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Shearon Harris Nuclear Power Plant Units 2 and 3
Docket Nos. 52-022 and 52-023
Supplemental Information Regarding Backfill Under Non-Seismic Structures

Gentlemen:

In conference calls held between Progress Energy - Carolinas (PEC) and the NRC on March 18, 2008 and April 1, 2008, discussions were held regarding the potential for liquefaction of the planned backfill materials under non-seismic structures and adjacent to the proposed Shearon Harris Nuclear Power Plant Units 2 and 3 nuclear islands. As a follow up to the discussions, PEC is providing the enclosed Technical Memorandum which provides supplemental information regarding the use of compacted granular fill, in-situ native soils and compacted cohesive backfill. This letter is being re-submitted since our submittal dated May 12, 2008 inadvertently omitted three pages of the enclosure.

Two empirical methods were used to address compacted granular backfill. As-compacted shear wave velocity of 500 feet per second provides an adequate factor of safety against liquefaction throughout the backfill depth. This conclusion is confirmed by relationships between relative compaction, relative density and standard penetration test (SPT) blowcounts. Spectral Analysis Surface Wave (SASW) testing will be used to validate as-compacted backfill shear wave velocity.

Geologic and soil texture-based screening of native soil reveal a general indication that liquefaction potential is low. This was confirmed with screening based on laboratory index properties. Native granular soils were evaluated based on SPT blowcounts. Because some SPT blowcounts indicate low or intermediate factors of safety, this material will not be used as confinement for granular fill.

Compacted cohesive fill was evaluated with tests performed for Shearon Harris Nuclear Power Plant Unit 1. The expected as-placed moisture content of cohesive backfill was compared to the average liquid limit of expected backfill material. This comparison revealed no susceptibility to liquefaction. The cohesive backfill specification will require as-compacted material to have a plasticity index of 7 or more and as-compacted moisture content less than 0.8 times the liquid limit.

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Pending NRC's concurrence with this approach, the Shearon Harris Nuclear Power Plant Units 2 and 3 FSAR subsection 2.5.4.5, Excavation and Backfill, and 2.5.4.8, Liquefaction, will be updated in a future revision to incorporate these results.

If you have any questions or need additional information, please contact Bob Kitchen at (919) 546-6992.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 29th day of May, 2008.

Sincerely,

James Scarola

Senior Vice President and
Chief Nuclear Officer

Enclosure

cc: (w/o encl) U.S. NRC Director, Office of New Reactors/NRLPO
U.S. NRC Office of Nuclear Reactor Regulation/NRLPO
U.S. NRC Region II, Regional Administrator
U.S. NRC Resident Inspector, SHNPP Unit 1
Mr. Manny Comar, Project Manager, Division of New Reactor Licensing

ATTACHMENT A

Tech Memo Approval Form

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Revision: 0

Project: PEC COLA

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Tech Memo Title:
Supplemental Liquefaction Evaluation - HAR Site Backfill and Native Soil

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Document Review and Approval

Originator:	Matt Gavin / Geotechnical Task Lead Name/Position	4/28/2008 Date
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TECHNICAL MEMORANDUM CH2MHILL

Supplemental Liquefaction Evaluation - HAR Site Backfill and Native Soil

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DATE: April 28, 2008

PROJECT NUMBER: 338884.NC

Subsection 2.5.4.8 of the Harris Advanced Reactor (HAR) Site Final Safety Analysis Report (FSAR) describes seismic Category (SC) 1 and 2 structures at both HAR 2 and HAR 3 as founded on sound rock or concrete fill over rock, and therefore liquefaction will not affect the foundations for these structures. During subsequent discussion with Nuclear Regulatory Commission (NRC) staff in March and April 2008, NRC identified a need for supplemental information to demonstrate that planned backfill under non-seismic structures adjacent to the HAR 2 and 3 nuclear islands, specifically the Turbine Buildings, will also not be susceptible to liquefaction.

Supplemental liquefaction evaluations of HAR structure backfill and native soils have been performed in accordance with guidance in Regulatory Guide (RG) 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites." Widely accepted soil texture-based screening criteria and empirical evaluation methods have been used as cited herein.

