REVISED RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

APR1400 Design Certification

Korea Electric Power Corporation / Korea Hydro & Nuclear Power Co., LTD

Docket No. 52-046

RAI No.:	**255-8285**
SRP Section:	**03.08.05 – Foundations**
Application Section:	**03.08.05**
Date of RAI Issue:	**10/19/2015**

Question No. 03.08.05-16

10 CFR 50.55a and Appendix A to 10 CFR Part 50, General Design Criteria 1, 2, 4, 16 and 50, provide the regulatory requirements for the design of the containment internal structures. Standard Review Plan (SRP) 3.8.5, Section II specifies analysis and design procedures applicable to the foundation of seismic Category I structures.

Technical Report (TR) APR1400-E-S-NR-14006-P, Rev 1, “Stability Check for NI Common Basemat,” Section 2, “Site Profiles for the APR1400 Nuclear Island Common Basemat,” describes the generic site profiles for the APR 1400 NI common basemat. The staff reviewed this section and noted that additional information is needed in order to perform its safety review of the DCD application. Per 10 CFR 50.55a; Appendix A to 10 CFR Part 50, General Design Criteria 1, 2, 4, 16 and 50; and SRP 3.8.5, the applicant is requested to address the following:

a. Section 2.2, “Review of the Elastic Modulus of Generic Sites,” states that “The HFE program is in effect from the start of the design through the completion of the initial plant startup testing program. At startup the HFE program results will be provided to the combined operating license (COL) holder.” The applicant is requested to describe what the HFE program is and how it relates to the design and analysis during the design certification phase and COL phase.

b. Section 2.2.1, “Elastic Modulus of Soil Sites,” describes the approach used to develop the static elastic modulus E_{static} and the dynamic elastic modulus $E_{dynamic}$ used in the finite element models. The following items need to be addressed:

   1. The approach for E_{static} is based on the relationship between E_{static} and the standard penetration test (STP) blow count. For the type of large structures in the APR1400 design, E_{static} is not normally generated using relationships based on STP blow counts. Therefore, the applicant is requested to utilize accepted industry methods for development of E_{static}.

2. The uncertainty in the relationships presented in APR1400-E-S-NR-14006-P, Rev.1, Section 2.2.1 between SPT blow count (N) and shear wave velocity (Vs) is very high. Any COL applicant will have to use site-specific measurements to define velocity profiles, including layer velocities and uncertainties, thicknesses, etc. that will then be used to compare with the range of profiles used in the DCD design. Therefore these SPT relationships are not considered acceptable for use in defining properties utilized in the design within the DCD and the technical report. Similarly, site velocity properties defined for rock layers will have to be generated by measurements and not by the relationships described in APR1400-E-S-NR-14006-P, Rev.1, Section 2.2 and Figure 2-2. The applicant is requested to adequately address the uncertainty between the STB blow count and the shear wave velocity.

3. The approach used for $E_{dynamic}$ is the elastic modulus. From the information provided, it is not clear how this formulation was used to capture the effects of soil confinement when representing the soil by compression only truss elements in the model. The applicant is requested to provide a detail description regarding its dynamic elastic approach.

4. APR1400-E-S-NR-14006-P, Rev.1, Section 2.2.1 indicated that the relationship between Elastic and $E_{dynamic}$ at the soil site is 0.1153. This ratio appears to be extremely low and is probably due to the questions raised in Item (a) and (b) above. The applicant is requested to update the approach to calculating $E_{static}/E_{dynamic}$ and confirm the adequacy of the resulting ratio based on other sources of information and industry practice.

c. In Section 2.3, “Material Properties and subgrade Modulus of Site Profiles for the APR1400,” it is stated, “The subgrade moduli of three site profiles are obtained from an ANSYS analysis.” The description of the development of the moduli should be expanded in order to understand the approach used. The applicant is requested to provide an explanation of the following: (1) whether only a vertical static 1 ksf load was applied to obtain the vertical soil moduli, (2) whether the vertical load was applied only to the basemat foundation region, (3) what is the technical basis for indicating that the horizontal subgrade moduli were determined using two-thirds of the horizontal displacement caused by what appears to be a vertically applied pressure load, and (4) if the LINK180 ANSYS element is only utilized to represent the soil in the settlement analysis and construction sequence, why is the horizontal moduli needed.

Response – (Rev. 1)

a. The description in Section 2.2 of the Technical Report is an editorial error. Therefore, the description will be revised, as shown in Attachment 1 to this response.

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2. Based on the uncertainty of the relationship between shear wave velocity and STP blow counts, the COL applicant should perform a site-specific evaluation if the site is found to have a shear wave velocity profile (i.e., throughout the depth) that is less than the shear wave velocity profile used in the various basemat evaluation. A site-specific evaluation (maximum bearing pressure, maximum tilting settlement, maximum differential settlement between structures, angular distortion, soil bearing pressure - static and dynamic loading cases), and sliding evaluation) and 3D FEM global analysis for basemat design of seismic category I structures using the site-specific parameters (measured E_{static}, $E_{dynamic}$ consistent with soil strain assumed in SSI analysis), methodology, as described in DCD Tier 2, Subsection 3.8.5 and Technical Report APR1400-E-S-NR-14006-P/NP will be performed. DCD Tier 2, Section 3.8.6 will be revised to include a COL item, COL 3.8(20), as shown in Attachment 1 to this response.

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4. Although the ratio of E_{static}/ $E_{dynamic}$ is extremely low, E_{static} is used for the basemat analysis in order to generate large settlement, which is a conservative approach. This ratio is applied to the soil foundation of site profile1 where the shear wave velocity is less than 1800 ft/sec.

c. 1. A vertical static load of 1ksf was applied to obtain the vertical subgrade modulus. In order to consider the boussinesq effect in soil vertical springs across the basemat, the subgrade modulus of the vertical soil springs was calculated based on the vertical displacement of each basemat node.

2. The vertical load is only applied to the basemat foundation region.

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4. The LINK 180 ANSYS element is utilized to represent soil in the structural and settlement and construction sequence analyses. The horizontal modulus is used in soil springs is modeled as a boundary condition for analyses of the NI common basemat. For a detailed construction sequence, refer to the RAI 255-8285 3.8.5 Question 7.

Supplemental Questions and Responses

Questions 1

The second sentence in the response which is quoted above indicates a site-specific evaluation will be performed for "(differential settlement, soil bearing pressure and sliding evaluation [if needed]) and 3D FEM global analysis for basemat design." The term differential settlement should be revised to encompass the several settlement evaluations (not just differential settlement) as described in the recent NRC feedback to RAI Question 3.8.5-7. Also explain what is meant by the phrase "[if needed]," explain to which loading in the listed loads it applies, describe how that would be determined, and explain why it wouldn't apply to all of the loading evaluations. Note that if the shear wave profile at the site is below the values used in the static and dynamic analyses, then sliding (which is related to the dynamic load case seismic) would be expected to be reevaluated too, unless otherwise justified. The resolution of the above items also affects COL 3.8(13) provided in the markups.

RESPONSE

The COL 3.8(13) was changed to COL 3.8(20).

The term of differential settlements are specified to incorporate into supplemental question as shown in COL 3.8(20) on page 2 of attachment 1.

Regarding the soil bearing pressure of COL 3.8 (20), it is revised to include "... soil bearing pressure (static and dynamic loading cases)..." as shown on page 2 of attachment 1.

For stability check, the detailed explanation for loads and loading determination is summarized as below,

	Loading	Method
Various Settlements	Static loading (D+L)	The various settlements will be checked in accordance with latest feedback of RAI 255-8285 Question 03.08.05- 7.
Soil Bearing Pressure	Static loading (D+L)	Spring force from basemat analysis divided by Tributary Area
	Dynamic loading (Abnormal/Extreme load combination)	Contact pressure contour
Uplift	8 combination from D+L+Es	For detailed explanation for uplift check, refer to the response of RAI 183-8197 Question 03.07.02-4, Rev.2.
Overturning Check	DCD Table 3.8-10	Calculate based on DCD Section 3.8.5.5.1
Sliding Evaluation		Calculate based on DCD Section 3.8.5.5.2

Flotation		Calculate based on DCD Section 3.8.5.5.3

Questions 2

1. The KHNP Input did not address the NRC Feedback transmitted previously which requested: "Please identify where in DCD Tier 2, Section 3.8.5 or 3.8A, the soil bearing pressure evaluation is discussed. There should be at least a summary of the soil bearing DCD evaluation (*and results*) which can then reference the technical report for more details. The current paragraph in DCD Section 3.8.5.4.3 doesn't address this."

2. The KHNP Input states: "Regarding bearing pressure in DCD Tier 2, Table 2.0-1 and DCD Tier 1, Table 2.1-1, the values indicate that the static loading case considers D+L and dynamic loading case considers abnormal/extreme load combination, not D+L+Es." Currently the DCD does not present this information in these sections and the staff cannot locate the markups with this information.

