-21

Follow-up to RAI 155, Question 03.08.03-3

The RAI response to Items 1 through 3 relies upon comparison results which were not provided in the RAI response, but instead are contained in AREVA internal documents. To determine to what extent the use of ACI 349-01 meets the guidance in SRP 3.8.3.II.2 and RG 1.142 (which endorse ACI 349-97 with certain exceptions), and since the comparisons were not provided as part of the response, the staff requests AREVA to provide a summary of these comparisons that explain how: (a) design methodologies in ACI 349-01 and ACI 349-97 remain “substantially

unchanged," with the exception of sway frame design; and (b) the design of sway frames per ACI 349-01 is a "significant improvement" over the methodology in ACI 349-97.

The response to Item 4 of this RAI explains that the strength reduction factor for shear in ACI 349-01 depends on whether the failure mode of a structural element is shear-controlled (φ=0.6) or bending-controlled (φ=0.85). In contrast, the strength reduction factor for shear strength in ACI 349-06 is the same in both cases. It is further indicated that the rationale for the relaxed provision in ACI 349-06 lies in the expected ductility demands in safety-related nuclear structures, which are typically lower than those in conventional building structures. The RAI response ascertains that, since the provision in ACI 349-01 is based on ACI 318 (code requirements for conventional building structures), then this provision is not necessary for safety-related nuclear structures. Finally, it is noted that while the load combinations in ACI 349-01 are different than those in ACI 349-06, Appendix C of the latter code still allows the use of the load combinations in the former code together with a strength reduction factor for shear equal to φ=0.85.

The staff believes that using isolated provisions (i.e., reduced shear strength reduction factor) of one code (ACI 349-06) together with the remaining provisions of another code (ACI 349-01) is not recommended since it may result in undesirable inconsistencies in the design. There may be other provisions in ACI 349-06 which could have an impact on the design but would not be utilized following the approach described by AREVA. Based on the preceding discussion, existing SRP 3.8 and applicable regulatory guides, and past precedent, the staff requests AREVA to identify one version of the ACI 349 code and to follow consistently the provisions of that code without making selected exceptions.

In addition, the RAI response indicates that the rationale for the elimination of the Section 9.3.4 (in ACI 349-01) requirement from ACI 349-06 is that ductility demands in safety-related nuclear structures are typically lower than those in conventional building structures. The staff emphasizes that this relaxed code provision is a significant departure from the ductile design philosophy utilized in the past, and cannot be accepted solely on the basis of a qualitative statement such as the one provided in the RAI response, especially considering that FSAR Section 3.8.3.4.4 (paragraph 5) indicates that "Seismic Category I concrete structural elements and their connections are detailed for ductility in accordance with ACI 349-2001, Chapter 21."

Therefore, AREVA is requested to identify which version of the ACI Code will be followed without reference to another code for exceptions. If ACI 349-06 is utilized for design and construction of reinforced concrete structures inside and outside the containment, provide a technical justification for any relaxation in the provisions of this code from those in ACI 349-97, which is endorsed by RG 1.142. This justification should provide the basis along with numerical data or test results where appropriate.

03.08.03-22

Follow-up to RAI 155, Question 03.08.03-5

The response to item 3 of this RAI confirms that all footnotes of Table Q1.5.7.1 in ANSI/AISC N690 are included in the design of the U.S. EPR. Furthermore, the design of all steel components, members, and connections follow the provisions of ANSI/AISC N690 (1994), including supplement 2 (2004), as referenced in FSAR Sections 3.8.3.2.1, 3.8.3.2.3, 3.8.3.3.2, 3.8.3.4, and Section 3.8.3.4.1. Finally, it is stated that footnotes in FSAR Sections 3.8.3.3.2 and

3.8.4.3.2 will be updated by deleting the text “The stress limit coefficient in shear must not exceed 1.4 in members and bolts” to avoid confusion. However, the staff notes that the markups included with the RAI response have been modified by adding an additional footnote 2 without deleting any text in footnote 1. If all the provisions of ANSI/AISC N690 (1994), including supplement 2 (2004), are being followed without exceptions then this should be clearly reflected in the FSAR. In particular, explain why certain footnotes are being selectively included in the FSAR and not others.