This memorandum presents the following supplemental information demonstrating that HAR site backfill and native soils will not be susceptible to liquefaction:

- Section 1.0 presents an overview of site-specific design ground motion parameters used in the liquefaction analyses. This includes general descriptions of the equations used to calculate the factor of safety (FS) against liquefaction, which are common to each of the empirical evaluations presented in this memorandum, including calculation of the cyclic stress ratio (CSR). Basis for selection of the site-specific peak ground acceleration (PGA) and design earthquake magnitude (M) are also presented.
- Section 2.0 presents the evaluation of liquefaction resistance of compacted granular backfill. Compacted granular fill may be used under non-seismic structures adjacent to the HAR nuclear islands (including the Turbine Buildings) and adjacent to the nuclear-island sidewalls. The source of granular backfill has not yet been selected, although minimum granular backfill engineering parameters are specified in HAR FSAR Table 2.5.4-212. The liquefaction evaluation was based on correlations between liquefaction resistance and as-compacted shear-wave velocity (V_s) presented in Andrus et al., 2004, which is the current update to the V_s-based procedure presented in Youd et

al., 2001. Section 2.0 also summarizes a secondary evaluation of the expected as compacted relative density (D_r) of the granular backfill, the corresponding standard penetration test (SPT) blowcount (N) values, and associated relationships with liquefaction resistance as presented in Youd et al., 2001.

- Section 3.0 presents an evaluation of liquefaction resistance of native in-situ soils at the HAR site. Native soils will be removed under HAR structures, but will be left in-place adjacent to backfill outside of the structure extents. Soil texture-based screening criteria presented in Seed et al., 2003 and by others were used to evaluate liquefaction susceptibility of native fine-grained soils. In addition, an empirical evaluation was used to determine the FS of potentially liquefiable native soils against liquefaction based on the SPT N values.
- Section 4.0 presents liquefaction resistance of compacted cohesive backfill. Compacted cohesive backfill is not planned under AP1000 structures, but may be placed adjacent to granular backfill outside the footprint of structures or adjacent to the nuclear-island sidewalls. Soil texture-based screening criteria presented in Seed et al., 2003 and by others were used to evaluate liquefaction susceptibility of cohesive backfill.
- Section 5.0 presents recommendations for construction based on the results of the liquefaction evaluations. Recommendations include proposed refinements of the granular backfill shear-wave velocity criteria presented in HAR FSAR Table 2.5.4-212 and proposed confirmatory tests to demonstrate acceptance of the compacted granular backfill during construction.

HAR FSAR Section 2.5.4 discussions, tables, and figures are referenced throughout this memorandum. Pending NRC's concurrence with the approach described in this technical memorandum, HAR FSAR Subsections 2.5.4.5 (Excavations and Backfill) and 2.5.4.8 (Liquefaction) will be updated in a future revision to incorporate the results summarized in this memorandum.

1.0 Site-Specific Ground Motion Parameters, CSR, and FS Criteria

In each of the empirical liquefaction evaluation methods presented in this memorandum (Sections 2.0 and 3.0), the FS against liquefaction is calculated based on the ratio of the cyclic resistance ratio (CRR, a measure of soil strength against cyclic loading), to the CSR (a measure of stress induced due to cyclic loading). Equations used to calculate FS and CSR are the same for each of the empirical methods, whereas equations used to calculate CRR vary by method (see Sections 2.0 and 3.0 for discussion of CRR calculation for each method). The equations discussed below are presented in detail in Youd et al., 2001.

In the empirical evaluations of liquefaction resistance, the FS is calculated at each depth of interest using the following relationships:

$FS = (CCRR/CSR) * (MSF)$,

$CCRR = CRR * K_\sigma * K_\alpha$, and

$MSF = 10^{2.24} / (M)^{2.56}$

Where CCRR is the corrected cyclic resistance ratio, K_σ and K_α are correction factors for overburden and sloping ground, MSF is the magnitude scaling factor, and M is the design earthquake magnitude for the site.