RESPONSE

1. The overall description of soil bearing pressure evaluation and maximum bearing pressure are added in subsection 3.8A.1.4.2.3.5 and 3.8A.3.4.1 as shown in attachment 6.

2. The description, " Regarding bearing pressure in DCD Tier 2, Table 2.0-1 and DCD Tier 1, Table 2.1-1, the values indicate that the static loading case considers D+L and dynamic loading case considers abnormal/extreme load combination, not D+L+Es" is provided by attachment 7.

Questions 3

a. Why is uplift included in the list of items evaluated in Appendix 3.8A? Based on prior discussion with KHNP, the evaluation for uplift is associated with the criterion in SRP 3.7.2 regarding the 80% contact ratio, and this was addressed separately by KHNP as part of Section 3.7. Therefore, the uplift evaluation in the technical report on stability and in DCD Sections 3.8.5 and 3.8A (including Section 3.8A.1.4.2.3.3 and Table 3.8A-16) is not applicable and should be removed. It should be confirmed by KHNP that such discussion to meet the 80% criterion is provided in the appropriate location in DCD Section 3.7.

RESPONSE

All issues related to seismic uplift analysis are dealt with the RAI 183-819, Question 03.07.02-4. DCD section 3.8A.1.4.2.3.3 is changed to make consistency with RAI 183-8197 Question 03.07.02-4 as shown in attachment 9.

Questions 4

b. The markups for a new DCD Section 3.8A.1.4.2.3.5 (page (3/3) in Attachment #6) for bearing pressure evaluation refer to DCD Section 3.8A.1.4.2.3 for the model of the superstructure, NI common basemat, and soil. However, the staff notes that DCD Section 3.8A.1.4.2.3 has not been updated to reflect the new NI basemat analysis (e.g., use of equivalent static analysis for all superstructures, use of soil media model for seismic, phasing of results from superstructure, combination of loads from superstructure, enveloping of results for linear and nonlinear, and other items). All applicable parts of DCD Sections 3.8.5 and 3.8A, and the technical report on stability, for the NI common basemat, should be updated to reflect the current models, analysis approach, and results.

RESPONSE

The explanation for analysis of NI common basemat (e.g., use of equivalent static analysis for all superstructures, phasing of results from superstructure, combination of loads from superstructure, enveloping of results for linear and nonlinear, and other items) is revised as shown in attachment 8.

Questions 5

c. The markups for a new DCD Section 3.8A.1.4.2.3.5 (page (3/3) in Attachment #6) for bearing pressure evaluation provide a description of the analysis for the NI common basemat model. A similar description for the EDGB and DFOT structure has not been provided in the appropriate parts of DCD Sections 3.8.5 and 3.8.A. A description for the EDGB and DFOT should also be provided. Regarding the soil pressures tabulated in the current technical report on stability, the staff notes that in Table A-7 of the technical report, the maximum dynamic (SSE + gravity) soil bearing pressures for the DFOT (for all three soil cases) are less than the static soil bearing pressures. Explain why this occurs.

RESPONSE

The description of bearing pressure in EDGB and DFOT structures is added in DCD section 3.8A.3.4.1 as shown on page 4 of attachment 6.

Regarding the bearing pressure of the DFOT (as shown in the table below based on reanalysis results), maximum dynamic bearing pressure corresponding to the design load combination, including the SSE load, is greater than static bearing pressure. Therefore, the bearing pressure for the EDGB and DFOT is revised as shown on pages 5 and 6 of attachment 6.

	Static load (D+L)	Design load combination including SSE load with buoyance	Design load combination including SSE load without buoyance
Maximum static bearing pressure (Unit: ksf)	7.37		
Maximum dynamic bearing pressure (Unit: ksf)		11.26	18.10

And, regarding to allowable soil bearing pressure capacities of 20 ksf for static and 60 ksf for dynamic, these soil bearing capacities are applied to all seismic category I structures (NI and EDGB and DFOT).

Questions 6

To avoid potential confusion based on Item 4 above (and as discussed in the next NRC feedback 7/25/17 below), DCD Tier 1 Table 2.1-1 and DCD Tier 2 Table 2.0-1, as well as any other related location in the DCD, should clearly identify the meaning of "dynamic soil bearing pressure or dynamic soil bearing capacity." Based on the latest markups in Attachment #10 of the previous KHNP Input submittal, these tables only identify the following:

"Allowable Static Bearing Capacity"

"Allowable Dynamic Bearing Capacity"

The term "Allowable Dynamic Bearing Capacity" should be clarified everywhere to identify what loading condition it corresponds to (e.g., static plus SSE loading condition).

RESPONSE

To avoid potential confusion of bearing capacity, DCD Tier 1 Table 2.1-1 and DCD Tier 2 Table 2.0-1 are revised as show in attachment 7.

Questions 7

(2) Explain why flotation is included in the list of site-specific evaluations when (as stated in COL(13)) the shear wave velocity is less than the shear wave velocity profile used in various basemat evaluations. According to SRP 3.8.5 and DCD Section 3.8.5.5.3, the flotation evaluation is only a function of the dead load of the structure and the buoyant force from the maximum groundwater table. Thus, this evaluation is independent of what the shear wave velocity is at the site. Also, the stability evaluation technical report (Section 4.2.3), referenced by COL(13) below, indicates the following load combination is used for the flotation check:

D+He+Es

“He” is presumably the upward buoyancy force that reduces the dead load D. Es is the seismic load. Therefore, as discussed above, explain why Es is included in the stability evaluation for flotation.

RESPONSE

Regarding to flotation check indicated in subsection 4.2.3 of technical report, APR1400-E-S-NR-14006-P, the description for flotation check is revised as shown in attachment 1.

The COL 3.8(13) was changed to COL 3.8(20).

The description related with shear wave velocity is modified as shown in COL 3.8(20) in page 2 of attachment 1.

Questions 8

Items b.1, b.3, b.4, c.1, c.2, c.3, and c.4:

The key supplemental information in these subsections of the response should be summarized in the technical report on stability in order to provide a complete understanding of the evaluations performed. While some of this information is in the markups provided in the response to RAI Question 3.8.5-8, any missing information should be supplemented. In addition, DCD Section 3.8.5 and/or 3.8A, should also be supplemented with a higher level summary and also reference the technical report on stability for more detailed description of the various analyses performed. This is important because COL 3.8(13) refers to the methodology in the DCD and the technical report.

RESPONSE

The summary regarding to items b.1, b.3, b.4, c.1, c.2, c.3 and c.4 is provided in DCD and Technical Report as shown in attachment 2, 3, 4, and 5.

Impact on DCD

DCD Tier 2 Table 1.8-2 and Subsection 3.8.6 will be revised, as indicated in Attachment 1.

DCD Tier 2 Subsection 3.8.5.4 will be revised, as indicated in Attachment 3.

DCD Tier 2 Subsection 3.8.5.4.3, 3.8A.1.4.2.3.5, 3.8A.3.4.1 will be revised or added, as indicated in Attachment 6.

DCD Tier 1 Table 2.1-1 and DCD Tier 2 Table 2.0-1 will be revised, as indicated in Attachment 7.

DCD Tier 2 Subsection 3.8A.1.4.2.3 will be revised, as indicated in Attachment 8.

DCD Tier 2 Subsection 3.8A.1.4.2.3.3 will be revised, as indicated in Attachment 9.

Impact on PRA

There is no impact on the PRA.

Impact on Technical Specifications

There is no impact on Technical Specifications.

Impact on Technical/Topical/Environmental Reports

Technical report APR1400-E-S-NR-14006-P/NP, Rev.3 Subsection 4.2.3 will be revised, as indicated in Attachment 1.

Technical report APR1400-E-S-NR-14006-P/NP, Rev.3 Section 2.1, 2.2, 2.2.1, and 7 will be revised, as indicated in Attachment 2.

Technical report APR1400-E-S-NR-14006-P/NP, Rev.3 Section 2.3, Table 2-4, and Figure 2-7 will be revised or added, as indicated in Attachment 4.

Technical report APR1400-E-S-NR-14006-P/NP, Rev.3 Subsection 3.2.4 Figure 3-13 will be revised or added, as indicated in Attachment 5.

Technical report APR1400-E-S-NR-14006-P/NP, Rev.3 Table A-7 will be revised, as indicated in Attachment 6.

4.2.3 Flotation Check

Flotation problems may be encountered during construction, operation, or flood condition. The deadweight of the structure is used to resist the hydrostatic uplift. For the flotation check, the hydrostatic force at flooding groundwater level (H_s) is used. Any skin friction between the subgrade exterior walls and backfill is conservatively neglected.