The response to item 4 of this RAI indicates that, for the load combinations described in FSAR Sections 3.8.2.3, 3.8.3.3, 3.8.4.3, and 3.8.5.3, the variation in the live loads (L) from zero to full value was not considered in the static model of the NI but was considered in local analysis and design. To determine if the load combinations utilized for the design of the U.S. EPR meet the acceptance criteria in SRP 3.8.2.II.3, 3.8.3.II.3, 3.8.4.II.3, and 3.8.5.II.3, and since the static model is used to obtain member forces for design, explain why full live load and no live load were not considered in the static model. Also, explain why a load factor of zero is not considered for loads where the use of the full value may reduce the effects of the other loads, as required in ACI 349. Information regarding this issue may be provided in conjunction with the response to the follow-up to RAI 3.8.1-7 Item 2.

Finally, the RAI response indicates that FSAR Sections 3.8.2.3.1, 3.8.3.3.1, and 3.8.4.3.1 will be updated to reflect the information presented in the RAI response. However, the staff notes that the markups included with the RAI response do not show these updates. Explain this inconsistency.

03.08.03-23

Follow-up to RAI 155, Question 03.08.03-6

The approach described in the RAI response, to address the impactive and impulsive loads for concrete and steel structures, is acceptable. However, the proposed change to the FSAR does not incorporate the description of this approach. Therefore, the applicant is requested to include in the proposed revision to the FSAR a description of the analytical approach and acceptance criteria which is stated to be in accordance with ACI 349 and AISC/N690, supplemented by the criteria in FSAR Section 3.5 for acceptable ductility limits.

03.08.03-24

Follow-up to RAI 155, Question 03.08.03-10

The response to this RAI indicates that the 100-40-40 method described in ASCE 4-98 is mathematically equivalent to the 100-40-40 method described in RG 1.92, Rev. 2, and that FSAR Sections 3.8.3.4.4 and 3.8.4.4.1 will be revised to clarify any ambiguity regarding the 100-40-40 method.

However, it is not clear to the staff that the 100-40-40 rule is being correctly implemented in the design of the U.S. EPR. For example, the response to RAI No. 248 Question 3.7.2-26 and FSAR Tables 3E.2-1 through 3E.2-5 appears to indicate that each 100-40-40 rule permutation is being considered as a different load combination in the design. This is not technically acceptable because it could lead to an unconservative structural design. There appears to be

some confusion between the combination of responses due to multiple ground motion directions and the combination of multiple interacting force/moment resultants. AREVA should compare their implementation of the 100-40-40 rule with the interpretation of this rule illustrated by the example given below, and any discrepancies should be addressed. AREVA should also confirm that the 100-40-40 rule is only being used in the context of linear analysis since the principle of superposition is no longer valid when nonlinear behavior is assumed.

Clarification of the 100-40-40 Rule for Use in Structural Design

To clarify the correct implementation of the 100-40-40 rule, consider the case of a concrete shell element as an illustrative example. The multiple interacting force/moment resultants are then Tx, Ty, Txy, Nx, Ny, Mx, My, and Mxy.

Assume that the seismic analysis yields the following results due to ground motions in directions 1, 2, and 3. The numerical values represent the maxima of each force/moment resultant (in absolute values) caused by each of the three ground motions calculated separately. Note that these maxima do not typically occur at the same time instant.

Direction	Tx	Ty	Txy	...	Mxy
E1	20	10	2	...	200
E2	37	6	5	...	300
E3	59	2	7	...	250

There are 24 different permutations of the 100-40-40 rule, of which only the first three and the last are listed below.

Permutation	Tx	Ty	Txy	...	Mxy
+1.0*E1+0.4*E2+0.4*E3	58.4	13.2	6.8	...	420
-1.0*E1+0.4*E2+0.4*E3	18.4	-6.8	2.8	...	20
+1.0*E1-0.4*E2+0.4*E3	28.8	8.4	2.8	...	180
....	...	...	...	...	...
-0.4*E1-0.4*E2-1.0*E3	-81.8	-8.4	-9.8	...	-450
Maximum	81.8	13.2	9.8	...	480

The technically correct interpretation of the 100-40-40 rule considers each force/moment resultant to be the maximum of the 24 permutations. Therefore, Txmax=81.8, Tymax=13.2, Txymax=9.8, ..., Mxymax=480. (It is interesting to note that, as expected, the SRSS rule results in lower values, Txmax=72.5, Tymax=11.8, Txymax=8.8, ..., Mxymax=439.)