CSR is calculated at each depth of interest as follows:

$$CSR = 0.65 * a_{max} * (\sigma_v / \sigma_v') * r_d$$

$$a_{max} = PGA (F_{pga})$$

Where a_{max} is the peak horizontal ground acceleration at the ground surface (site grade), r_d is the shear stress-reduction coefficient (a function of depth), σ_v is the total overburden pressure, σ_v' is the design effective stress, PGA is the site-specific PGA on rock, and F_{pga} is a site factor (assigned value of 1.2) to convert the PGA at top of rock to a ground surface motion (IBC, 2006).

Selection of the site-specific PGA, M, and groundwater depth (d) used as input to the empirical evaluations is based on the following:

- **Design earthquake magnitude (M) = 7.1.** This is selected as the mean moment magnitude of the Charleston source zone, as summarized in HAR FSAR Subsection 2.5.2. Based on the deaggregation of the 5 and 10 hertz motions from the site-specific probabilistic hazard analysis (PSHA), there is appreciable contribution to the high-frequency ground motion hazard from nearby smaller magnitude earthquakes (M=5 to 6). Therefore, use of the Charleston source magnitude as contributing to the entire hazard used in the liquefaction analysis is conservative.

- **Peak ground acceleration (PGA) = 0.173g.** This is based on the ground motion response spectrum (GMRS) for HAR 3 (PGA = 0.137g) as presented in HAR FSAR Subsection 2.5.2, divided by the factor 0.792 to remove the contribution of cumulative average velocity adjustments (a conservative value based on 10^{-4} mean annual frequency). The GMRS is defined at the top of the shallowest competent layer, and incorporates some degree of site amplification relative to deeper hard rock motions. Therefore, use of the GMRS in this manner results in a conservatively high estimate of PGA.

- **Groundwater depth (d) = Zero feet below ground surface (bgs).** This is based on the conservative assumption that groundwater elevation is present at site grade. Actual groundwater elevations will occur in the upper 5 to 10 feet bgs.

These site-specific parameters and the equations to calculate FS and CSR presented above are common to each of the empirical methods presented in Sections 2.0 and 3.0. The resulting FS values are compared to the liquefaction-susceptibility criteria presented in RG 1.198, as summarized below:

- **Soil elements with low FS (FS <= 1.1)** would achieve conditions wherein soil liquefaction should be considered to have triggered. Conservative undrained residual strengths should be assigned.

- **Soil elements with high FS (FS >= 1.4)** would suffer relatively minor cyclic pore pressure generation. Some large fraction of the static (drained) strength should be assigned.

- **Soil elements with intermediate FS (FS between 1.1 and 1.4)** should be assigned intermediate strength values. In contractive soils, the possibility of progressive failure or deformation should be considered and undrained residual strengths assumed.

Based on these criteria, FS greater than 1.4 was considered indicative of soils that are not susceptible to liquefaction or significant strength reduction.

2.0 Liquefaction Resistance of Compacted Granular Backfill

Compacted granular fill may be placed under non-seismic structures adjacent to the HAR nuclear islands (including the Turbine Buildings) and adjacent to the nuclear-island sidewalls. The source of granular backfill has not yet been selected. As summarized in HAR FSAR Table 2.5.4-212, granular backfill will consist of well-graded sand or gravel placed to at least 95 percent relative compaction (modified Proctor method). Granular backfill will have a minimum drained friction angle of 35 degrees. A wide range of allowable in-situ V_s is currently specified in the HAR FSAR (350 to 1250 fps), although it is expected that granular backfill V_s in excess of 500 fps can be readily achieved for the specified material.

2.1 Summary of Evaluation Methods

Following is a summary of the two empirical liquefaction evaluation methods applied to granular backfill.

2.1.1 Primary Empirical Method – Based on Shear-Wave Velocity

For this V_s-based evaluation, CRR was calculated at each depth of interest based on the overburden corrected shear-wave velocity (V_{s1}) using the following relationships presented in Andrus et al., 2004. This method is the current update to the V_s-based procedure presented in Youd et al., 2001:

$$V_{S1} = V_S C_{VS}$$

$$C_{VS} = \left(\frac{Pa}{\sigma_v'}\right)^{0.25}\left(\frac{0.5}{K_0'}\right)^{0.125} \quad \text{(Maximum } C_{VS} \text{ of 1.4)}$$

$$CRR = \left[0.022\left(\frac{K_{a1}V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{S1}^* - (K_{a1}V_{S1})} - \frac{1}{V_{S1}^*}\right)\right]K_{a2}$$

Where C_{VS} is an overburden correction factor, P_a is the reference pressure (one atmosphere or 2,116 psf), σ_v' is the design effective stress, K_o' is the coefficient of effective earth pressure at rest, K_{a1} and K_{a2} are factors to correct for aging, and V_{S1}^* is the limiting upper value of V_{S1} for liquefaction occurrence.