- Resisting force = 1,232,270 kips
- Maximum driving force = 364,029.4 kips
- Factor of safety (FOS) for D+He+Es load combination

  resisting force / maximum driving Force = 3.39 > 1.1

- Resisting force (D)= 1,232,270 kips
  - Weights of the basemat and superstructures are the resisting force.
- Driving force (Hs) = 364,029.4 kips
  - The hydrostatic force at flooding ground water level is the driving force.
- Factor of safety (FOS) for D+Hs)
  - 1,232,270 / 364,029.4 = 3.39 > 1.1

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than the shear wave velocity profile used in the various basemat evaluations for design certificaiton.

parameters (measured E_{static}, $E_{dynamic}$ consistent with soil strain assumed in SSI analysis)

Table 1.8-2 (9 of 38)

Item No.	Description
COL 3.8(20)	The COL applicant shall perform site-specific evaluations if the shear wave velocity is less ~~than 1,000 ft/s.~~ The site-specific evaluations (~~differential settlement, soil bearing pressure, and sliding evaluation [if needed]~~) and 3D FEM global analysis for basemat design of seismic Category I structures shall be performed using the site-specific ~~measured Estatic~~ and the methodology described in Subsection 3.8.5 and Technical report APR1400-E-S-NR-14006-P, Subsection 4.
COL 3.9(1)	The COL applicant is to provide the inspection results for the APR1400 reactor internals classified as non-prototype Category I in accordance with RG 1.20.
COL 3.9(2)	The COL applicant is to identify the site-specific active pumps.
COL 3.9(3)	The COL applicant is to provide a full description of the IST program (including PST and MOV testing) for pumps, valves and dynamic restraints that will be administratively controlled such that the applicable requirements of the ASME OM Code edition and addenda are incorporated in the IST program.
COL 3.9(4)	The COL applicant is to provide an IST program including the type of testing and frequency of site-specific pumps subject to IST in accordance with the ASME OM Code and Table 3.9-13
COL 3.9(5)	The COL applicant is to provide an IST program including the type of testing and frequency of any site-specific valves subject to IST in accordance with the ASME OM Code and Table 3.9-13
COL 3.9(6)	The COL applicant is to provide a table listing all safety-related components that use snubbers in their support systems.
COL 3.10(1)	The COL applicant is to provide documentation that the designs of seismic Category I SSCs are analyzed for OBE, if OBE is higher than 1/3 SSE.
COL 3.10(2)	The COL applicant is to investigate if site-specific spectra generated for the COLA exceed the APR1400 design spectra in the high-frequency range. Accordingly, the COL applicant is to provide reasonable assurance of the functional performance of vibration-sensitive components in the high-frequency range.
COL 3.10(3)	The COL applicant is to develop the equipment seismic qualification files that summarize the component's qualification, including a list of equipment classified as seismic Category I in Table 3.2-1 and seismic qualification summary data sheets (SQSDS) for each piece of seismic Category I equipment.
COL 3.10(4)	The COL applicant is to perform equipment seismic qualification for seismic Category I equipment and provide milestones and completion dates of equipment seismic qualification program.
COL 3.11(1)	The COL applicant is to identify and qualify the site-specific mechanical, electrical, I&C, and accident monitoring equipment specified in RG 1.97.
COL 3.11(2)	The COL applicant is to identify the nonmetallic parts of mechanical equipment in procurement process.
COL 3.11(3)	The COL applicant is to operational address aspects for maintaining the environmental qualification status of components after initial qualification.
COL 3.11(4)	The COL applicant is to provide a full description of the environmental qualification of mechanical and electrical equipment program.

[settlement (maximum vertical displacement, tilt, differential settlement between structures, angular distortion), soil bearing pressure (static and dynamic loading cases), overturning, and sliding] mudmat, and non-uniformity of soil layers, are identified. Then, a site-specific evaluation will be performed.

3) The time (short term vs long term), instantaneous settlement and time-consolidation effect, shall be considered in accordance with surveyed soil profiles. And the differential settlement of the basemat and bearing stress shall be checked to demonstrate acceptability.

4) COL applicant will build the seismic Category I structure according to the construction sequence used in construction sequence analysis.

5) If site-specific evaluation is required, the COL applicant performs construction sequence analysis based on the site-specific parameters. And if the settlement including results of construction sequence analysis exceeds the acceptance criteria in the DCD Table 2.0-1, the construction sequence will be modified to meet the acceptance criteria in the DCD Table 2.0-1 by COL applicant

6) The effect of the design for seismic Category I structures due to construction sequence analysis shall be accounted by COL applicant.

the shear wave velocity profile used in the various basemat evaluations for design certificaiton.

COL 3.8(20) The COL applicant shall perform site-specific evaluations if the shear wave velocity is less than ~~1,000 ft/s.~~ The site-specific evaluations (~~differential settlement, soil bearing pressure, and sliding evaluation [if needed]~~) and 3D FEM global analysis for basemat design of seismic Category I structures shall be performed using the site-specific ~~measured Estatic~~ and the methodology described in DCD Tier 2, Subsection 3.8.5 and Technical report APR1400-E-S-NR-14006-P, Subsection 4.

[settlement (maximum vertical displacement, tilt, differential settlement between structures, angular distortion), soil bearing pressure (static and dynamic loading cases), overturning, and sliding]

parameters (measured E_{static}, $E_{dynamic}$ consistent with soil strain assumed in SSI anlaysis)

3.8.7 <u>References</u>

1. 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," U.S. Nuclear Regulatory Commission.

2. ASME Section III, Subsection NE, "Class MC Components," The American Society of Mechanical Engineers, the 2007 Edition with the 2008 Addenda.

3. ASME Section III, Division 2, "Code for Concrete Containments," Subsection CC, American Society of Mechanical Engineers, 2001 Edition with 2003 Addenda.

2 SITE PROFILES FOR THE APR1400 NUCLEAR ISLAND COMMON BASEMAT

This section describes the low-strain site profiles for the APR1400 NI common basemat.

Replace to pages 2 and 3 of attachment 2

2.1 Shear Wave Velocities of APR1400 Sites

The APR 1400 is designed with a standard design concept to enable construction on various foundation conditions enveloping rock and soil foundations. The low-strain site profiles for the APR1400 include nine site categories (S1 through S9) that represent a variety of characteristics and configurations of rock and soil foundations as well as one fixed case. Figure 2-1 shows the profile of the shear wave velocities of the nine low-strain site profiles categories. As shown in Table 2-1, unit weight and Poisson's Ratio corresponding to shear wave velocity are used to evaluate each site property. Table 2-2 shows the soil and rock definition by shear wave velocity based on the international building code (IBC).

2.2 Review of the Elastic Modulus of Low-strain Site Profiles

2.2.1 Elastic Modulus of Soil Sites

In accordance with IBC, the N value from the standard penetration test (SPT) in the ground with a shear wave velocity V_s = 600~1,200 ft/sec is 15 < N < 50. Therefore, where V_s =1,000 ft/sec, the N value can be interpolated as follows:

$$N = 15 + (1{,}000-600) / (1{,}200-600) \times (50-15) = 38$$

In addition, the relationship between the N value and V_s is described in Zen et al. (1987) as follows:

$$V_s = 89.1 \times (N)^{0.34} \text{ m/sec}$$

Where Vs =1,000 ft/sec (= 304.8 m/sec), the N value can be calculated as N = 37. Based on the results from IBC and Zen et al. (1987), the range of N values at Vs =1,000 ft/sec is between 37 and 38.

The relationship between the static elastic modulus (Estatic) and the N value is provided in Bowles (1982) as follows:

$$E_{static} = 18{,}000 + 750 \times N \text{ (kPa)}$$

$$E_{static} = (15{,}200 \text{ to } 22{,}000) \times \ln N \text{ (kPa)}$$

Where, N = 37 (V_s =1,000 ft/sec, minimum value), the static elastic modulus is obtained as E_{static} = 45,750 kPa, 54,885 kPa, and 79,440 kPa from the relationship between E_{static} and N, respectively. Therefore, the mean static elastic modulus can be determined as E_{static} = 60,025 kPa = 60 MPa = 1,253 ksf.

In addition, the relationship between the maximum dynamic elastic modulus ($E_{dynamic}$) and V_s is as follows:

$$E_{dynamic} = (\gamma / g) \times (V_s)^2 \times [2 \times (1+ \nu)]$$

Where, γ is unit weight, ν is Poisson's ratio, and g is gravity acceleration. Where V_s =1,000 ft/sec, γ = 125 pcf, and ν = 0.4, the dynamic elastic modulus is $E_{dynamic}$ = 10,860 ksf = 520 MPa. The relationship between E_{static} and $E_{dynamic}$ at the soil site is E_{static}/ $E_{dynamic}$ = 0.1153.

The APR 1400 low-strain site profiles are classified as the soil foundation where the shear wave velocity (V_s) is less than 1,800 ft/sec, and the static elastic modulus (E_{static}) is obtained from shear wave velocity (V_s) using the relationships defined in this subsection.

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In the basemat analysis, the static elastic modulus (Es) of soil is normally determined by the results of the site-specific pressure meter test. However, when the site-specific information of soil is not provided, our approach for computing the static elastic modulus is determined as reference value.

