PMHarrisCOL PEmails

From: Ricardo Rodriguez
Sent: Wednesday, June 04, 2008 2:54 PM
To: Alice Stieve
Subject: Update (disregard that last email)
Attachments: Summary of Application-rev1.doc; Possible RAIs.doc

This one has the attachments…sorry!

From: Ricardo Rodriguez
Sent: Wednesday, June 04, 2008 2:53 PM
To: Alice Stieve
Subject: Update (disregard that last email)

From: Ricardo Rodriguez
Sent: Wednesday, June 04, 2008 2:52 PM
To: Alice Stieve
Subject: Update

Alice:

I'm attaching a revised version of the Summary and a list of possible RAIs found by me. I have another list of RAIs that Cliff gave me some time ago that I still have to go over them with Wayne. Do you want me to give you a copy of that list? Also, I'll be taking a couple of days off, but I should be back next Wednesday. Just to let you know.

Thanks a million!

Ricardo E. Rodriguez EIT
Geotechnical Engineer, NSPDP
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O-11F16
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415-3185

Hearing Identifier: ShearonHarris_COL_Public
Email Number: 23

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Subject: Update (disregard that last email)
Sent Date: 6/4/2008 2:53:42 PM
Received Date: 6/4/2008 2:53:44 PM
From: Ricardo Rodriguez

Created By: Ricardo.Rodriguez@nrc.gov

Recipients:
"Alice Stieve" <Alice.Stieve@nrc.gov>
Tracking Status: None

Post Office: HQCLSTR02.nrc.gov

Files	**Size**	**Date & Time**
MESSAGE	823	6/4/2008 2:53:44 PM
Summary of Application-rev1.doc	163834	
Possible RAIs.doc	24058	

Options
Priority: Standard
Return Notification: No
Reply Requested: No
Sensitivity: Normal
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2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS (RELATED TO RG 1.206, SECTION 2.5.4, "Stability of Subsurface Materials and Foundations")

2.5.4.1 Introduction/Overview/General

Section 2.5.4 of this PSER provides a summary of the SHNPP FSAR section 2.5.4 on the stability of subsurface materials and foundations for the Shearon Harris Nuclear Power Plant Units 2 and 3 sites (HAR 2 and HAR 3, respectively). Section 2.5.4.2 of this PSER provides a summary of relevant geologic and seismic information contained in FSAR Section 2.5.4 of the Harris Nuclear Power Plant Units 2 and 3 COL applications. PSER Section 2.5.4.3 provides a summary of the regulations and guidance used by the applicant to perform their investigation. PSER Section 2.5.4.4 provides a review of the staff's evaluation of FSAR Section 2.5.4, including any requests for additional information, any open items, and any confirmatory analyses performed by the staff. PSER Section 2.5.4.5 discusses any post combined license activities. Finally, PSER Section 2.5.4.6 provides an overall summary of the applicant's conclusions, as well as the staff's conclusions, restates any bases covered in the application, and confirms that regulations were met or fulfilled by the applicant.

2.5.4.2 Summary of Application

2.5.4.2.1 <u>Geologic Features</u>

FSAR section 2.5.4.1 assesses the geologic processes and geologic features that could be the source of permanent ground deformation or foundation instability if present at the HAR 2 and HAR 3 sites. The applicant first presented a summary of the subsurface conditions found at the site. The applicant emphasized issues such as: areas of actual or potential subsurface subsidence, solution activity, uplift, or collapse; zones of alteration, irregular weathering, or structural weakness; unrelieved stresses in bedrock; rocks or soils that may become unstable; history of deposition and erosion; and estimates of preconsolidation pressures.

Summary of Subsurface Conditions at HAR 2 and HAR 3

FSAR subsection 2.5.4.1.1 provides a summary of the subsurface conditions found at the site. Emphasis if on the different kinds of soils and rocks discovered during the course of field work that are relevant to the geology of the area.

The applicant stated that the HAR 2 and HAR3 nuclear islands will be founded at a subgrade elevation of 220 ft amsl. The subsurface profile consists of a gradual transition of soil to weathered rock and to fresh and unweathered rock. The soil profile consists of primarily lean clay, sand, and silt that rests over the parent bedrock. Soils depths range from 5 to 15 ft and 10 to 25 ft at HAR 2 and HAR3, respectively. Rock types are field classified as: sandstone, siltstone, claystone, shale and conglomerate.

The applicant made the following key observations:

- The depth to sound rock is shallower at HAR 2 than at HAR 3 because surficial soil and weathered rock were removed in the 1970's to create the existing site grade during construction of the HNP
- Bedrock found consists mainly of reddish-brown siltstone and reddish-brown to gray sandstone
- Rock is predominantly sound below the nuclear island foundation elevation of 220 ft

There were isolated intervals of altered rock less than 3 ft thick found, except in borehole BPA-6 where larger intervals were found. Also, the applicant found evidence of thin intact clay seams bounded by surrounding bedrock that showed signs of bedding. Regarding the seams, it was stated by the applicant that some of them were likely formed by weathering of the parent rock while others were likely formed by the coring process itself. The applicant concluded that these seams have never fully lithified. Slickensides were found by the applicant in the rock cores at numerous locations in the form of oriented grains on bedding surfaces or within mechanical rock breaks. The applicant concluded that these slickensides were formed due to compaction and settlement of the parent bedrock, therefore they are not an indication of tectonic movements.

The applicant calculated the dip of rock strata by three methods (see Table 2.5.4-202):

- Triangulation of contact elevations of marker beds
- Correlation of common shear wave velocity patterns
- Correlation of bedding features identified in oriented acoustic televiewer holes.

The applicant concluded that:

- Variations in dip under both nuclear islands are due to lateral deposition changes
- Variations in dip between the nuclear islands footprints are the result of folding or wrapping
- The dip magnitude at HAR 2 ranges from 6 to 9 degrees to the east and at HAR 3 from 19 to 23 degrees to the east-southeast.

Subsidence, Solution Activity, Uplift, or Collapse

FSAR subsection 2.5.4.1.2 briefly discusses the potential for subsidence, solution activity, uplift, or collapse in the HAR site. The applicant stated that there is no risk of subsidence or collapse due to solution due to the lack of carbonates and evaporates in the site area (see FSAR subsection 2.5.1.2.5 for more details).

Zones of Alteration, Irregular Weathering, or Structural Weakness

FSAR subsection 2.5.4.1.3 describes zones of alteration, irregular weathering, and structural weaknesses encountered at the site as found by the applicant.