For structural design, simultaneous interaction of the force/moment resultants is necessary (e.g. for use with interaction diagrams). However, as previously noted, the 100-40-40 rule (or SRSS) yields only maxima that do not typically occur at the same time instant. An acceptable conservative approach for structural design, also endorsed by ASCE 4-98, is to use permutations of the values Tx=±81.8, Ty=±13.2, Txy=±9.8, ..., Mxy=±480 (a total of 2^8=256 permutations). For example, if only Tx and Ty are assumed to interact, then only the pairs (+81.8, +13.2), (-81.8, +13.2), (+81.8, -13.2), and (-81.8, -13.2) need to be used.

The approach described above is consistent with ASCE 4-98. If this approach or RG 1.92 Rev. 2 is not utilized, then AREVA should provide the technical basis for any proposed alternative.

03.08.05-24

Follow-up to RAI 155, Question 03.08.05-4

The response to this RAI provided additional information about general procedures applicable to Seismic Category I (SC I) foundations. However, the staff finds that portions of this RAI response may be superseded in light of the response to RAI 3.8.5-8, which states that: (a) a new SSI analysis of the NI Common Basemat Structure has been performed using fully embedded conditions to address sliding and overturning issues; and (b) the new analysis models the tendon gallery as a shear key. Consequently, revise the RAI response and the relevant sections of the FSAR so that they are compatible with the aforementioned new analysis assumptions. The revised response should address the questions raised in the original RAI for all SC I structures, not just the NI Common Basemat Structure. In addition, provide further clarification on several issues as discussed below.

1. The response to Item 1 of this RAI provides the formulas utilized in the calculation of factors of safety for sliding, overturning, and flotation. These formulas are based on standard free body formulations of moments and forces. However, the staff notes that vertical wind forces ("Fz_wind") are added to the resisting forces/moments in the sliding and overturning calculations; in addition, forces due to soil pressure (e.g., Fy, Fz, and Mx) are also utilized. In order to determine whether the stability evaluations are performed in accordance with the criteria in SRP 3.8.5.II, provide a description along with a diagram that explains the use of these forces in the methodology to calculate the minimum factors of safety.

2. The response to Item 2 of this RAI indicates that minimum factors of safety for overturning and sliding for the load combination including E' will be provided in response to RAI 3.8.5-8. However, the response to RAI 3.8.5-8 only provides the minimum factors of safety for the NI Common Basemat Structure. The response to this Item should be revised to address all SC I structures.

3. Revise the response to Item 3 of this RAI in light of the response to RAI 3.8.5-8. In addition, provide the technical basis for assuming that the value $\mu = 0.7$ is applicable as a dynamic coefficient of friction as well as the minimum static coefficient of friction, for all soil types considered. The staff notes that dynamic coefficients of friction have values that are typically lower than static coefficients of friction. If these values are overestimated, then the corresponding factors of safety against sliding could also be overestimated and it would not be possible to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

4. Revise the response to Item 4 of this RAI in light of the response to RAI 3.8.5-8. In addition, since the current response to Item 4 states that the submerged unit weight of the soil was used for all structural analyses, provide the technical bases for assuming that this is always the worst condition or describe what other conditions were considered. This information is needed to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

5. The response to Item 5 of this RAI is not complete. As requested in the original RAI, compare the seismic soil pressure used in the design of foundation walls to the maximum calculated (lateral) soil pressure load distribution from the sliding and overturning seismic analysis. This information is needed because the assessment of lateral soil pressures is explicitly identified in SRP 3.8.5.II.4 as an area of staff review.

6. The response to Item 7 of this RAI indicates that negligible impact was noted when the NI Common Basemat was subjected to a differential displacement of 1 inch in 50 feet. However, as stated in the staff's evaluation of RAI 3.8.5-11, it appears that a rigid body-type evaluation was performed to assess differential settlement effects. If this approach was used, the staff finds this unacceptable. For a typical shear wall structure, a differential settlement of 1 inch in 50 feet can result in significant demands on the structure when the differential settlement is defined as an imposed vertical shear deformation and not as a rigid body rotation. The demands on the structure will be a function of the stiffness of the foundation soils supporting the basemat. If such demands are ignored in the design of the basemat, then overstressed conditions and cracking of the basemat could result. Therefore, clarify the approach used to determine differential settlement effects. If differential settlement effects are not negligible, then address the issues raised by Item 7 of the original RAI in regard to load cases and load combinations. In addition, the RAI response indicates that differential settlement effects were evaluated for the softest soil type (soil case 1u). However, the staff notes that the use of the softest soil profile does not necessarily lead to the largest demands on the structure at all locations. Therefore, provide the results of analyses performed to show that bounding estimates were determined, and that induced demands are small for any acceptable soil type. Finally, no mention is made of how differential settlement effects were evaluated for SC I structures outside of the NI Common Basemat. Confirm that differential settlement effects were analyzed for these other SC I structures and provide a discussion of the analysis findings.