Based on the shear wave velocity, the elastic modulus of the soil is generally calculated by the following equation.

$$E = \rho V_S^2 \times [2 \times (1+\mu)]$$

According to subsection 1613.5.2 of IBC (2009) (Ref.1), it defines the relationship between soil shear wave velocity and the standard penetration test (STP) blow count for soils. The use of this relationship shall be limited due to the uncertainty between the STP blow count and the shear wave velocity and utilized to compute the reduction factor. The estimated E and G values are reduced to account for the material's strain softening due to higher strains.

When the shear wave velocity is less than 1800ft/s(sand soil), the range of N values at Vs =1,000 ft/sec is between 37 and 38 based on the results from IBC and Zen et al. (1987) as follows:

$$N = 15 + (1{,}000\text{-}600) / (1{,}200\text{-}600) \times (50\text{-}15) = 38 \text{ (IBC)}$$
$$V_s = 89.1 \times (N)^{0.34} \text{ [m/sec]} \quad \text{(Zen et al.)}$$

The relationship between the static elastic modulus (E_{static}) and the N value is provided in Bowles (1982) as follows:

$$E_{static} = 18{,}000 + 750 \times N \text{ (kPa)}$$
$$E_{static} = (15{,}200 \text{ to } 22{,}000) \times \ln N \text{ (kPa)}$$

Where, N = 37 (Vs =1,000 ft/sec, minimum value), the static elastic modulus is obtained as Estatic = 45,750 kPa, 54,885 kPa, and 79,440 kPa from the relationship between Estatic and N, respectively. Therefore, the mean static elastic modulus can be determined as Estatic = 60,025 kPa = 60 MPa = 1,253 ksf.
Based on the relationship between the elastic modulus and the static elastic modulus, a reduction factor 0.1153, is considered for conservatism based on the STP blow count.

According to in ASCE 4-98 C.3.3.2.2 (Ref.15) and Seed & Idriss (1970) (Ref.21), regarding reduction factor 0.1153, the shear modulus with shear strain level for sand varies as shown in Figure 2-5. Here G is the dynamic shear modulus at very low strain (less than 0.0001%) and $G_d = \rho V_S^2$.
Figure 2-5 demonstrates that the reduction of shear modulus with strain level for sand and the typical variability in the relationship. Generic data from the many field and laboratory test results supported the nonlinear behavior of soil with strain level, as shown in the figure.
Many researchers studied the nonlinear behavior of soil with strain level, that is, the change of elastic modulus of soil with strain level. by Jardine et al. (1986) (Ref.20), Mair(1993) (Ref.19) have shown that the typical static strain levels around geotechnical structures such as retaining walls, foundations, piles, and tunnels fall in the range of 0.01~0.1% (Clayton, 2011) (Ref.17). Burland (1989) (Ref.16) and Finno et al. (2006) (Ref.18) suggested that the working static strain level of soil for the well-designed foundation is on the order of 0.1%.
Considering both the Seed-Idriss curve (Figure 2-5) and the soil static working strain level of 0.1% for foundations, the G/G_d value corresponding to static strain level of 0.1% is in the range of 0.23 ~ 0.37 as shown in Figure 2-6. The lower bound value is 0.23. The relationship between G_s and G_d at a soil site can be considered as $G_s/G_d = 0.23$.

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Considering the relationship between Elastic modulus E and Shear Modulus G, E = G × [2 × (1 + μ)], the relationship between E_s and E_d at a soil site also can be considered as E_s/E_d = 0.23.
The value of E_s/E_d from the Seed-Idriss curve (0.23) is larger than 0.1153 from the SPT blow count related equation. Therefore use of E_s/E_d = 0.1153 from SPT blow count is a conservative approach.

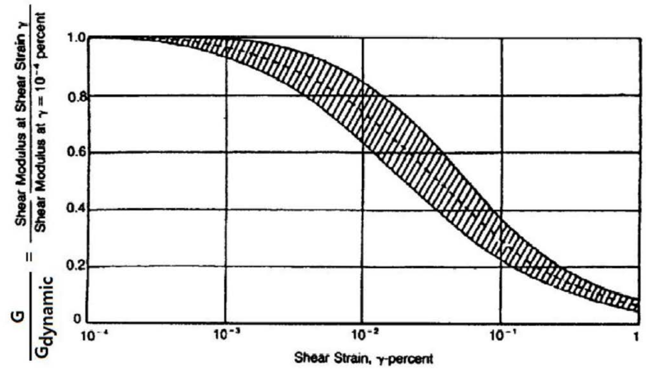

Figure 2-5 Variation of Shear Modulus with Shear Strain for Sands

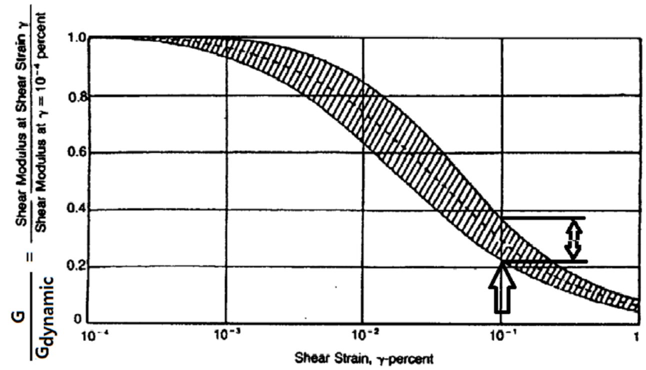

Figure 2-6 $G/G_{dynamic}$ of Soil at strain level 0.1%

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Insert to page 5 of attachment 2

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15. ASCE 4-98 Seismic Analysis of Safety related Nuclear Structures and Commentary

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design code for the foundations below the auxiliary building and EDG building is ACI 349. As the design criteria are different, the applications of loads and load combinations for the foundations are in accordance with each code. While the ACI 349 code concentrates on the requirements as one of the concrete structures in a nuclear power plant, the ASME code describes the requirements with more focus on the functionality of containment. The load factors in Tables 3.8-2 and 3.8-7A are based on these design concepts according to the two codes. For the code application scope and jurisdiction boundary of the NI common basemat, refer to Subsection 3.8.1.1.2 and Figure 3.8-26.

3.8.5.4 Design and Analysis Procedures

The NI common basemat is analyzed using the ANSYS computer program. Stiffening effects of the reactor containment building wall, internal concrete structures, and auxiliary building are included in the model.

The NI common basemat is modeled with eight-node solid element in the ANSYS computer program. In addition, in order to consider the soil effect, the link element in ANSYS is used with the NI common basemat model.

The reinforced concrete basemat of the reactor containment building is designed in accordance with ASME Section III, Division 2, Subsection CC. Other seismic Category I basemats of reinforced concrete are designed in accordance with ACI 349 and the provisions of NRC RG 1.142 where applicable.

As the design criteria for the RCB and AB area of the NI common basemat are different, the application of loads is also divided into two parts, as shown in Figure 3.8-26. The load combinations provided by the ASME and the ACI codes are used in the analysis and design of the RCB and AB foundations, respectively. Regarding the portion beyond the RCB foundation directly beneath the containment shell, the following aspects are additionally considered in the analysis and design of NI common basemat.

At the interface between the two codes, a greater amount of reinforcement required by either code is used, and the reinforcement of the RCB foundation is developed into the AB foundation as shown in Figures 3.8A-16 and 3.8A-17. The provisions of both codes are used to select a conservative development length.

for static loading case. In case of seismic loading, the foundation model is used with the NI common basemat model. Detailed explanation of spring and foundation stiffness is described in Subsection 2 of Technical Report, APR1400-E-S-NP-14006-P (Reference 40)

2.2.2 Elastic Modulus of Rock Site

The dynamic elastic modulus ($E_{dynamic}$) of the rock foundation is obtained from the relationship between $E_{dynamic}$ and Vs in the same way as the soil foundation. In the rock foundation, the static elastic modulus can be calculated as the relationship between static and dynamic elastic moduli of rock.

Figure 2-2 shows the relationship between the static and dynamic elastic moduli of rock ($E_{static}/E_{dynamic}$). V_P and V_L in Figure 2-2 denote the compressional wave velocity in the field and the indoor sound test wave velocity, respectively (Deere, 1966). The reduction factor β is the ratio of static elastic modulus to the dynamic elastic modulus (E_{static} /$E_{dynamic}$). According to Figure 2-2, at soft rock with a low rock quality ($[V_P / V_L]^2$ or rock-quality designation), β is between approximately 0.15 and 0.2. For relatively hard rock with a rock quality greater than 0.6, β is approximately 0.3.

The relationship between the maximum dynamic elastic modulus and shear wave velocity at a rock site is identical to that provided for soil. The static and dynamic elastic moduli of soft rock ($V_s \leq$ 2,500 ft/sec) and relatively hard rock ($V_s >$ 2,500 ft/sec) are 0.15 and 0.3, respectively.