Based on borehole data, the FSAR notes that isolated intervals of very weak rock and clay seams are found below the top of sound rock. The applicant concluded that due to their limited thickness (less than 3 ft) and sub horizontal orientation of these layers that they do not present a weakness in the overall rock mass for foundation purposes. High angle joints that cross bedding planes are also described in the FSAR; however the applicant stated that these were tight and lacking significant effects of weathering or infilling. In borehole BPA-6, the applicant observed that a total clay thickness of approximately 9 ft was found. Comparison between boreholes at the site determined that UCS results from intact rock core samples from BPA-6 were lower than samples obtained from boreholes located under HAR 2 or HAR 3. The applicant stated that detailed exploration and mapping of the nuclear islands excavations will be performed after excavation and prior to construction in order to ascertain that possible fault features are not present in the vicinity of the proposed safety related buildings as indicative of the borehole conditions found in BPA-6.

Unrelieved Stresses in Bedrock

FSAR subsection 2.5.4.1.4 notes that no evidence of unrelieved stresses in bedrock were found by the applicant (see FSAR subsection 2.5.1.2.5)

Rocks or Soils that May Become Unstable

FSAR subsection 2.5.4.1.5 states that the potential hazard from rocks or soils that may become unstable is low. The applicant concluded that:

- Rock strength varies with depth and between boreholes based on results of laboratory tested samples of clay intervals that indicate low water content less than its plastic limit, which is typical of low compressibility soils (see FSAR subsection 2.5.4.10.3).
- Intervals of isolated weak rocks and clays were included in the settlement and stability analyses
- Soil liquefaction will not be an issue because the structures will be founded on sound rock or concrete fills over sound rock. Granular or cohesive fill adjacent to nuclear walls will be compacted (see FSAR subsection 2.5.4.5.3).

- High angle joints and fractures will not affect the stability of the structures due to their tightness (see FSAR subsection 2.5.1.2.4 and 2.5.4.4.2.2).

History of Deposition and Erosion

FSAR subsection 2.5.4.1.6 briefly summarizes the history of deposition and erosion at the HAR sites. FSAR subsection 2.5.1.2 describes in more detail the deposition history. The applicant described the rocks near the surface as deposited in a half graben formed in Triassic time that was later filled with sediment from other areas and later eroded to its present state.

Estimates of Preconsolidation Pressure

FSAR subsection 2.5.4.1.7 describes the estimated preconsolidation pressure. The applicant stated that residual soils found at the site are highly overconsolidated and are consistent with consolidation tests results performed in the laboratory (see FSAR subsection 2.5.4.2.3.2). Weathering processes and erosion are the reasons for the reduction in the past maximum preconsolidation pressure, according to the applicant.

(NEED MORE INFO ON PRECONSOLIDATION PRESSURE VALUES OBTAINED)

2.5.4.2.2 <u>Properties of Subsurface Materials</u>

Description of Investigation Activities

FSAR subsection 2.5.4.2.1 describes field investigation activities and laboratory tests completed for the FSAR. The applicant summarized the different field and laboratory tests conducted alongside a detailed explanation of each activity. The applicant stated that Regulatory Guides 1.132 and 1.138 were followed and that any changes to the planned activities made to address observations during the investigation were described.

83 boreholes were completed in distribution over the site in order to obtain a representative characterization of subsurface conditions in the site and to obtain soil and rock samples for various tests (see Figures 2.5.4-201 and 2.5.4-202 in the FSAR). The dataset included:

- 50 boreholes near the planned AP1000 structures (BPA – series)
- 18 general characterization boreholes around the HAR site area (BGA – series)
- 8 boreholes at planned cooling tower locations (BCTA – series)
- 7 boreholes at conveyance lines and at the intake structure (BCA-series)

Regarding the location of said boreholes, the applicant stated that they are sufficient to characterize foundation performance of safety related structures and to comply with NRC Regulatory Guides. The boring plan included criteria for completion of borehole depths chosen in order to comply with NRC Regulatory Guide 1.1.32. The FSAR describes the criteria used as:

- Principal boreholes at safety related structures were advanced below elevation 192 ft amsl, which is 38 ft deeper than the proposed nuclear island foundation depth.
- Principal boreholes at safety related structures were penetrated to a depth of 20 ft below the top of sound rock
- One borehole at each unit was advanced to a depth equivalent to the maximum width of the nuclear island foundation (160 ft) below the subgrade elevation.

The applicant observed that the top of sound bedrock was reached at a shallower depth than the proposed nuclear island subgrade elevation on both sites. During the field investigation, the applicant decided to extend the depths of some principal boreholes in order to better characterize isolated intervals of weak rock. The applicant chose mud rotary drilling for their soil boreholes, NQ or HQ size double-tube rotary wireline core barrels for rock coring, disturbed soil samples were collected by SPT sampling, and undisturbed soil samples were collected by Pitcher tube and Shelby tube. These methods were considered appropriate by the applicant to provide reliable data of subsurface conditions.

To characterize soil and rock types; and soil consistency, the applicant kept a record of field observations on soil boring and rock coring logs. Rock soundness and strength were established by the applicant based on the Rock Quality Designation (RQD), the R-value indicator of strength and field point load tests.

The applicant also implemented the following programs and methods for soil and/or rock characterization in-situ:

- Borehole and surface geophysical surveys
- Rock Pressuremeter Testing to provide information on the rock mass modulus of in-situ rock.
- Groundwater Monitoring Wells (twenty in total) and Hydraulic Conductivity Tests (in two boreholes) to monitor seasonal fluctuations in groundwater elevations and to evaluate hydraulic conductivity of soil and rock.

Laboratory tests on soil and rock samples that are reported in FSAR subsection 2.5.4.2.1.6:

- Unconfined Compression Tests (UCS) were performed on 80 intact rock core samples.

- Sixteen rock samples were submitted for petrographic examination to provide a detailed assessment of the lithology and mineralogy of the rocks at the HAR site.
- Soil index tests were performed on 26 soils samples collected by Pitcher tube and Shelby tube samples
- Soil consolidation tests were performed on 6 samples
- Soil Strength was tested using the following:
    - Unconfined compression (UC) tests on 13 samples
    - Unconsolidated-undrained (UU) triaxial compression tests on 9 samples
    - Consolidated-undrained (CU) triaxial compression tests on 3 samples.

The applicant stated that the criteria used to collect rock core samples was: to target elevation ranges, to characterize different rock types, to span the range of apparent rock core soundness, and to identify locations of relatively weak rock. On the other hand, laboratory soil samples were collected, according to the applicant, to provide data on soils that will be left in place and to support slope stability data.