03.08.05-25

Follow-up to RAI 155, Question 03.08.05-5

The response to this RAI provided additional information on the static FE model used to analyze and design the NI basemat foundation, and to determine the static bearing pressure on the supporting soils. The staff finds that further clarification is necessary on several issues, as discussed below. This clarification is necessary to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

1. The response to Item 1 of this RAI states that the Gazetas equation was developed for dynamic, not static, conditions. Provide additional technical justification on why the Gazetas equation is appropriate for use in the equivalent-static seismic analysis of the NI structures, for the design of the basemat foundation, and for the evaluation of soil bearing pressures. This technical justification should include a comparison with results obtained from the SSI analysis, for all soil types considered appropriate for foundation support.

2. The response to Item 2 of this RAI states that the simplified tri-linear soil spring stiffnesses are determined from the dynamic soil shear modulus. As indicated in the

staff's evaluation of RAI 3.8.5-7, further clarification is required regarding the development and use of tri-linear springs in the analysis of the NI foundation basemat. The issues raised by Item 2 of this RAI response are evaluated under RAI 3.8.5-7.

3. The response to Item 3 of this RAI does not address the intent of the original RAI, which requests AREVA to discuss the issue of variability of soil conditions (i.e., stiff and soft spots in the foundation soil), and their effect in the design of the NI foundation basemat. The staff notes that FSAR Section 2.5.4.10.3 states that, “The design of the U.S. EPR is based on analyses that assume the underlying layers of soil and rock are horizontal with uniform properties. Furthermore, the U.S. EPR is designed for application at a site where the foundation conditions do not have extreme variation within the foundation footprints. However, the design does have margin that allows for adaptation to many sites that might be classified as non-uniform or having highly variable properties.” From this statement, it follows that allowance for horizontal variability of soil conditions should be an important design consideration. The RAI response indicates that the softest soil case 1u bounds the design NI foundation basemat. It is not clear, however, if the design of the basemat has considered the potential effects of horizontal variability of soil conditions. Therefore, as requested in the original RAI, describe what studies were performed to evaluate the effects of different soil stiffnesses across the foundation footprint (e.g., higher soil stiffness in the central portion with lower soil stiffness beyond the center, and vice versa), or provide the technical basis for not doing such a study.

In addition, since the RAI response appears to indicate that the softest soil case 1u bounds the design NI foundation basemat, provide information on how bending and shear demands in the basemat will be modified for the case of stiffer foundation soils, and to confirm if the use of the soft soil cases bound the expected demands on the basemat.

03.08.05-26

Follow-up to RAI 155, Question 03.08.05-6

The response to Item 2 of this RAI indicates that: “The Gazetas equation was used because it is more suitable for the assumed inhomogeneous subsurface conditions, and because it yields similar results as the Wong-Luco methodology.”

The staff finds that the RAI response does not provide sufficient technical justification for the use of the Gazetas equation. As noted in the staff's evaluation of RAI 3.8.5-5 Item 1, the Gazetas equation was developed for dynamic, not static, conditions. To complete the response to this RAI, provide additional technical justification on why the Gazetas equation is appropriate for use in the equivalent-static seismic analysis of the NI structures, for the design of the basemat foundation, and for the evaluation of soil bearing pressures. This technical justification should include a comparison with results obtained from the SSI analysis, for all soil types considered appropriate for foundation support, and an explanation of why the Gazetas equation is “more suitable for the assumed inhomogeneous subsurface conditions.” This information may be included in the supplemental response to RAI 3.8.5-5 Item 1.