2.3 Material Properties and Subgrade Modulus of Site Profiles for the APR1400

The material properties according to depth from the ground level of the site profiles for the APR1400 are obtained from Sections 2.1 and 2.2. In the tables in Sections 2.1 and 2.2, the material properties are provided according to the depth from the ground level. Among the nine site categories and fixed case, S1, S4, and S8 are considered in the analysis. S1, S4, and S8 denote weak, moderate, and strong site properties, respectively. The analyses used to evaluate the site cases enveloped all of the site categories considered for the APR1400.

The subgrade moduli of three site profiles are obtained from an ANSYS analysis. The site properties used in the ANSYS ground model with 11 layers are based on the basemat analysis calculation and are shown in Table 2-3. ~~For subgrade moduli, a unit pressure of 1 ksf is applied to the ground model and the maximum deformation is calculated. For the horizontal subgrade moduli of the site, two thirds of the maximum horizontal deformation is used.~~ Figure 2-3 shows the deformation contour of ground models. Table 2-4 shows the subgrade moduli of site profiles that are obtained from the ANSYS analysis.

The analysis for computing subgrade modulus of vertical and horizontal was performed separately for horizontal displacement and vertical displacement.
For subgrade modulus to consider soil characteristics, vertical soil pressure 1ksf was applied to surface of basemat foundation region. In order to consider the boussinesq effect in soil vertical spring throughout the basemat, the subgrade modulus of the vertical soil spring was calculated based on the vertical displacement of each basemat node.
In case of horizontal subgrade modulus, it was determined using two-thirds of the horizontal displacement since the horizontal displacements corresponding to the depth are parabolic shape. In order to consider the equivalent subgrade modulus against the embedment length, two-thirds of the maximum displacement was used for horizontal subgrade modulus. Figure 2-7 below provides justification for the horizontal subgrade modulus used. In the figure 2-7, nodes are expected to occur at the maximum horizontal displacement based on the analysis result for horizontal subgrade modulus. The trapezoid area of “A” is almost 2.65 and the quadrangle area “B” considered equivalent value is almost 2.66. So, the horizontal subgrade modulus used in the analysis is equivalent value of the embedment length of building structure.

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Figure 2-7 Maximum Deformation Sketch for Horizontal Subgrade Modulus

Table 2-4

Equivalent Subgrade Moduli of Site Profiles

Site Profile	Max. Displacement (ft)	Subgrade modulus (kcf)(1)	Remark
NI Basemat			
S1	0.028046 (Z, Vertical)	~~k_v = 35.66~~ k_v=33.14 ~ 178.70	-
	0.072731 (X, Horizontal)	k_h = 20.62	2/3 of maximum value
	0.073070 (Y, Horizontal)	k_h = 20.53	
S4	0.005769 (Z, Vertical)	~~k_v = 173.34~~ k_v=152.88~ 777.06	-
	0.023239 (X, Horizontal)	k_h = 64.55	2/3 of maximum value
	0.023245 (Y, Horizontal)	k_h = 64.53	
S8	0.001162 (Z, Vertical)	~~k_v = 860.59~~ k_v=809.01~ 2507.84	-
	0.001099 (X, Horizontal)	k_h = 1,364.88	2/3 of maximum value
	0.001123 (Y, Horizontal)	k_h = 1,335.71	
TGB Basemat			
S1	0.035069 (Z, Vertical)	kv = 28.52	-
	0.041371 (X, Horizontal)	kh = 24.17	
	0.041708 (Y, Horizontal)	kh = 23.98	
S4	0.008239 (Z, Vertical)	kv = 121.37	
	0.013406 (X, Horizontal)	kh = 74.59	
	0.013465 (Y, Horizontal)	kh = 74.27	
S8	0.001140 (Z, Vertical)	kv = 877.20	
	0.000595 (X, Horizontal)	kh = 1,680.67	
	0.000608 (Y, Horizontal)	kh = 1,644.74	

(1) Subgrade modulus (kcf) = Pressure (1ksf) / Max. Displacement (ft)

3.2.2 Material Properties

Linear-elastic material properties of concrete including modulus of elasticity, Poisson's Ratio and mass density are used in accordance with design criteria for the APR1400. The material properties of the NI structures are summarized in Table 3-1.

3.2.3 Finite Element Model

The NI structure is modeled using the following ANSYS program shell, solid, beam, and link elements:

- NI common basemat: SOLID185 elements
- RCB shell and dome: SOLID185 elements
- In-containment refueling water storage tank (IRWST) and fill concrete: SOLID185 elements
- Primary shield wall (PSW): SOLID185 elements
- Secondary shield wall (SSW): SHELL181 elements
- AB concrete wall and slab: SHELL181 elements
- AB steel column and girder: BEAM4
- Nonlinear ground (compression only): LINK180

The nominal element size in the NI common basemat is approximately 5 feet. Figure 3-1 shows the full FE model for the basemat structural analysis. In addition, the AB structure, RCB internal structure, RCB shell and dome, and basemat structure analysis models are shown in Figures 3-2 through 3-5, respectively.

3.2.4 Boundary Condition

Replaced to description on page 2 and 3 of attachment 5

Link (LINK180) elements are used for boundary conditions between the basemat structure and ground to consider the compressive behavior of the underlying subgrade. The LINK180 element is a uniaxial tension-compression element with three degrees of freedom for translation in the nodal x, y, and z directions at each node. It is useful to describe the tension-only (cable) and/or compression-only (gap) condition.

Figure 3-6 shows the LINK180 element application as the boundary condition. The compression-only option is applied to the LINK180 elements connected directionally with the basemat structure, and the fixed-boundary condition is applied to the opposite side node of the LINK180 element. Axial (tributary) areas of LINK180 elements are calculated by applying unit pressure to additional modeled shell element models that have the same geometry as the basemat model. Figure 3-7 shows the analysis model for the tributary area calculation.

3.2.5 Applied Loads

The applied loads analysis considers dead loads, live loads, post-tension loads for tendons embedded in the RCB shell and dome, containment pressure loads, pipe break load, seismic load, and buoyancy load due to groundwater.

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In order to represent the soil characteristics, the basemat analysis considered different approaches corresponding to applied loading; one approach is the soil spring approach for static loading case, another approach is the foundation media approach for dynamic loading case.

- Static Case : Link 180

In the case of a nonlinear soil spring (LINK180), it was applied for structural design member forces of basemat for load combination except seismic loading case (LC01~ LC07).
Link (LINK180) elements are used for boundary conditions between the basemat structure and ground to consider the compressive behavior of the underlying subgrade. The LINK180 element is a uniaxial tension-compression element with three degrees of freedom for translation in the nodal x, y, and z directions at each node. It is useful to describe the tension-only (cable) and/or compression-only (gap) condition

The horizontal springs are not located beneath the basemat. These are only located along the vertical side surface of the basemat since these are enough to sustain horizontal forces. The horizontal springs along the embedded walls are not considered due to uncertainty of passive soil pressure and the fact that the horizontal forces are not dominant for analysis.

Figure 3-6 shows the LINK180 element application as the boundary condition. The compression-only option is applied to the LINK180 elements connected directionally with the basemat structure, and the fixed-boundary condition is applied to the opposite side node of the LINK180 element. Axial (tributary) areas of LINK180 elements are calculated by applying unit pressure to additional modeled shell element models that have the same geometry as the basemat model. Figure 3-7 shows the analysis model for the tributary area calculation.

- Seismic Case : Foundation Media Model (Solid 185)

Foundation model was used for structural design member forces of the basemat for load combinations including seismic loading. For the material characteristic of the foundation model, the strain-compactable shear wave velocity was utilized to calculate the dynamic elastic modulus for soil stiffness in the foundation media model based on the following equation.

$$E = \rho V_S^2 \times [2 \times (1 + \mu)]$$

Figure 3-13 shows the foundation media model application as the boundary condition.