Soil and Rock Engineering Properties from Field Investigations

FSAR subsection 2.5.4.2.2 describes a variety of field tests and observations performed in order to obtain quantitative and qualitative information. Field observations and tests made by the applicant included:

- Standard Penetration Tests Blow Counts (N) – indicator of soil consistency. Performed for each soil borehole (see Appendix 2BB). The applicant defined “soil depth” as the depth where the SPT blow count was greater than 50 blows over 3 inches. Regarding the “depth to sound rock”, it was defined as the shallowest occurrence of at least 5 ft of contiguous slightly weathered to fresh rock.
- Rock Quality Designation (RQD) – indicator of rock soundness. Performed for each rock core run (see Table 2.5.4-203).
- R-scale Strength Values – semi quantitative indicator of rock strength. Performed for each rock core run (see Appendix 2BB).
- Point Load Strength Index – quantitative field measurement of rock hardness. Performed in four boreholes (BPA-47, BPA-48, BPA-49, AND BPA-50) (see Figures 2.5.4-207A, 2.5.4-207B, 2.5.4-207C, and 2.5.4-207D)
- Rock Pressuremeter Test (PMT) Modulus (E_{pmt}) – quantitative indicator of rock compressibility. Performed at four boreholes (BPA-3, BPA-23, BPA-41, and BPA-43) (see Table 2.5.4-205)
- Hydraulic Conductivity Tests – monitor seasonal fluctuations in groundwater elevations and evaluate hydraulic conductivity of soil and

rock. Twenty monitoring wells were made (see Figures 2.4.12-203 and 2.3.12-204; and tables 2.4.12-206 and 2.4.12-207)

  - Hydraulic Pressure (Packer) Tests – Performed at 2 different Boreholes (BPA-45 and BPA-46) near the center of HAR 2 and HAR 3, respectively. The tests were made at 10 ft intervals at each one of these locations for a total of 5 intervals each. According to the applicant, flow could only be measured in one interval at each hole; 0.05 ft/day at BPA-45 and 0.5 ft/day at BPA-46. The applicant indicated that the only significant groundwater flow is through secondary porosity from fractures within the rock mass.

Soil and Rock Engineering Properties from Laboratory Tests

FSAR subsection 2.5.4.2.3 describes the engineering properties obtained from laboratory samples. The applicant stated the following results of laboratory tests on rock and soil samples:

- Rock UCS and Index Test Results – On tests performed from samples obtained at the nuclear islands subgrade elevations, the applicant concluded that the underlying rock is of generally of high quality, due to the average UCS strength of greater than 6000 psi (see table 2.5.4-206).
- Petrographic Examination Results – This test was performed on 16 rock core specimens to characterize the gradation, mineralogy, and lithologic description of representative rock core sections (see Table 2.5.4-207). The applicant concluded that the total porosity and cementation of the samples were too low and that these results indicated that compaction was the main cause of lithification of the rock.
- Soil Laboratory Test Results –
  - Index tests - the applicant performed Index tests on residual soils samples collected above bedrock, and on soil-like samples obtained within sound rock (see Table 2.5.4-208).
  - Soil Strength tests – the applicant performed UC, UU, and CU triaxial tests on Pitcher Tube residual soil samples collected above bedrock and on soil like intervals within bedrock (see Table 2.5.4-209)
  - Consolidation tests – the applicant performed consolidation tests on five residual soil samples (see Table 2.5.4-210), but didn't perform this test on soil like samples due to the undisturbed nature of them.

Rock and Soil Properties for Use in Engineering Analyses

FSAR 2.5.4.2.4 presents a summary of the engineering properties used in the engineering analyses. The applicant derived static engineering properties of rock from laboratory tests performed (see Tables 2.5.4-205 and 2.5.4-206). These properties included: UCS, Elastic Modulus (secant and tangent), Poisson's ratio (secant and tangent), moisture content and bulk density. These were used by the applicant to carry out bearing capacity and settlement calculations. Rock mass strength properties were calculated using the Hoek-Brown criteria. This method takes into consideration the UCS and the discontinuities found in the rock.

The dynamic rock engineering properties were calculated based on the suspension logging surveys at HAR 2 and HAR3 (see Table 2.5.4-211). The applicant presented values of Shear Wave Velocity (V_s), compressional wave velocity (V_p), and Poisson's ratio. These properties were used on the calculation of elastic settlement and on site response analyses.

The rock's elastic modulus was calculated on the Vs profile at HAR 2 as 2.73 x 10^5 ksf and at HAR 3 as 2.84 x 10^5 ksf. According to the applicant, these values were depth weighted and a 0.5 reduction factor was applied in order to account for shear strain amplitude effects. Alternatively, the applicant used the results of different UCS tests performed to also calculate the Elastic Modulus. The average secant modulus at 50% failure strain was reported as 1.54 x 10^6 psi and 1.43 x 10^6 psi at HAR 2 and HAR 3 respectively. According to the applicant, these values are 20 to 30 percent lower than the previous ones due to sample recompression effects during the UCS testing. The average tangent modulus was also calculated by the applicant as 1.90 x 10^6 psi and 1.83 x 10^6 psi at HAR 2 and HAR 3 respectively. Results are very similar to the values based on the Vs profile.

The applicant concluded that the rock PMT moduli were significantly lower than the elastic modulus calculated from Vs and UCS data (see table 2.5.4-204). According to the applicant, at HAR 3 some of the tests performed yielded unreasonable values of Elastic Modulus due to: borehole widening (results not considered) and limited data obtained (results considered). The applicant stated that these values were considered only for upper bound estimates of elastic settlement.

The applicant calculated standard engineering properties of soil including the following: soil index tests (see table 2.5.4-208), soil strength tests (table 2.5.4-209), and soil consolidation properties (see table 2.5.4-210). The applicant restated that soils present under safety related structures are to be removed; therefore these properties were used to evaluate nuclear island construction slope stability.

2.5.4.2.3 <u>Foundation Interfaces</u>

FSAR section 2.5.4.3 describes the current surface conditions at the HAR sites. HAR 2 is located in an area that was graded previously with ground elevation ranging from 260 to 266 ft amsl. HAR 3 is located in an underdeveloped area north of HAR 2 with elevations ranging from 255 to 270 ft asml. The applicant stated that the nominal site grade for both sites will be at elevation 261 ft amsl and that the surrounding grade will be at a lower elevation in order to comply with site grading, drainage, and local site flooding requirements. Regarding the foundation specifics, the applicant described that the basemat will be founded at an elevation of 221.5 ft amsl alongside a mudmat and waterproofing geomembrane extending below to an elevation of 221.0 ft amsl. See figures 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B for planned excavation profiles.