03.08.05-27

Follow-up to RAI 155, Question 03.08.05-7

The response to this RAI provided additional information regarding the development and use of tri-linear springs in the analysis of the NI basemat foundation. The staff finds that further clarification is necessary on several issues, as discussed below. This clarification is necessary to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

1. The response to Item 1 of this RAI requires further clarification on the following issues: (a) provide information to demonstrate that the behavior of soils beneath and near the edge of a foundation can be represented by modulus reduction curves, presumably associated with simple one-dimensional site response analyses; (b) if the soil to the side of an embedded foundation is confined, provide information on why the tri-linear model with strain softening is appropriate for such an application, and that strain-stiffening does not occur; and (c) provide a basis for judging when a particular calculation yields "unrealistic" results. With regard to (c), explain why Figure 03.08.05-7.1-1 is compared to Figure 03.08.05-7.1-2 when, in both cases, the maximum values shown are relatively close. The staff notes that, unless these issues are clarified, the use of tri-linear springs with strain softening does not appear to be technically appropriate, and may underestimate soil bearing pressures or foundation stresses.

2. The response to Item 2 of this RAI indicates that non-linear (strain-softening) springs are specifically used for examination of maximum bearing pressures due to seismic overturning moments, and the sliding factor of safety. This appears to contradict FSAR Section 3.8.5.4.2, which states: “The NI Common Basemat Structure foundation basemat is analyzed and designed using the ANSYS V10.0 SP1 finite element overall computer model (a static model) for NI Common Basemat Structure Seismic Category I structures. (...) This model is also used to determine the static pressure on the supporting soils. (...) Springs are used to represent soil that provides support for the concrete foundation basemat in the ANSYS model. (...) Tri-linear soil springs are developed for soil cases 4u and 2sn4u. (...) A second model was developed to evaluate the soil bearing pressures, sliding and overturning due to seismic events. This (second) model explicitly represents the nonlinearities of sliding and uplift, the transient nature of the seismic loadings, the properties of the soils, and the dynamic characteristics of the structure. (...) (In the second model) Soil is modeled with one layer of solid elements beneath the shells, representing the basemat.” Explain which FE model was used to assess maximum bearing pressures due to seismic overturning moments, and sliding factor of safety. The staff notes that the “second model” is being evaluated under RAI 3.8.5-8.

In addition, the RAI response requires further clarification on the following issues: (a) provide the technical basis for stating that soil cases 1u, 4u and 2sn4u, previously defined as soft to medium soil sites with shear wave velocity properties selected for SSI analyses, are in fact classified as clay soils having hysteretic damping and static stress-strain properties appropriate for soils with a plasticity index (PI) greater than 50, as indicated in FSAR Section 3.8.5.4.2 and in the response to Items 3 and 4 of this RAI; (b) explain if sites with granular soils and shear wave velocity properties matching those of soil cases 1u, 4u or 2sn4u are excluded from the U.S. EPR design; (c) provide the technical basis for stating that tri-linear static and dynamic spring stiffness for soil cases

1u, 4u and 2sn4u represent bounding cases for clay soils or, more generally, for assuming that these non-linear analyses yield bounding seismic responses for all soil types considered appropriate for foundation support. As indicated in Item 1 above, unless these issues are clarified, the use of tri-linear springs with strain softening does not appear to be technically appropriate, and may underestimate soil bearing pressures or foundation stresses.

3. The response to Item 3 of this RAI states that tri-linear springs are based on shear strain curves used in calculating dynamic spring constants for soil cases 4u and 2sn4u. It adds that these soils are assumed to be comprised of stiff clays having a PI of 60. However, the staff notes that soils having a PI of 50 or greater are typically labeled as fat clays, which may not be appropriate for foundation support of a NPP unless they are found in a highly pre-consolidated state. In that case, the clays would be expected to be stiff rather than soft. Provide additional information to justify why the characteristics of such soils can be considered appropriate for NPP applications.

4. The response to Item 4 of this RAI requires further information to explain why the tri-linear properties selected for these high PI soils are not so soft as to be unacceptable for use in a foundation of a NPP. Additional information in this regard is requested under Item 2 above.

5. The response to Item 5 of this RAI requires additional information to explain how the vertical displacement scale of Figure 03.08.05-7.5-1 was developed, as well as how the properties shown in the Figure relate to the S-wave and P-wave properties used to represent soil cases 4u or 2sn4u in SSI analyses. As indicated in Item 1 above, unless this issue is clarified, the use of tri-linear springs with strain softening does not appear to be technically appropriate, and may underestimate soil bearing pressures or foundation stresses.