Once the CRR at the depth of interest was calculated based on V_s data, the MSF, CCRR, CSR, and FS against liquefaction were calculated as presented in Section 1.0 above. Values of CRR, CCRR, CSR, and FS were calculated for various fill depths ranging from zero (site grade) to 40 feet bgs (NI foundation level).

The following site-specific parameters were used to calculate CRR for this calculation:

- **Granular backfill unit weight = 140 pounds per cubic foot (pcf) (saturated).** These are considered typical value for well graded granular soil at 95 percent relative compaction (Kulhawy and Mayne, 1990; Table 2-8). The saturated unit weight, along with a design groundwater depth of zero, was used to calculate total and effective stresses (σ_v and σ_v') at depth intervals in the fill.
- **Coefficient of earth pressure at rest, K_o'.** Conservatively assigned as 3.0 for compacted fill, which bounds nearly all typical values reported for granular fill against structures (Terzaghi, Peck, and Mesri, 1996; Table 44.1). This is conservative in that high K_o' results in a lower C_{VS}, and hence lower CRR and FS for a given value of V_S.
- **Age correction factors K_{a1} and K_{a2}.** These correction factors are each conservatively assigned as 1.0 because the compacted backfill is considered equivalent to a recent deposit (no age correction is appropriate).
- **Limiting upper value V_{S1}^*:** This is conservatively assumed to be 215 meters per second (m/sec) (705 feet per second [ft/sec]), corresponding to sand with less than 5 percent fines (Andrus et al., 2004). A velocity of 705 ft/sec is conservative in that this value is the highest of the three soil types considered in Andrus et al., 2004, and higher V_{S1}^* results in lower calculated CRR and FS for a given value of V_S.

These parameters were used to calculate the required value of in-situ V_S that would result in FS greater than 1.4 at each depth of interest.

2.1.2 Secondary Empirical Method – Based on Relationship between Relative Density and SPT N Values

A second empirical evaluation of the FS against liquefaction for granular backfill was performed as confirmation of the primary method summarized in Section 2.1.1 above. This secondary method is based on Youd et al., 2001 in which liquefaction resistance (CRR) at the depth of interest is related to the SPT N value. For this application, an equivalent SPT N of the compacted granular fill was first calculated based on relationships between relative compaction (RC), relative density (D_r), and SPT N, as summarized below. These equivalent SPT N values were then used as input to calculate CRR per Youd et al., 2001, as described below.

Relative density (D_r) of the granular backfill was calculated based on the following definitions:

$$D_r = \left[\frac{\gamma_d - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}}\right]\left[\frac{\gamma_{d(\max)}}{\gamma_d}\right], \text{ and}$$

$$\gamma_d = \gamma_{d(\max)} RC$$

Where γ_d is the as-compacted dry density, $\gamma_{d(max)}$ is the maximum dry density, $\gamma_{d(min)}$ is the minimum dry density, and RC is the relative compaction of the backfill.

Typical values of $\gamma_{d(max)}$ and $\gamma_{d(min)}$ for clean, well-graded sand are 138 and 85 pcf, respectively (Kulhawy and Mayne, 1990, Table 2-8). Relative compaction of 95 percent corresponds to a γ_d of 131 pcf. Based on the above relationships, this corresponds to D_r of 91.4 percent (very dense).

Typical relationships between D_r and SPT N have been developed (Kulhawy and Mayne, 1990). Very dense sand (with D_r between 85 and 100 percent) commonly has SPT N values greater than 50. SPT N has also been shown to vary with D_r and with overburden pressure, σ_v'. For D_r of 90 percent, the estimated SPT N varies with σ_v' as follows (Kulhawy and Mayne, 1990):

σ_v'/P_a	σ_v' (psf)	Equivalent SPT N
0	0	16
0.36	762	28
1.36	2,878	42
2.72	5,761	67

In this application, the equivalent SPT N was assigned based on the calculated σ_v' at each depth of interest and linear interpolation of the values in the above table.