2.5.4.2.4 Geophysical Surveys

Description of Geophysical Surveys

FSAR section 2.5.4.4 describes two general types of geophysical survey methods used: in-hole surveys and surface geophysical surveys (see Figures 2.5.4-203A and 2.5.4-203B). The applicant described the scope of the different methods as follows:

- Suspension Logging Surveys - Used to characterize shear and compressional wave velocities (Vs and Vp) profiles with depth. This method measures the time a compression or shear wave travels through a known distance between the seismic source and a pair of geophones. The probe was moved at intervals of 1.35 ft vertically in order to create a semi-continuous profile. Performed in boreholes BPA-5, BPA-25, BPA-39 and in BPA-47 trough BPA-50.
- Acoustic Televiewer Surveys – This device provides high-resolution, oriented images of the borehole walls by scanning an ultrasound beam. It was used to calculate the strike and dip of fractures and bedding planes. Performed in boreholes BPA-5, BPA-25, BPA-39 and in BPA-47 trough BPA-50.
- Downhole Surveys – Used as an alternate method to characterize the Vs profiles with depth. This method consists of an oriented geophone probe located at a known depth inside the hole while a shear wave is created at the surface. The probe was moved in intervals of 5 to 10 f5t. Performed at BPA-48 and BPA-49.
- Seismic Refraction Surveys – Performed to characterize the Vp profiles in soil and shallow bedrock. Data was recorded using a spread of 24 geophones spaced 10 m horizontally apart. The compression waves were generated by using an elastic weight drop hammer and 10 lb sledge hammer. 3200 linear feet of survey lines were performed among the two sites (see Fig. 2.5.4-203A and 2.5.4-203B)
- Magnetometer Surveys – Performed to identify diabase dikes and to trace previously identified dikes at the HAR site. A G-858 cesium vapor Magmapper system was used. Three lines each were performed at the HAR 2 and HAR3 sites (see Fig. 2.5.4-213)
- Spectral Analysis of Surface Waves Surveys (SASW) – Performed to characterize the Vs profiles within soil and shallow bedrock to a depth of 10 to 100 ft. This test is done in pairs at each location and at orthogonal

directions using multiple geophone spacings to measure surface wave velocity. SASW tests were performed at BPA-5, BPA-48, BPA-25, and BPA-49 (see Figs. 2.5.4-203A and 2.5.4-203B)
- Multi-Channel Analysis of Surface Wave Surveys (MASW) – Performed to characterize the spatial variability in Vs within soil and shallow bedrock. Due to inconsistent measurements, this method was not considered.

Geophysical Survey Investigation Results

FSAR subsection 2.5.4.4.2.1.1 discusses the results of Suspension Logging Surveys performed at HAR 2. The applicant concluded the following:

- Vs is greater than 4500 fps below the HAR 2 nuclear island elevation. There was an isolated layer (a few feet or so) in this profile that had a Vs value between 3660 to 4100 fps (see Table 2.5.4-202) which the applicant concluded to be associated with increased fracture density and isolated clay seams along bedding planes. The applicant observed that this feature is continuous along the dip under the nuclear island.
- Superimposition of Vs profiles of the three suspension surveys showed that Vs increases with depth at HAR 2 (see Fig. 2.5.4-214A).
- Vs data obtained from the three boreholes at HAR 2 are reasonably consistent between each other (see Fig. 2.5.4-214A).
- The Vs measured in the boreholes generally fall within 20% of the average of the three holes in dip-correlated strata.
- The dip calculated in this site has a magnitude of 8.9 degrees, directed 91 degrees clockwise from north.

Regarding, the Suspension Logging Surveys results at HAR 3, the applicant stated that:

- There is a more gradual transition from low Vs to high Vs due to the deeper extent of soil and weathered rock when compared to HAR 2 (see Fig. 2.5.4-209B, 2.5.4-209F, and 2.5.4-209G).
- Vs is typically greater than 3500 fps except in some intervals. The applicant associated these phenomena to isolated clay seams.
- Superimposition of Vs profiles indicate that Vs increases with depth at HAR 3. Any scatter observed, the applicant concluded it to be caused by dipping strata (see Fig2.5.4-215A)
- Vs is reasonably consistent in dip-correlated strata (see Fig. 2.5.4-215B).
- Vs measured in the boreholes generally fall within 20% of the average of the three holes in dip-correlated strata.
- The dip found in this site has a magnitude of 19.9 degrees, directed 109 degrees clockwise from north

Acoustic Televiewer Surveys were used to calculate the dip and orientation of bedding planes and fractures. The applicant stated that the “aggregate mean dip and direction of all such features grouped by HAR 2 and HAR 3 provide an estimate of true stratigraphic dip at each site" (see Fig. 2.5.4-216A and 2.5.4-216B). According to the applicant, dip magnitude and direction of such features were grouped into two sets at HAR 2 and HAR 3. The applicant concluded that Fracture set 1 at both sites were close to the dip magnitude and direction calculated; on the other hand, Fracture Set 2's orientation and dip magnitude was consistent with a primary joint set characterized in FSAR Subsection 2.5.1.2.4. (see Fig 2.5.4-216A and 2.5.4-216B). The applicant explained that the results of this method are considered secondary and that the results obtained from stratigraphic interpretation between boreholes and Vs profile matching were considered more accurate.

(NEED TO VERIFY STATEMENT WITH GEOLOGISTS)

Regarding the Downhole Velocity Surveys and the SASW surveys, the applicant concluded that the results of both these methods were consistent with the Suspension Logging Reports (see Figs. 2.5.4-209A, 2.5.4-209B, 2.5.4-209E, and 2.5.4-209F). The applicant reiterated that both these methods were used as a secondary measure to the Suspension Logging Reports due their lower resolutions and sensitivity to changes in Vs with depth.