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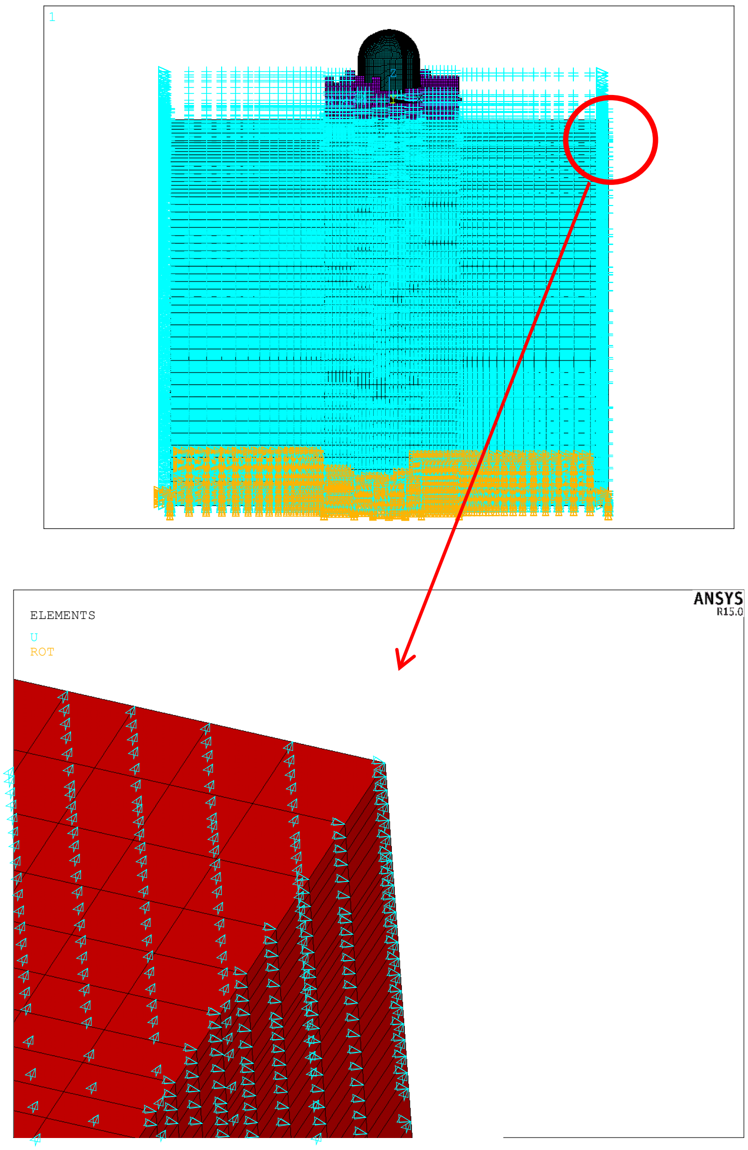

Figure 3-13 Boundary Condition of Foundation Media Model

gradient is approximately 50 °F and a uniform temperature change is less than 10 °C (50 °F). The analysis of the foundation mat is performed by a three-dimensional finite element structure model, and the forces and moments determined in the analysis are input to the structural design.

The analysis and design of the foundations consider the effects of potential mat uplift, with particular emphasis on differential settlements of the basemat.

The foundation of the seismic Category I structure analysis is performed considering a soil/rock properties beneath the foundation as a nonlinear spring elements. The model is capable of determining the possibility of uplift of the basemat from the subgrade during postulated SSE events. The vertical spring at each node in the analytical model acts in compression only. The horizontal springs are active when the vertical spring is in compression and inactive when the vertical spring lifts off.

3.8.5.4.2 Analyses of Settlement during Construction

The basemat is analyzed and designed to consider settlements in various phases of construction.

The basemat is sufficiently reinforced to control stresses until the concrete placement of basemat walls and containment internal structure is completed.

3.8.5.4.3 Design Summary Report

A design summary report for the basemats is presented in Appendix 3.8A, where the design of representative critical sections of the structures is described.

The evaluation considering the deviations of as-procured or as-built construction to the design will be performed with the acceptance criteria, as described in Technical Report, APR1400-E-S-NR-14006-P (Reference 40).

3.8.5.5 Structural Acceptance Criteria

The structural acceptance criteria for the containment and other seismic Category I structures excluding the reactor containment building are described in Subsections 3.8.1.5 and 3.8.4.5, respectively. In particular, the acceptance criteria for the stability of seismic Category I structures are checked together with the structural acceptance criteria against the

The detailed description of stability, uplift, settlement, bearing pressure check is presented in Appendix 3.8A. A d etailed description of analysis for design and stability check is presented in Technical Report, APR1400-E-S-NR-14006-P (Reference 40).

selected to check the different settlements. Table 3.8A-17 shows the differential settlements at site profiles 1, 4, and 8. The maximum differential settlements per 15.24 m (50 ft) at site profiles 1, 4, and 8 are 4.470 mm (0.176 in.), 1.829 mm (0.072 in.), and 0.940 mm (0.037 in.), respectively.

For the differential settlements by seismic loading, the displacements of the basemat relative to the free field are calculated at the 50 nodes as shown in Figure 3.8A-19. Figures 3.8A-20 through 3.8A-22 show the Z-displacement of the basemat relative to the free field according to the site profiles. It is noted that these results are obtained from the analysis of seismic loading only (not including dead load). The maximum differential settlement by seismic loading is approximately 1.829 mm (0.072 in.).

The maximum probable differential settlements of the APR1400 NI common basemat are 4.470 mm (0.176 in.) and 1,829 mm (0.072 in.) in the static and seismic loading conditions. In addition, the differential settlement between the NI and TGB basemat is checked. The maximum differential settlements corresponding to soil sites (Soil #1, Soil #4, and Soil #8) of the NI and TGB basemat are 2.46 mm (0.091 in.), 6.35 mm (0.250 in.), and 0.46 mm (0.018 in.), respectively.

Added 3.8A.1.4.2.3.5 subsection in page 3 of attachment 6

3.8A.1.4.2.4 Conclusion

The basemat concrete section strengths determined from the ASME criteria are sufficient to resist the design basis loads. It is feasible to design and construct the structural components considered. The assumptions envelop the given parameters so that the design presented is adequate for any specific site conditions within those parameters.

3.8A.1.4.3 Internal Structures

3.8A.1.4.3.1 Primary Shield Wall

3.8A.1.4.3.1.1 Description

The PSW is a massive rectangular concrete structure, 18.80 m (61 ft 8 in.) long by 11.43 m (37 ft 6 in) wide, with cavities consisting of the following:

a. Vertical chase, 2.03 m (6 ft 8 in.) by 5.18 m (17 ft 0 in.), for in-core instrumentation (ICI) guide tubes from the seal table at the bottom of the refueling pool, El. 130 ft 0 in, down to the bottom of the ICI tunnel at El. 69 ft 0 in.

3.8A.1.4.2.3.5 Bearing Pressure

The bearing pressures of the NI common basemat is evaluated for soil profile S01 (weakest), S04 (moderate), S08 (strongest) under static and dynamic loading conditions.
The model for the superstructure, NI common basemat, and soil used for the bearing pressure evaluation is identical as that used for the design described in 3.8A.1.4.2.3.

The maximum static bearing pressure is determined as the soil spring forces divided by the tributary area of the soil spring under the dead and live load conditions. The maximum dynamic bearing pressure is determined as the contact pressure between the basemat and foundation media model for statics plus seismic loads. The maximum static bearing pressure of 937.1kPa (19,570 lb/ft^2) in the APR1400 NI common basemat is obtained from the basemat analysis results of soil profile S01. A value of 20ksf is defined as the allowable static bearing demand to provide an additional margin of safety at the site. The maximum dynamic bearing pressure of 2586.2kPa (54,010 lb/ft^2) is obtained from the basemat analysis results of soil profile S08. A value of 60ksf is defined as the allowable dynamic bearing demand to provide an additional margin of safety at the site.

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Settlement Check

Differential settlements are divided by the differential settlement within the EDG building basemat and the differential settlement within DFOT building. For the differential settlements within the each basemat, the static (dead and live loads) loading case is calculated.

The distance of approximately 15.24 m (50 ft) is selected to check the differential settlement. Table 3.8A-39 shows the differential settlements of each soil profile. The maximum differential settlement for EDG building per 15.24 m (50 ft) is 4.52 mm (0.18 in.). The maximum differential settlement for DFOT building per 15.24 m (50 ft) is 7.21 mm (0.28 in.).

The differential settlement of each soil profiles between the NI common basemat and EDG building is checked. The maximum differential settlement between the NI common basemat and EDG building is 58.14 mm (2.29 in.).

The differential settlement of each soil profiles between the NI common basemat and DFOT building is checked. The maximum differential settlement between the NI common basemat and DFOT building is 53.32 mm (2.10 in.).

The differential settlement of each soil profiles between the EDG building and DFOT building is checked. The maximum differential settlement between the EDG building and DFOT building is 4.83 mm (0.19 in.).

Figure 3.8A-58 and Figure 3.8A-59 show the node locations at the bottom of the EDG & DFOT basemat for checking the settlements. The analysis of multiple of settlements (long and short term) will use these nodes.

Added description in page 5 of attachment 6

3.8A.3.4.2 Shear Walls

Description

The shear walls and slabs of the EDG building representing the primary lateral load-resisting system are designed against seismic or extreme wind-related loads. The concrete slab distributes lateral forces through diaphragm action to the shear walls as in-plane loads in proportion to the relative stiffness of the shear walls. These in-plane shear forces are

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Bearing Pressure Check

The bearing pressures of the EDG building basemat and DFOT basemat are evaluated for soil profile S (weakest), S04 (moderate), S08 (strongest) under static and dynamic loading conditions.
The analysis and design methods used for bearing pressure evaluation is identical as that used for the design described in 3.8A.3.4.1.

The static and dynamic bearing pressure is determined as the soil spring forces divided by the tributary area of the soil spring under static (dead and live load) and dynamic conditions. The maximum static bearing pressure of 396.0kPa (8,270 lb/ft^2) in the EDGB basemat is obtained from the basemat analysis results of soil profile S08. The maximum static bearing pressure of 352.9kPa (7,370 lb/ft^2) in the DFOT basemat is obtained from the basemat analysis results of soil profile S08. The maximum dynamic bearing pressure of 861.37kPa (17,990 lb/ft^2) in the EDGB basemat is obtained from the basemat analaysis results of soil profile S08. The maximum dynamic bearing pressure of 866.7kPa (18,100 lb/ft^2) in the DFOT basemat is obtained from the basemat analysis results of soil profile S08.