03.08.05-28

Follow-up to RAI 155, Question 03.08.05-8

The response to this RAI indicates that a new FEM SSI analysis of the NI Common Basemat Structure has been performed using fully embedded conditions to address sliding and overturning issues as well as to determine dynamic soil bearing pressures. This new analysis models the tendon gallery as a shear key. In light of this new information, several items in the original RAI (related to the “second model” as described in FSAR Section 3.8.5.4 Revision 0) are no longer applicable. Nevertheless, the staff finds that portions of the original RAI are still relevant and require further clarification as discussed below. This clarification is necessary to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

1. The responses to Items 1 through 3 of this RAI indicate that the “second model” in question has been superseded by a new analysis methodology and the second model is no longer applicable. To resolve Items 1 through 3 of this RAI, provide a detailed description of the new FEM SSI analysis methodology, including a figure showing the relevant details of the computer model, and a description of the computer code used to perform the analysis, and all relevant analysis assumptions. This information should be included in FSAR Section 3.8.5.

2. The response to Item 4 of the RAI indicates that “the static coefficient of friction at the soil-concrete interface is based upon the angle of internal friction and on a roughened contact surface.” It adds that a coefficient of friction of 0.5 (representing saturated conditions) and 0.7 (representing dry conditions) are used in the new analysis methodology. To resolve Item 4 of this RAI, the staff requests the following information:

(a) The issue of potential sliding at the various interfaces (i.e., basemat-upper mudmat, upper mudmat-membrane, membrane-lower mudmat, lower mudmat-soil) is explicitly identified in SRP 3.8.5.II.4 as an area of staff review; however, this issue has not been addressed. Explain how it is assured (e.g., COL action items) that the coefficients of friction at these various interfaces are higher than those listed in the RAI response. This information should be included in FSAR Section 3.8.5 and other relevant sections of the FSAR.

(b) The coefficient of friction of 0.5 assumed in the new analysis for saturated conditions is lower than indicated in FSAR Sections 2.5.4 and 3.8.5 and in other RAI responses (e.g., RAI 3.8.5-4), and is therefore more conservative. This new information should be incorporated to the relevant sections of the FSAR.

(c) The RAI response explicitly mentions that static coefficients of friction are used in the new analysis. To justify this assumption, confirm that the new analysis findings demonstrate that no sliding of the structure occurs for any soil cases considered in the design certification. Otherwise, as mentioned in the original RAI, dynamic coefficients of friction need to be used (typically having lower values). Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.5-4 Item 3. The staff notes that if the coefficients of friction are overestimated then the corresponding factors of safety against sliding could also be overestimated, and it would not be possible to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

3. The response to Item 5 of the RAI indicates that the “second model” in question has been superseded by a new analysis methodology and is no longer applicable. The new analysis uses an embedded FEM SSI model with the tendon gallery as a shear key. To resolve Item 5 of this RAI, and since the new analysis uses an embedded FEM SSI model, explain the methodology used to determine the dynamic lateral pressure loads at the interface between the soil and the exterior foundation walls, tendon gallery walls, and vertical edges of the NI Common Basemat Structure. This explanation should include a comparison to the lateral pressure loads used in the design of these vertical elements as well as a comparison to the full passive pressures that can be developed in the soil, for all soil cases referenced in the design certification. This information is needed to determine if the foundation design related to soil pressures is in accordance with the criteria in SRP 3.8.5.II.4. Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.5-4 Item 5.

4. The response to Item 7 of the RAI indicates that the “second model” in question has been superseded by a new analysis methodology and is no longer applicable. The markup to FSAR Section 3.8.5.4.2 (paragraph 11) included with the RAI response further indicates that the new analysis considers excitation by the three EUR seismic transients (CSDRS) for soil cases 2sn4u, 4u, and 5a corresponding to soft, medium and hard soils. To resolve Item 7 of this RAI, provide the technical justification for not

considering in the new analysis the other soil cases referenced in the design certification.