Based on the results of the Seismic Refraction Surveys, the applicant identified 3 different layers of Vp as follows (see. Figs. 2.5.4-217a and 2.5.4-217B):

- Layer 1 - unconsolidated residual soils
- Layer 2 - altered or weathered rock
- Layer 3 - generally unweathered and sound bedrock

The applicant identified the top of layer 3 at HAR 2 to be between elevations 240 and 250 ft amsl and at HAR 3 to be between 230 and 240 ft. The applicant stated that the difference in elevations was due to the greater extent of layers 1 and 2 at HAR 3. Regarding the values of Vp obtained, the applicant observed that the Vp for Layer 2 at HAR 2 and HAR 3 ranged from 5203 and 6439 fps; and from 3174 and 3788 fps, respectively. On the other hand, the applicant stated that the results for Layer 3 at both sites were consistent around 10,223 to 12,007 fps. The applicant concluded that the top of Layer 3 indicated is considered to represent the shallowest encounter of fresh or slightly weathered rock at the profile locations.

Regarding the results of the Magnetometer Surveys (see Fig 2.5.4-213), the applicant indicated that there are no dikes present at the HAR 2 and HAR 3 sites. The HNP FSAR describes two dikes "East Dike 1" and "East Dike 2" that are present in the vicinity of the area, but the applicant confirms that both dikes pass by to the east and are not directly underneath the sites.

The applicant stated that the results of MASW surveys were not used in the analysis because the wavelengths detected were not accurate and the Vs results at some location were underestimated. The applicant stated that the procedure was implemented in accordance with industry practice.

FSAR subsection 2.5.4.4.2.8 summarizes all the results obtained from the geophysical surveys as follows:

- Suspension Logging Survey Data – the applicant concluded that the Vs profiles obtained within dip-related strata correlate very well.
- Downhole Velocity Survey Data – the applicant confirmed that these results serve as confirmation of the suspension logging survey results
- SASW Survey Data – the applicant used these results as yet another confirmation of suspension logging profiles at shallow depths of rock
- Magnetometer Survey Data – the applicant concluded that no dikes are present under the HAR structures.

2.5.4.2.5 Excavations and Backfill

Excavation Extents

FSAR section 2.5.4.5 describes the excavation and backfill plans for the nuclear islands, including excavation methods, backfill properties, among other things. The applicant stated that before excavation is commenced, the site will be graded to an elevation of 260 ft asml to accommodate site grading, drainage, and flooding requirements. During the excavation itself, the applicant described that from ground surface to the top of sound rock, sideslopes will be between 1.5H:1V to 2H:1V; while on rock, sideslopes will be between 0.25H:1V and 0.5H:1V. The applicant confirmed that the excavation depth to the top of sound bedrock may vary along the sides of the nuclear islands based on conditions encountered during construction (see Fig. 2.5.4-211A and 2.5.4-211B).

The applicant included in its excavation slope stability analysis, bedding planes and weak clay intervals found within the bedrock that may create potential sliding problems. Regarding this scenario, the applicant concluded that the excavation slopes suggested will remain stable during construction without external support and with an acceptable Factor of Safety (FS).

(NEED MORE INFO ON FS OBTAINED, ACCEPTABILITY CRITERIA, METHODS USED ON SLOPE STABILITY ANALYSIS)

Excavation Methods and Subgrade Improvement

FSAR subsection 2.5.4.5.2 describes excavation and subgrade improvement methods. The applicant established the following criteria that need to be satisfied at the nuclear island subgrade elevation:

- 90 percent recovery in rock borings at subgrade elevation
- 50 percent RQD at subgrade elevation
- Rock will be fresh to slightly weathered
- Subgrade will not have solution features, loose rock, nor open or soil-filled joints or fractures

The applicant concluded that any rock that does not comply with the aforementioned criteria will be removed or improved. The applicant stated that prior to the actual excavation; a detailed program will be created that includes the following:

- Specification of excavation methods.
- Quality control and quality assurance programs
- Dewatering methods and subgrade protection from degradation during dewatering and excavation.
- Specification of construction methods for dewatering and management of seepage and piping.
- Prior to subgrade improvement activities, a complete geologic mapping of the subgrade and excavation sidewalls will be performed.
- Rock that is excessively fractured or weathered will be over-excavated and filled with dental concrete.
- Soil-filled joints or fractures will be washed free of soil infilling to at least 5 ft below subgrade, and filled with dental concrete.
- The inspection and mapping of the excavation will be performed by qualified personnel.

Properties of Backfill Adjacent to Nuclear Islands

FSAR subsection 2.5.4.5.3 describes the necessary properties of backfill material that will be placed adjacent to the sidewalls of the nuclear islands (see Figs. 2.5.4-211A, 2.5.4-211B, 2.5.4-212A, and 2.5.4-212B). Depending on their location, the applicant described the materials and characteristics as follows (see Table 2.5.4-212):

- Concrete Fill – It will consist of structural concrete with no reinforcing.
- Compacted granular fill – Granular, well graded sand and gravel
- Compacted cohesive fill – Cohesive soils present at the HAR 2 and HAR 3 sites, with USCS classifications of lean clay (CL), silt (ML), clayey sand (SC) or silty sand (SM).

The applicant stated that nuclear islands were designed for safe performance independent of the type of backfill adjacent to it. Prior to construction, backfill sources will be verified, tested and all specifications will be developed.

2.5.4.2.6 <u>Groundwater Conditions</u>

Groundwater Elevation

FSAR subsection 2.4.12.5 describes in better detail groundwater elevation behavior at the HAR site (see Figs. 2.4.12-203 through 2.4.12-210). Nevertheless, the applicant stated that post-construction groundwater elevations will not exceed elevation 258 ft amsl. The applicant reiterated that the nuclear islands will be founded on sound rock and that foundation performance will not be affected unfavorably.

Construction Dewatering

FSAR subsection 2.5.4.6.2 describes how construction dewatering is required to maintain groundwater below elevation 220 ft amsl during excavation and construction. The applicant calculated the expected construction dewatering flow rates from hydraulic conductivity results, pre-construction groundwater elevations, and excavation limits.

The applicant developed orthogonal cross section models at HAR 2 and HAR 3 to calculate the groundwater inflow into the excavation cavity. To perform this calculation, the applicant assumed the steady-state groundwater elevation to be at 260 and 255 ft amsl at HAR 2 and HAR 3 respectively. Based on the results of hydraulic conductivity tests of all soils and rock, the applicant calculated the hydraulic conductivity to be 2.3 ft/day and 0.2 ft/day in all soils and rock, respectively (see Table 2.4.12-208).

Regarding the groundwater inflow rates, the applicant calculated, based on the aforementioned results, that 2.5×10^4 gpd at HAR 2 and 4.5×10^4 gpd at HAR 3 are expected. The applicant stated that this flow can be managed with pumps installed inside the excavation and any unexpected flow rate that occurs could be handled by more active methods during the construction process.