Table A-5

Differential Settlement between NI Basemat and DFOT Room (Static Loading)

	Max. Settlement (inches)		
	Soil 01	Soil 04	Soil 08
NI Basemat	3.959	0.821	0.172
DFOT Room Basemat	1.860	0.582	0.047
Differential Settlement	2.099	0.239	0.125

Table A-6

Differential Settlement between EDGB Basemat and DFOT Room(Static Loading)

	Max. Settlement (inches)		
	Soil 01	Soil 04	Soil 08
DFOT Room Basemat	1.860	0.582	0.047
EDGB Basemat	1.670	0.537	0.059
Differential Settlement	0.190	0.045	0.012

Table A-7

Soil Pressure of EDGB & DFOT Room Basemat

	Load Case	Max. Soil Pressure (ksf)		
		Soil 1	Soil 4	Soil 8
EDGB	Static Case	4.83	5.08	8.10
	Dynamic Case	7.61	8.06	14.02
DFOT	Static Case	5.38	5.51	6.41
	Dynamic Case	4.22	4.21	5.19

* Bearing pressure (ksf) = Soil spring reaction (kips) / Tributary area (ft^2)

Replaced

	Load Case	Max. Soil Pressure (ksf)		
		Soil 1	Soil 4	Soil 8
EDGB	Static Case	4.92	5.17	8.27
	Dynamic Case	9.70	10.27	17.99
DFOT	Static Case	5.98	6.09	7.37
	Dynamic Case	13.98	14.24	18.10

Static Case: D+L
Dynamic Case: Design load combination including SSE load

Table 2.1-1 (2 of 4)

Extreme Wind	
50-Year 3-Second Wind Gust Speed	64.8 m/s (145 mph)
Importance Factors	1.15 (2)
Tornado	
Maximum Tornado Wind Speed	102.8 m/s (230 mph)
Translational Speed	20.6 m/s (46 mph)
Maximum Rotational Speed	82.2 m/s (184 mph)
Radius of Maximum Rotational Speed	45.7 m (150 feet)
Pressure Drop	8.274 kPa (1.2 psi)
Rate of Pressure Drop	3.447 kPa/s (0.5 psi/s)
Missile Spectra	Table 2 (Region I) of NRC RG 1.76 (2007)
Hurricane	
Maximum 3-Second Wind Gust Speed	116 m/s (260 mph)
Missile Spectra	Table 1 of NRC RG 1.221 (2011)
Soil Properties	
Allowable Static Bearing Capacity for Seismic Category I Structures (Dead and Live Load)	The allowable static bearing capacity, including a factor of safety appropriate for the design load combinations, shall be greater than or equal to the maximum static bearing demand of 718.2 kPa (15 ksf). The allowable static bearing capacity is the value of ultimate bearing capacity divided by 3.0.
Allowable Dynamic Bearing Capacity for Seismic Category I Structures (Design Load Combination including SSE Load)	The allowable dynamic bearing capacity, including a factor of safety appropriate for the design load combinations, shall be greater than or equal to the maximum dynamic bearing demand of 2,872.8 kPa (60 ksf). The allowable dynamic bearing capacity is the value of ultimate bearing capacity divided by 2.0.
Minimum Factor of Safety for Slope on Static Condition	1.5
Minimum Factor of Safety for Slope on Dynamic Condition (SSE)	1.2
Minimum Shear Wave Velocity	304.8 m/s (1,000 ft/sec)

Table 2.0-1 (3 of 4)

Parameter Description	Parameter Value
Certified Seismic Design Response Spectra (CSDRS) Referencing SSE	See Figures 2.0-1 and 2.0-2
Hard Rock High Frequency (HRHF) Response Spectra(4)	0.46g peak ground acceleration See Figures 2.0-3 and 2.0-4
Tectonic and Non-tectonic Surface Deformation Potential	See Subsection 2.5.3
Allowable Static Bearing Capacity for Seismic Category I Structures (Dead and Live Load)	The allowable static bearing capacity, including a factor of safety appropriate for the design load combinations, shall be greater than or equal to the maximum static bearing demand of 718.2 kPa (15 ksf). The allowable static bearing capacity is the value of ultimate bearing capacity divided by 3.0.
Allowable Dynamic Bearing Capacity for Seismic Category I Structures (Design Load Combination including SSE Load)	The allowable dynamic bearing capacity, including a factor of safety appropriate for the design load combinations, shall be greater than or equal to the maximum dynamic bearing demand of 2,872.8 kPa (60 ksf). The allowable dynamic bearing capacity is the value of ultimate bearing capacity divided by 2.0.
Minimum Factor of Safety for Slope on Static condition	1.5
Minimum Factor of Safety for Slope on Dynamic condition (SSE)	1.2
Minimum Shear Wave Velocity	304.8 m/s (1,000 ft/s)
Maximum Dip Angle for Soil Uniformity	20 degrees
Liquefaction Potential	See Subsection 2.5.4.8
Maximum Allowable Differential Settlement inside Building	12.7 mm (0.5 in.) per 15.24 m (50 ft) in any direction for seismic Category I structures under static and seismic load
Maximum Allowable Differential Settlement between Buildings	76.2 mm (3.0 in.) between NI Common Basemat and EDG Building & DFOT Building 12.7 mm (0.5 in.) under static and seismic load
Minimum Soil Angle of Internal Friction	Greater than or equal to 35 degrees below the footprint of the seismic Category I structures at their excavation depth
Slope Failure Potential (yes/no)	No
Backfill Material Density	2.2 g/cm^3 (137 pcf)

a non-radial mesh pattern, run down to the tendon gallery on the opposite side, and are anchored at each end in the tendon gallery, as shown in Figure 3.8-4. As the tendon gallery is located entirely within the NI common basemat, it is analyzed and designed as a part of the common basemat. The codes and standards, loads and load combinations, design and analysis procedures, and structural materials for the tendon gallery are the same as those for the NI common basemat, and are described in Subsections 3.8.5.2 through 3.8.5.4, Subsections 3.8A.1.2.1 and 3.8A.1.2.3, and the following Subsections 3.8A.1.4.2.2 through 3.8A.1.4.2.4. For the analysis model, design section forces and design results of the NI common basemat, including the tendon gallery, are presented in Tables 3.8A-5 through 3.8A-13 and Figures 3.8A-13 through 3.8A-17.

The reactor containment basemat is reinforced at the top and bottom with layers of reinforcing steel bars. The reinforcing bars are arranged in the radial and hoop directions for top layers and in the orthogonal directions for bottom layers. The reinforcement at the upper portion of the tendon gallery is in the radial and hoop directions, and the reinforcement at the lower portion of the tendon gallery is in the rectangular pattern aligned with the plant NS and EW directions as shown in Figures 3.8A-16 and 3.8A-17, and Table 3.8A-12.

3.8A.1.4.2.2 Load Combinations Considered

The following loading combinations are critical for the analysis and design of the basemat:

a. Test: $1.0D + 1.0L + 1.0L_h + 1.0F + 1.0P_t$

b. Normal: $1.0D + 1.0L + 1.0L_h + 1.0F$

c. Severe: $1.0D + 1.3L + 1.3L_h + 1.0F$

d. Abnormal: $1.0D + 1.0L + 1.0L_h + 1.0F + 1.5P_a$

e. Abnormal/Extreme: $1.0D + 1.0L + 1.0L_h + 1.0F + 1.0P_a + 1.0Y_r + 1.0E_s$

3.8A.1.4.2.3 Analysis and Design Procedures

~~The design of the APR1400 adheres to a standardized design concept and can be constructed on various sites, including rock site even soil site.~~

This description will be modified as shown page 4 of attachment 8.