CRR was then calculated by assigning the calculated equivalent SPT N as the field-measured SPT N and using the following relationships (Youd et al., 2001):

$$CRR = \frac{1}{34-(N_1)_{60CS}} + \frac{(N_1)_{60CS}}{135} + \frac{50}{[10*(N_1)_{60CS}+45]^2} - \frac{1}{200}$$

$$(N_1)_{60CS} = \alpha + \beta\,(N\,C_N\,C_E\,C_B\,C_R\,C_S\,C_{CS})$$

Where $(N_1)_{60CS}$ is the corrected, clean sand SPT N at the depth of interest, and K_σ and K_α are overburden and sloping ground correction factors, respectively. $(N_1)_{60CS}$ is calculated from the field-measured SPT N based on correction factors for overburden (C_N), hammer energy ratio (C_E), borehole diameter (C_B), rod length (C_R), and sample liner (C_S). The α and β factors were used to correct for fines content, and are conservatively assigned as 0 and 1.0, respectively, for clean sands (less than 5 percent fines) following guidance given in Youd et al., 2001.

Once the CRR at each depth of interest was calculated based on the equivalent SPT N value, the MSF, CCRR, CSR, and FS against liquefaction were calculated as presented in Section 1.0. Values of CRR, CCRR, CSR, and FS were calculated for various fill depths ranging from zero (site grade) to 40 feet bgs (NI foundation level).

2.2 Summary of Results

The V_s-based empirical evaluation presented in Section 2.1.1 above indicates that an as-compacted V_s of 500 fps or greater will provide FS against liquefaction greater than 1.4 throughout the backfill depth. The equivalent SPT evaluation described in Section 2.1.2 above also indicates that granular backfill compacted to 95 percent relative compaction will give SPT N values greater than 30, which results in FS against liquefaction greater than 1.4 throughout the backfill depth.

These results provide confirmation that the compacted granular backfill will not be subject to liquefaction if the compacted backfill is placed at 95 percent relative compaction and has a minimum post-construction V_s of 500 fps or higher. Recommendations for application of these results to the construction specifications are provided in Section 5.0.

3.0 Liquefaction Resistance of In-Situ Native Soils

Native soils will be removed to top of rock and replaced with engineered fill under each of the AP1000 structures. However, native soils will be left in-place adjacent to excavation sidewalls outside the perimeter of structures, as indicated on HAR FSAR figures 2.5.4-211A, 211B, 212A, and 212B. Liquefaction of these in-situ native soils would not directly affect AP1000 SC 1 or 2 structures (nuclear islands or Annex Buildings), but could result in loss of lateral confinement of backfill placed under the adjacent non-seismic structures (such as the Turbine Buildings).

Therefore, the liquefaction potential of native in-situ soils was evaluated using two different methods. First, soil texture-based screening criteria were used to identify soils that would not be susceptible to liquefaction because of their textural characteristics, following the method presented in Seed et al., 2003 and others (Boulanger and Idriss, 2006; Bray and Sancio, 2006). Second, evaluations were used to determine the FS against liquefaction for potentially liquefiable soils based on the observed SPT N values.

3.1 Summary of Evaluation Methods

Following is a summary of the screening and empirical liquefaction evaluation methods applied to native in-situ soils.

3.1.1 Soil Texture-Based Screening of Native Soils

A geologic and soil texture-based screening assessment of native soil samples from near the HAR 2 and HAR 3 structures was performed based on soil texture screening methods recommended by Seed et al., 2003, Bray and Sancio, 2006, and Boulanger and Idriss, 2006. Screening techniques identified in RG 1.198 involve use of the so-called "Modified Chinese Criteria," however, these techniques were not used for this assessment because they have been shown to give unconservative results in some cases (Seed et al., 2003). Native soils at the HAR sites are residual soils weathered from parent rock, and are generally present in the upper 5 to 15 feet bgs at HAR 2 and upper 10 to 25 feet bgs at HAR 3.