2.5.4.2.7 <u>Response of Soil and Rock to Dynamic Loading</u>

FSAR subsection 2.5.4.7 describes information regarding the response to dynamic loading of soil and rock. Surface faulting features are discussed in more detail in FSAR section 2.5.3. The applicant stated that no zones of Quaternary deformation were identified in the HAR site area; and that none of the mapped bedrock faults within a 25 mile radius or lineaments inside a 5 mile radius of the site are believed to be capable tectonic sources.

Regarding the site specific dynamic velocity profiles, the applicant presented those in FSAR section 2.5.4.4 (see PSER section 2.5.4.2.4). The applicant observed that the Vs profiles of rock below safety related structures and the Annex Buildings were greater than 3000 fps. Both types of structures are going to be founded on either sound rock or concrete fill, therefore the applicant decided not to perform resonant column/cyclic torsional shear tests for modulus degradation and damping due to the materials' high stiffness. Regarding the Ground Motion Response Spectra (GMRS), FSAR subsection 2.5.2.5.1.4 describes in better detail the modulus degradation and damping relationships developed for both HAR 2 and HAR 3.

2.5.4.2.8 Liquefaction Potential

FSAR subsection 2.5.4.8 discusses the liquefaction potential underneath the HAR sites. The applicant stated that the liquefaction potential underneath the HAR 2 and HAR 3 nuclear islands is nonexistent due to the fact that these two structures will be founded in sound rock. On the other hand, the Annex Buildings will be founded on top or near sound bedrock according to the applicant. The applicant stated that any overburden soil or weathered rock will be excavated to sound rock and replaced with concrete fill. The applicant concluded that this practice will diminish the liquefaction potential of the Annex Buildings. The applicant stated that prior to construction any left in place soil will be evaluated for soil liquefaction potential according to Regulatory Guide 1.198.

2.5.4.2.9 Earthquake Site Characteristics

FSAR subsection 2.5.4.9 summarizes the earthquake characteristics at HAR 2 and HAR 3. According to FSAR subsection 2.5.2.5, two subsurface profiles were adequate to represent the subsurface profile underneath HAR 3. The applicant noticed that there were differences in the Vs profiles at BPA-25, BPA-49, and BPA-50. These were associated with dipping strata and the extent of weathering related to ground elevations. On the other hand, at HAR 2 only one profile was considered representative of the subsurface profile. The applicant stated that the differences at HAR 2 are smaller than in HAR 3 due to the fact that the dip in bedding planes is shallower and the extent of weathering is more consistent. Both of these scenarios were taken into consideration to create the GMRS profile for both sites (see Figs. 2.5.2-306 and 2.5.2-307)

2.5.4.2.10 Static Stability

FSAR subsection 2.5.4.10 discusses the results obtained by the applicant for foundation bearing capacity, sliding, foundation settlement, and lateral pressures against below-grade walls.

Bearing Capacity

The applicant developed the following methodology to calculate the ultimate bearing capacity of the rock mass at HAR 2 and HAR 3:

- Classic bearing capacity equations adopted for rock foundations – This was used as the primary method.

- Two empirical methods – Used as a secondary check.
- Finite element modeling – Used as an alternate method.

The general bearing capacity equation takes in consideration many factors, mainly: cohesion, effective surcharge pressure, foundation width, and the effective unit weight of the foundation media. There are other empirical factors included that are dependent of the friction angle (see Ref. 2.5.4-231). The applicant also considered local shear failure conditions in the analysis. According to the applicant, this last condition resulted in a conservative version of the general bearing capacity equation due to the fact that a term related to effective surcharge pressure is not included. Regarding the rock mass properties, the applicant determined them using the Hoek-Brown criterion alongside results of the UCS tests and the condition of discontinuities.

The applicant took the following measures into consideration in the analysis:

- For static ultimate bearing capacity, the applicant used a reduced depth weighted foundation width (124 ft from 160 ft) to account for the variable east-west width of the nuclear island.
- For dynamic ultimate bearing capacity, the applicant considered both horizontal and vertical components of seismic load that would occur. The horizontal load consisted of an asymmetrical load that decreased rapidly from one edge of the basemat to the other. The applicant made the following assumptions to accommodate this feature:
  - The load was converted to an equivalent vertical load and moment.
  - The eccentricity (e) was calculated and an effective width of B'= B-2e was used.
  - The same equation used for static bearing capacity was used with the calculated effective width.

After calculating both static and dynamic ultimate bearing capacity, the applicant proceeded to calculate the allowable bearing capacity for each case by dividing each result by Factor of Safety (FS).

Regarding the empirical methods, the applicant used the following:

- American Association of State Highway and Transportation Officials (AASHTO) method (see Ref. 2.5.4-233) – This method is based on a correlation between the intact rock's UCS value and the RQD. The applicant approached this method in a conservative manner by assuming jointed or broken rock in the calculation.
- Hoek-Brown strength criterion (see Ref. 2.5.4-232) – This method is based on the UCS of intact rock and the Hoek-Brown strength criterion constants, which are based on type and quality of rock.

As mentioned before, the applicant also performed a 2D finite element analysis as an alternate method to the aforementioned ones. The applicant stated that the other purpose for this method was to conservatively evaluate the effects of thin soil intervals within rock found in BPA-6 at HAR 2. The applicant generated two-dimensional models for both sites in the north-south direction. In the models developed, the applicant considered the following: spatial variations of subsurface stratigraphy, material properties, pore pressure distribution, excavation extents, and backfill material properties. The applicant used an uniform bearing pressure of 8600 psf for its static model and a variable pressure distribution for its dynamic model (see Fig. 2.4-2 of Ref. 2.5.4-234). The following cases were evaluated:

- Case 1: HAR 2 Lower-Bound Properties
- Case 2: HAR 3 Lower-Bound Properties
- Case 3: Potential Influence of Borehole BPA-6 Conditions – A 5 feet thick soil seam was included where it was encountered at BPA-6. The applicant assumed the dip to the south and considered friction angles of 33 and 10 degrees.

Regarding the selection of rock strength parameters for the bearing capacity analyses, the applicant performed the following steps:

- The mean and standard deviation of all rock UCS tests results for samples collected within one foundation width B of the basemat elevation were calculated(see Table 2.5.4-206).
- Mean and lower bound values for all GSI values were developed (see Table 2.5.4-213)
- “Mean” Hoek-Brown strength parameters based on the average UCS and GSI values for rock below the subgrade elevation in both sites were estimated.
- “Lower-bound” Hoek-Brown strength parameters using the lower bound values of rock UCS and GSI below the subgrade elevation in both sites were calculated. Lower bound values were selected at one standard deviation below the mean.
- “Worst-Case” Hoek-Brown strength parameters based on the lowest UCS value obtained below subgrade elevation and the lower bound GSI were considered.