5. The response to Item 8 of the RAI indicates that the “second model” in question has been superseded by a new analysis methodology and is no longer applicable. The new analysis uses an embedded FEM SSI model, with the tendon gallery as a shear key, to address sliding and overturning issues. The response adds that instantaneous demand-to-capacity ratios to determine sliding and overturning factors of safety are generated by time history methods. Finally, the minimum factors of safety against sliding and overturning are listed as 1.16 and 1.78 respectively, which are in conformance with the required factors of safety given in FSAR Table 3.8-11. To resolve Item 8 of this RAI, and to ensure that stability evaluations and calculation of soil pressures are performed in accordance with the criteria in SRP 3.8.5.II, the staff requests the following information:

(a) Provide a detailed description of the methodology used to determine the minimum factors of safety against sliding and overturning “generated by time history methods” from the FEM SSI model. This description should provide numerical results for the three soil cases analyzed and also be included in FSAR Section 3.8.5.

(b) Since the RAI response indicates that the FEM SSI model used in the new analysis assumes linear behavior with a coefficient of friction of 0.5 to 0.7, explain how sliding can be included in the analysis and how is the issue of potential uplift of the NI foundation basemat addressed.

(c) The markup to FSAR Section 3.8.5.4.2 (paragraphs 1 and 9) included with the RAI response indicates that the embedded FEM SSI model is also used to calculate dynamic soil bearing pressures. Confirm that this is the case and provide representative calculation results for all soil cases referenced in the design certification. In addition, there appears to be some inconsistency in the values of the maximum soil bearing pressures listed in the FSAR. The markup to FSAR Section 3.8.5.4.1 (paragraph 8) indicates the maximum static bearing pressure is 34,560 psf, whereas the markup to FSAR Section 3.8.5.5.1 (paragraph 2) lists the maximum static and dynamic bearing pressures as 22,000 psf and 26,000 psf respectively. This inconsistency should be clarified.

(d) The RAI response indicates that the tendon gallery is modeled as a shear key to resist seismic induced shear forces. However, the markup to FSAR Section 3.8.5.1.1 (paragraph 2) included with the RAI response states: “No credit is taken in the design for the tendon gallery transmitting vertical loads into the soil, and the connection of the tendon gallery to the NI Common Basemat Structure foundation basement allows for differential movement between the concrete structures. However, the tendon gallery acts as a shear key and transfers lateral loads into the basemat.” Clarify this statement. In particular, explain how the connection of the tendon gallery to the NI basement allows for differential movement and, simultaneously, transfers seismic induced shear loads. Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.5-2.

(e) The RAI response does not mention if the results of the new analysis are utilized in the concrete design of the NI foundation basemat, for load cases involving seismic loads. If this is not the case and the static FE model of the NI Common Basemat Structure (with soil springs as described in FSAR Section 3.8.5.4.2) is used in the

design, then confirm that comparative studies have been performed to show that the equivalent-static seismic analysis results (e.g. soil bearing pressures and internal forces and moments in the basemat) are in agreement with the results of the new FEM SSI analysis. Information regarding this issue should be provided in conjunction with the response to the follow-up to RAIs 3.8.5-5 Item 1 and 3.8.5-6, for all soil cases referenced in the design certification.

6. Since a new FEM model and analyses are being performed as discussed in this RAI, and other changes in the analysis methods (e.g., a new FEM used in the SSI analyses, revised set of soil cases) are being implemented, as noted during the meeting with AREVA on December 14 and 15, 2009, AREVA is requested to reconcile this and the other applicable RAIs regarding these changes.

03.08.05-29

Follow-up to RAI 155, Question 03.08.05-9

The RAI response states that the use of a uniform subgrade modulus distribution underestimates soil bearing pressures, but that this is acceptable since the bearing pressure requirements are controlled by the larger and more massive NI structure. However, this may not be the case since the EPGB and ESWB structures may have different foundation material with bearing pressure requirements that are less than the NI structure. Therefore, provide further justification for not considering the elliptical subgrade modulus distribution for all soil cases in determining the bearing pressure requirements for the EPGB and ESWB structures. The bearing pressure requirements for these structures should be specified in the FSAR along with a discussion of the technical bases for the requirements. This information is needed to ensure that the soil bearing pressures for all safety-related structures meet the acceptance criteria in SRP 3.8.5.II.

The RAI response also states that Ko for the EPGB soil case 5a is 13,944 kcf. This is considered to be a very high value for typical soils and as compared to all the other reported values. Clarify if this value is correct. If it is, describe how this value was determined.