~~Among the nine soil profiles and one fixed-base condition, three profiles (Soil Profiles #1, #4, and #8) are considered. Soil Profiles #1, #4, and #8 denote weak, moderate, and strong soil properties, respectively.~~

~~Although only three soil profiles (upper-, medium-, and lower-bound soil cases) are considered in the basemat structural analysis, the superstructure analysis results from enveloped seismic loading in 10 analysis cases are conservatively used in the basemat structural analysis. Therefore, it is concluded that the basemat structural analyses of the three soil profiles cover all of the basemat analyses of the soil categories given for the APR1400.~~

~~The load combinations for the basemat structure are summarized in Section 3.8A.1.4.2.2. A total of 36 load combinations (12 combinations × 3 soil profiles) were examined for the RCB basemat structure.~~

~~The NI common basemat structure is analyzed using the ANSYS program FE computer model. The NI common basemat structure model includes the containment wall, dome, internal structures, AB, and common basemat foundation structure.~~

~~The FEM for superstructures, including the RCB wall and dome, RCB internal structure, and AB structure, are connected to the solid basemat model to simulate the stiffness effect of superstructures to the basemat. The FEM consists of a total of 317,373 nodes and 313,101 elements. Figure 3.8A-12 shows the full FEM for the basemat structural analysis. The AB structure, RCB internal structure, RCB wall and dome, and basemat structure FEMs are shown in Figure 3.8A-13.~~

~~The LINK180 element in ANSYS was used as a boundary condition between the basemat structure and soil to consider the compressing behavior of the underlying subgrade. The compression-only option was applied to the LINK180 elements of the ANSYS connected direction with the basemat structure, and the fixed boundary condition was applied to the other end side node of LINK180 element, as shown in Figure 3.8A-14. Axial (tributary) areas of each LINK180 element were calculated by applying unit pressure to the additional modeled shell element model, which has the same geometry as the basemat model.~~

~~The dead load of the basemat structure was calculated by applying vertical acceleration to the basemat structure. In addition, the reactions calculated from the analysis results of each superstructure are applied as nodal force to the basemat structure. Buoyancy loads (L_h) due to underground water are applied to the bottom of the basemat structure. Probable~~

This description will be modified as shown page 4 of attachment 8.

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~~maximum water level used for the buoyancy loads calculation is El. 100 ft 0 in (ground level) for added conservatism. For SSE loads, the enveloped seismic loading from 10 analysis cases is conservatively used in each superstructure. The reactions from these analysis results are applied as nodal force to the basemat structure using the 100-40-40 effect of the three directions of seismic motion in which one component is taken at 100 percent of its maximum value and the others are taken at 40 percent of their maximum values.~~

The analysis results are expressed as the normal stresses and the shear stresses of solid elements. The stresses of solid elements are filed with respect to the rectangular and cylindrical coordinate systems to fit with the arrangements of reinforcement.

To envelop the flexural and shear reinforcement for the 36 load combinations, the RCB basemat is divided into eight design sections as represented in Table 3.8A-5. Figure 3.8A-15 shows design sections for the containment basemat.

Tables 3.8A-6 through 3.8A-9 show the calculated section forces and moments for the design. The calculated design forces and moments are used as input in the concrete section design program DARTEM for the design of flexural reinforcement and shear reinforcement. The design of the concrete sections is based on the ASME Section III, Division 2.

3.8A.1.4.2.3.1 Design Summary

The results on the design of the flexural and shear reinforcement are summarized in Tables 3.8A-10 through 3.8A-13. For the flexural reinforcement, it is confirmed that the maximum stresses of the provided reinforcement do not exceed the allowable stresses for both the service and factored load conditions. For the shear reinforcement, it is confirmed that the amounts of provided reinforcement are sufficient to meet the demands of the required reinforcement for each design section. The margins of safety of the flexural and shear reinforcement and concrete stresses are shown in Table 3.8A-10 and 3.8A-11. The design envelops the given parameters so that the design is adequate for any specific site conditions within those parameters. Figures 3.8A-16 and 3.8A-17 show the rebar arrangement for the basemat of the RCB.

3.8A.1.4.2.3.2 Stability Check

The NI common basemat structure is evaluated for stability against overturning, sliding, and flotation. The calculated factors of safety against overturning, sliding, and flotation for

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3.8A.1.4.2.3 Analysis and Design Procedures

The design of the APR1400 adheres to a standardized design concept and can be constructed on various sites, including soil and rock sites.
Among the nine soil profiles, three profiles (Soil profiles #1, #4, and #8) are considered as weak, moderate, and hard soil profiles for the NI common basemat based on comparison to subgrade moduli of each soil profile.

Although only three soil profiles (upper-, medium-, and lower-bound soil cases) are considered in the basemat structural analysis, the superstructure analysis results from enveloped seismic loading in 10 analysis cases are conservatively used in the basemat structural analysis. Therefore, it is concluded that the basemat structural analyses of the three soil profiles cover all of the basemat analyses of the soil categories given for the APR1400.

The load combinations for the basemat structure are summarized in Section 3.8A.1.4.2.2.
A total of 288 load combinations (96 combinations × 3 soil profiles) were examined for the NI common basemat structure including phasing consideration of superstructures.
The NI common basemat structure is analyzed using the ANSYS program FE computer model. The NI common basemat structure model includes the containment wall, dome, internal structures, AB, and common basemat foundation structure.

The FEM for superstructures, including the RCB wall and dome, RCB internal structure, and AB structure, are connected to the solid basemat model to simulate the stiffness effect of superstructures to the basemat. The FEM consists of a total of 317,373 nodes and 313,101 elements. Figure 3.8A-12 shows the full FEM for the basemat structural analysis.
The AB structure, RCB internal structure, RCB wall and dome, and basemat structure FEMs are shown in Figure 3.8A-13.

In the case of soil stiffness, the distributed springs and soil finite elements are used for appropriate loading conditions. In the case of static loading, the distributed springs (LINK180) applied different subgrade moduli is used to consider boussinesq effect. The compression-only option was applied to the LINK180 elements of the ANSYS connected direction with the basemat strucutre, and the fixed boundary condition was applied to the other end side node of LINK180 element, as shown in Figure 3.8A-14. Axial (tributary) areas of each LINK180 element were calculated by applying unit pressure to the surface, which has the same geometry as the basemat model.

In the case of load combination including the seismic case, the soil finite elements are used to represent the overall dynamic foundation stiffness. In a manner consistent with the seismic SSI analysis, the material for the foundation media model is calculated based on strain-compatible shear wave velocity from SASSI.

The reactions from the analysis results of the each superstructure except seismic load are used as nodal forces and moments to the basemat. However, seismic loads from superstructures and basemat are applied by using the equivalent static acceleration method to the basemat. Torsional load is separately considered in the separate basemat analysis. The results from this separate analysis are combined by the absolute sum method to the results from the seismic load analysis. Buoyancy loads (L_h) due to groundwater are applied to the bottom of the basemat structure. The probable maximum water level used for the buoyancy loads calculation is El. 100 ft 0 in (ground level) for added conservatism.

Both the linear case (fully connected basemat to foundation) and nonlinear case (no connectivity between basemat and foundation when basemat uplift occurs) are included in the design. Based on a comparison between member forces and between the nonlinear case and the SSI analysis, 96 cases are sufficient to encompass all permutations caused by the superstructure. Therefore, the conservative design of the basemat is performed under linear and nonlinear condition since it bounds the problem of no uplift/uplift. The envelop of these two cases is used for the design of the members. Under the nonlinear condition, 96 were performed cases using the 100-40-40 method , considering different phasing of three superstructures. A detailed description and comparison results are presented in Technical Report, APR1400-E-S-NR-14006-P (Reference 40).

the load combinations meet the criteria of Section II of SRP 3.8.5 as shown in Table 3.8A-14.

The sliding and overturning factors of safety are determined using load combination containing dead load (D), SSE (E_s), and buoyant load at normal (H_e). The floatation factor of safety is determined based on dead load (D) and buoyant force at flood (H_s).

For calculation of buoyant load at normal (H_e), the design groundwater level is applied, while the extreme groundwater level is applied to calculate the buoyant force at flood (H_s). The design groundwater level is El. 96 ft 8 in. The extreme groundwater level is the same as plant grade level (El. 98 ft 8 in.) considering probable maximum flood.

In the earthquake load, axial force, shear force, and moment due to horizontal and vertical excitation of the structure are obtained from seismic analysis. Since seismic load governs over wind load, stability checks are not considered under wind load. A summary of overturning, sliding, and flotation check is provided in Table 3.8A-15.

3.8A.1.4.2.3.3 Basemat Uplift Check

The ground contact uplift ratio between the basemat and soils is carried out to provide reasonable assurance that the linear soil-structure interaction (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. Among the results from the NI common basemat analysis, the load combination cases, which are shown, the uplift phenomena are considered for uplift check. Table 3.8A-16 shows the uplift area ratios of NI common basemat. The APR1400 NI common basemat has an 80 percent or more contact area during basemat uplift, and it can be concluded that the contact area would be acceptable.

Replace with paragraph "A" in page 2 of attachment 9.

3.8A.1.4.2.3.4 Settlement Check

Differential settlements are divided by the differential settlement within the NI common basemat and the differential settlement between the NI basemat and the turbine generator building (TGB). For the differential settlements within the NI common basemat, the static (dead and live loads) and seismic loading cases are calculated.

Figure 3.8A-18 shows the node location at the bottom of the NI common basemat for checking the settlement. The nodes within a distance of approximately 15.24 m (50 ft) are

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The ground contact ratio between the basemat and soil is carried out to provide reasonable assurance that the linear soil-structure interaction (SSI) analysis remains valid. The ground contact ratio is defined as combining the stresses obtained from soil spring under static loads with stresses obtained from time histories under seismic loads. Table 3.8A-16 shows the ground contact ratio of NI common basemat. The APR1400 NI common basemat has an 80 percent or more contact area during basemat uplift, and it can be concluded that the contact area would be acceptable.