Based on the results of the previous analyses (see Table 2.5.4-213), the applicant observed that UCS lower-bound values at one foundation width below HAR 2 are higher than at HAR 3. The applicant also indicated that there is more strength variation with depth within one foundation width at HAR 3 than at HAR 2. The applicant concluded that the reason for this difference lies in the fact that there is a larger depth of soil and weathered rock at HAR 3. For bearing capacity analysis purposes, the applicant divided the HAR 3 profile into a shallow layer and a deeper layer. The shallow layer ranged between elevations 170 and 220 ft amsl, while the deeper layer ranged from elevation 170 ft amsl and under. On the other hand, at HAR 2, the applicant indicated that the GSI and UCS results were more consistent with depth; therefore only one rock layer was modeled.

Regarding the results obtained for ultimate bearing capacities using the general bearing capacity method and the two alternate empirical methods, the applicant developed Table 2.5.4-214. The applicant considered a minimum FS of 3.0 and 2.0 for static and dynamic loads, respectively. The applicant concluded that these Factors of Safety were satisfied on both sites and for each case. Concerning the allowable bearing pressures, the applicant calculated them based only on the lower-bound strength parameters and the aforementioned Factors of Safety, as follows:

- HAR 2 – Static Loading: 54 ksf
- HAR 2 – Dynamic Loading: 68 ksf
- HAR 3 – Static Loading: 29 ksf
- HAR 3 – Dynamic Loading: 37 ksf

Based on these results, the applicant concluded the following:

- The allowable bearing capacity values are conservative because they are based on lower bound strength parameters.
- 84% of the rock has higher strength, therefore there's a high confidence that UCS and GSI values are greater than the lower bound values.
- These values are highly conservative because they only represent thin zones, and bearing capacity values will probably resemble more the average within the zone of influence of the foundation which is considerably greater.

Regarding the results of the 2D finite element analysis, the applicant stated that the Factors of Safety obtained were greater than the other methods. The applicant indicated that the reason for the discrepancy is mainly because the shear strength of the soil and rock above the basemat was not included in the general bearing capacity method, while in the finite element model it was considered. Concerning the conditions present at BPA-6, the applicant indicated that the models ran for this case resulted in an FS greater than 4 in both static and dynamic cases, therefore foundation performance is not expected to degrade. Overall, the applicant concluded that FS requirements were fulfilled in all cases considered and that the results of the finite element analysis prove that the general bearing capacity and the empirical methods are conservative.

Resistance to Sliding

FSAR subsection 2.5.4.10.2 briefly summarizes the resistance to sliding of both HAR 2 and HAR 3 nuclear islands. The applicant restated that both nuclear islands will be founded on sounded rock and that any loose material found at the subgrade elevation will be removed (see FSAR subsection 2.5.4.5.2). To avoid sliding during the SSE event, the applicant indicated that a concrete fill will be placed on top of the rock subgrade in order to interlock with it and minimize the movement. The applicant indicated that the space between the nuclear island sidewalls and the excavation will be filled with either concrete fill, compacted granular fill or compacted cohesive fill (see FSAR subsection 2.5.4.5.3). According to the applicant, any passive resistance from this backfill will not be necessary to prevent sliding.

Settlement

FSAR subsection 2.5.4.10.3 discusses the expected settlement of the nuclear islands and the methods used for such calculations. The applicant estimated that both total (elastic and recompression) and differential settlements will be significantly small for both HAR 2 and HAR 3.

To calculate the elastic settlement, the applicant used two separate methods:

- Small strain elastic and constrained moduli values of the rock were calculated based on Vs, Vp, and Poisson's ratio results from suspension logging surveys. The applicant stated that the small strain constrained modulus was reduced by 50% in order to conservatively model larger strains.
- The constrained modulus profile was estimated based on the results of the PMT.

The applicant used simple elastic theory to calculate settlement for both nuclear islands. The simple elastic theory is a summing function that basically divides the zone of influence underneath the foundation into numerous layers with their own properties. Settlement is inversely proportional to the constrained modulus of the layer and directly proportional to the change in vertical stress and height of the layer in question. The applicant concluded that by using the first method, the elastic settlement will be approximately 0.03 to 0.04 in. at both sites. On the other hand, the second method resulted in an approximate settlement of 0.1 and 0.2 in at HAR 2 and HAR 3, respectively. The applicant stated that the first results are considered the best estimates; while the second ones are the upper bound estimates (see Table 2.5.4-215).

Recompression settlement was only considered by the applicant for the thin clay layers encountered within sound rock. The applicant stated that due to the thinness of the clay layers, a sample could not be extracted for testing purposes. Furthermore, the applicant concluded that the magnitude of settlements related to recompression is considered to be small, due to the following reasons:

- Clay seams found were overconsolidated (see Table 2.5.208). This is evidenced by the fact that the water content found in all samples is approximately half the plastic limit.
- Consolidation tests on residual soils samples with similar water content and plastic limit showed that they were overconsolidated and with low compressibility.
- Some of the clay seams found were likely made by disturbances in the coring process. HQ sized rock cores presented less incidence of clay seams when compared to the NQ sized cores.
- Not fully continuous clay seams will produce smaller settlements than laterally continuous seams because some portion of the vertical stress will redistribute around the seams themselves.

Based on the aforementioned reasons, clay thicknesses, and representative consolidation properties, the applicant estimated conservative recompression settlements for representative boreholes at both sites (see table 2.5.4-215). The applicant stated that settlement calculations were based on a conservatively high net foundation pressure of 8600 psf. The applicant explained that this pressure assumes that the clay seams will fully reduce to the pressure and that the underlying clay seams will fully rebound after excavation prior to the loading; which in reality they will not fully relieve.

Total settlement is calculated by adding the results of elastic and recompression settlement (see Table 2.5.4-215). The applicant concluded the following:

- Total settlements will range from 0.03 to 0.5 in. at HAR 2 and from 0.1 to 0.5 in. at HAR 3.
- If the thick clay seams found on borehole BPA-6 (see FSAR subsection 2.5.4.1.3) were to be present underneath HAR 2, a recompression settlement of 1.2 in. would be expected. Due to the fact that such seams were not found underneath the nuclear island, the total settlements calculated are representative.
- Total settlements calculated are within the range of acceptable settlements for the AP1000 (3 in.).