03.08.05-30

Follow-up to RAI 155, Question 03.08.05-11

The response to this RAI indicates that the criterion for differential settlements of 0.5 inches per 50 feet in any direction across the NI foundation basemat is obtained from a geotechnical report prepared for U.S. EPR Design Certification. In addition, it refers to a study in which structural analyses were performed with and without initial tilt settlements for the softest soil type considered. The results of this study show negligible differences in both soil bearing pressures and stresses in the basemat. The staff finds that the RAI response requires further clarification as discussed below. This clarification is needed to determine if the foundation design related to differential settlement meets the acceptance criteria in SRP 3.8.5.II.

1. The RAI response did not provide a detailed explanation as to how the differential settlement criterion was determined. As requested in the original RAI, provide the technical basis for determining this differential settlement criterion.

2. The RAI response did not provide a detailed explanation on how the effects of the differential settlements are considered in the design of the NI basemat (i.e., details of the FE model used, how the prescribed displacements are imposed on the model, and how these prescribed displacements were considered in the load combinations). Therefore, the staff requests that this information be provided, as requested in the original RAI.

3. Since the RAI response uses the terms “tilt settlement” and “inclination”, it appears that a rigid body-type evaluation was performed. The staff finds this approach unacceptable. For a typical shear wall structure, a differential settlement of 0.5 inches per 50 feet can result in significant demands on the structure when the differential settlement is defined as a vertical shear deformation imposed on the basemat, and not as a rigid body rotation. The demands on the structure will be a function of the stiffness of the foundation soils supporting the basemat. If such demands are ignored in the design of the basemat, then overstressed conditions and cracking of the basemat could result. Therefore, clarify the approach used to determine the effects of differential settlements.

03.08.05-31

Follow-up to RAI 155, Questions 03.08.05-10 and 03.88.05-12

The staff finds that the information provided in the responses to RAIs 3.8.5-10 and 3.8.5-12 requires additional clarification as discussed below. This clarification is needed to determine if the foundation design related to stability evaluations and soil pressures meets the acceptance criteria in SRP 3.8.5.II.

1. Provide a summary of the procedure used to determine the static and dynamic soil bearing pressures, including representative values for all soil cases considered in the design certification, and include this information in the relevant sections of the FSAR. In this regard, the staff notes that the markup to FSAR Section 3.8.5.4.1 (paragraph 1), included with the response to RAI 3.8.5-8, states: “The underlying soil medium is represented by FEM for SSI analysis for the NI and by soil springs for other Category I structures as described in subsequent sections.” This statement appears to indicate that the dynamic soil bearing pressures are determined from an equivalent-static seismic analysis with the soil represented by equivalent springs. If this is the case, then final values of soil bearing pressures will need to be reconfirmed after resolution of RAI 3.8.1-28 (adequacy of modification factors used in equivalent-static seismic analysis) and RAI 3.8.5-9 (adequacy of soil springs utilized in the analysis of the EPGB and ESWB).

2. Provide a summary of the procedure used to calculate minimum factors of safety against sliding and overturning, and include this information in the relevant sections of the FSAR.

3. Confirm whether the coefficients of friction used in the sliding stability analyses are consistent with those given in the response to RAI 3.8.5-8 Item 4; that is, static coefficients of friction of 0.5 representing saturated conditions and 0.7 representing dry

conditions. If these values are used, additional justification should be provided to demonstrate that no sliding of the structure occurs for any soil cases considered in the design certification. Otherwise, as mentioned in the staff's evaluation of RAI 3.8.5-8 Item 4, dynamic coefficients of friction need to be used, typically having lower values. It is important to note that if the coefficients of friction are overestimated then the corresponding factors of safety against sliding could also be overestimated, and it would not be possible to determine if the foundation design meets the acceptance criteria in SRP 3.8.5.II.

4. Explain the procedures used to calculate seismic induced lateral soil pressures and provide the pressure distributions on foundations for the following cases: (a) seismic SSI analyses, (b) sliding and overturning stability analyses, and (b) design of below-grade foundation walls. In addition, the explanation should demonstrate that these pressures are bounded by the full passive pressures that can be developed in the soil, for all soil cases referenced in the design certification, and that the design of the foundation walls is performed for the envelop of cases (a) and (b) identified above. Finally, in the case of stability analyses, the explanation should be consistent with the sliding/non-sliding assumption discussed in Item 3 above (i.e. full passive pressures in the soil cannot be mobilized if no sliding of the structures occurs). Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.5-4 Item 5.