The applicant also made differential settlement calculations (see Table 2.5.4-216). Differential settlement is defined as the difference in total settlement between a pair of boreholes divided by the distance between them. This calculation can also be seen in terms of angular distortion, which basically represents the slope of differential settlement between boreholes. According to AP1000 specifications an angular distortion of less than 0.00083 is acceptable. The applicant concluded that all expected differential settlements under the nuclear islands will be within the acceptable range (see Table 2.5.4-216).

Regarding differential settlement between the nuclear islands and the adjacent structures, the applicant stated that it is not expected to exceed 0.5 in. The applicant stated that a detailed analysis will be performed after foundation bearing loads for the adjacent structures are finalized.

During the excavation, the applicant stated that the nuclear island subgrade is expected to rebound by approximately half of the calculated total settlement. Rebound is basically caused by the relief of vertical effective stress by the removal of overlying soil.

Overall, the applicant's conclusions about foundation settlement were based on the following AP1000 criteria:

- Total settlement under nuclear island: up to 3 inches.
- Differential settlement across nuclear island: up to 0.5 inches per 50 ft. (1/1200 slope)
- Differential settlement between nuclear island and adjacent structures: up to 3 inches.

To verify the expected settlements, the applicant stated that a monitoring program will be developed as follows:

- After the excavation, settlement benchmarks will be installed within the subgrade mudmat at the four corners of each island and at the northernmost point of each containment building. These will be monitored during the construction of the basemat and prior to the backfill placement.
- After the construction of the nuclear island, more benchmarks will be placed directly over the locations of the original ones. They will be connected to the nuclear island walls at approximately 1 meter above site grade. They will be monitored during backfilling operation and during the construction of the nuclear island structures.
- Monitoring will be continued until 90% of the expected settlement has occurred.

Lateral Earth Pressures

FSAR subsection 2.5.4.10.4 describes the lateral earth pressures expected on the nuclear island sidewalls due to placement and compaction of backfill. The applicant stated that granular and cohesive backfills will be compacted as described in FSAR subsection 2.5.4.5.3. The applicant considered the following effective stress parameters in the calculation of lateral pressures:

- Compacted granular backfill – Friction angle of 35 degrees and cohesion of 0 psf.
- Compacted cohesive backfill – Friction angle of 20 degrees and cohesion of 400 psf.

The applicant stated that at-rest earth pressures will act on the nuclear island sidewalls based partly on the assumption that backfill will be properly compacted. According to the applicant, passive pressure will not fully develop on the sidewalls, because the wall would have to move a considerable distance into the backfill. The applicant affirms that this movement is not expected due to the stiffness of the nuclear structures, and furthermore, because stability against sliding without contribution from passive pressures was considered in the design.

The at-rest pressure is a function of the friction angle of the backfill, overconsolidation ratio (OCR), effective overburden pressure, groundwater pressure, and surface surcharge pressures. The applicant considered the maximum estimated groundwater elevation of 258 ft amsl in the groundwater pressure calculation. Regarding the surface surcharge pressure, the applicant stated that adjacent structures will likely be founded on sound rock and therefore is not expected for said loads to transfer into the soil backfill. The applicant concluded that these surcharge loads were not considered in the at-rest pressure calculation. These calculations are present in Table 2.5.4-217 as a function of sidewall elevation. The maximum lateral earth pressure calculated by the applicant was 3.6 and 4.2 ksf for compacted granular and cohesive fill, respectively. The applicant stated that the extent of soil backfill will vary by location. The applicant concluded that any concrete backfill that may be placed will not contribute to lateral earth pressures.

(WHAT'S THE CONCLUSION? IS IT ACCEPTABLE? WHAT'S THE ALLOWABLE LATERAL EARTH PRESSURE?)

2.5.4.2.11 Design Criteria

FSAR subsection 2.5.4.11 summarizes all of the design criteria used by the applicant in the stability evaluations for safety-related structures, etc, as follows:

- DCD site geotechnical parameter criteria with corresponding site characteristics (Table 2.0-201)
- Criteria for selection of borehole locations and depths (FSAR subsection 2.5.4.2.1.1.1 and 2.5.4.2.1.1.2
- Criteria for selection of rock core and soil samples (FSAR subsection 2.5.4.2.1.6.2 and 2.5.4.2.1.6.3)
- Criteria for selection of rock and soil properties (FSAR subsection 2.5.4.2.4)
- Criteria for selection of geophysical survey results (FSAR subsection 2.5.4.4.2.8)
- Criteria for evaluation of nuclear island subgrade conditions, and identification of the need for subgrade improvement (FSAR subsection 2.5.4.5.2)
- Criteria for groundwater elevations and construction dewatering methods (FSAR subsection 2.5.4.6.1 and 2.5.4.6.2)

- Criteria for determination of nuclear island allowable bearing pressures and selection on static and dynamic factors of safety (FSAR subsection 2.5.4.10.1)
- Criteria for determination of nuclear island settlement, tolerable settlement limits, and subgrade rebound (FSAR subsection 2.5.4.10.3 and 2.5.4.10.3.6)
- Criteria for estimation of nuclear island sidewall lateral pressures (FSAR subsection 2.5.4.10.4)

2.5.4.2.12 <u>Techniques to Improve Subsurface Conditions</u>

FSAR subsection 2.5.4.12 briefly summarizes techniques to improve subsurface conditions discussed before. The applicant reiterated that the nuclear islands will be founded on sound rock and that any rock that does not satisfy the criteria at the nuclear subgrade elevation will be removed (see FSAR subsection 2.5.4.5.2). The applicant stated that a detailed excavation, subgrade improvement, and verification program will be developed prior to construction.

Possible RAIs:

1. Paragraph 2.5.4.1.7, Estimates of Preconsolidation Pressure states that high preconsolidation pressures were obtained for residual soils in-situ. Please indicate what were the preconsolidation pressures obtained.

2. Paragraph 2.5.4.5.1 Excavation Events states “Stability analyses were performed using the software package SLIDE Version 5.026.” It also states “Results confirm that the excavation slopes shown will remain stable during construction without external support”. What slope stability methods (Ordinary Method of Slices, Simplified Bishop, etc) were used in the SLIDE analysis? What were the FS obtained?

3. Paragraph 2.5.4.10.4 Lateral Earth Pressures describes the lateral pressures exerted on the nuclear sidewalls. What’s the allowable lateral earth pressure on the walls?