The screening assessment considered the following criteria:

- **Geomorphology:** Liquefaction is most commonly observed in fluvial-alluvial deposits, eolian sands and silts, beach sands, reclaimed land, and uncompacted hydraulic fills, though it can occur in other soils (RG 1.198). Youd, 1998 Table 3-1 also summarizes the estimated susceptibility of various sedimentary deposits to liquefaction during strong seismic shaking.

- **Fines Fraction:** Soils with more than 15 to 35 percent fines (passing the No. 200 sieve) usually have sufficient fines to separate individual sand and gravel grains. Such material can exhibit cohesive or cohesionless behavior depending on the characteristics of the fines (Seed et al., 2003). Where fines are either non-plastic or are low plasticity silts and/or silty clays, a potential for liquefaction can exist, therefore, further textural screening should be performed for fine-grained materials. Granular soils (with less than 35 percent fines) are evaluated for liquefaction potential based on in-situ consistency conditions (see Section 2.1.2).

- **Plasticity and Water Content (soils with significant fines content):** Soils with significant fines content can be screened for liquefaction potential based on the plasticity index (PI), liquid limit (LL), and water content (w) (Seed et al., 2003), as follows:
  - "Zone A" soils have PI < 12 and LL < 37, and are considered potentially liquefiable if w > 0.80(LL).
  - "Zone B" soils have PI < 20 and LL < 47 (and are not "Zone A"), and are considered potentially liquefiable if w > 0.85(LL).
  - "Zone C" soils have PI > 20 or LL > 47, and are not considered susceptible to liquefaction.

Laboratory index properties for soil samples collected at or near HAR 2 and HAR 3 were screened against these criteria.

Supplemental checks on the screening were also performed using methods recently recommended by Boulanger and Idriss, 2006 and Bray and Sancio, 2006. The screening methods suggested by these researchers are not as broad as those suggested by Seed et al., 2003. For example, the cutoff for liquefaction identified by Boulanger and Idriss is a PI ≤ 7. Bray and Sancio generally follow the Seed et al. criteria for PI and w relative to LL, though they drop the criteria for minimum LL values that affect liquefaction susceptibility. Use of these alternate soil screening methods results in fewer soils being considered liquefiable, and therefore Seed et al., 2003 conservatively bounds these other methods.

3.1.2 Empirical Evaluation of Native Granular Soils – Based on SPT Blowcounts:

An evaluation of the FS against liquefaction of native granular soils was performed based on empirical correlations between the liquefaction potential and the SPT N (Youd et al., 2001). For this evaluation, soils that could not be screened as non-liquefiable fine-grained soils in Section 2.1.1 above were further considered.

Field classifications of soil samples and SPT N values from boreholes under and near the AP1000 structures were used in this evaluation. This included the BGA and BPA-series boreholes advanced at locations shown on HAR FSAR Figure 2.5.4-202. Based on the screening evaluation in Section 2.1.1 above, soils which were field-classified as clay (CL or CH) were categorized as being non-liquefiable and did not require further evaluation. Soils that were field-classified as silt (ML) are also likely not liquefiable based on low water content relative to LL, but these were nonetheless further evaluated based on SPT N. Soils that were classified as sand (SW, SP, SM, SC) or gravel were considered granular, and were further evaluated based on SPT N.

CRR was calculated using the field-measured SPT N values based on the following relationships (Youd et al., 2001):

$$CRR = \frac{1}{34-(N_1)_{60CS}} + \frac{(N_1)_{60CS}}{135} + \frac{50}{[10*(N_1)_{60CS}+45]^2} - \frac{1}{200}$$

$$(N_1)_{60CS} = \alpha + \beta\,(N\ C_N\ C_E\ C_B\ C_R\ C_S\ C_{CS})$$

Where $(N_1)_{60CS}$ is the corrected, clean sand SPT N at the depth of interest, and K_σ and K_α are overburden and sloping ground correction factors, respectively. $(N_1)_{60CS}$ is calculated from the field-measured SPT N based on correction factors for overburden (C_N), hammer energy ratio (C_E), borehole diameter (C_B), rod length (C_R), and sample liner (C_S). The α and β factors were used to correct for fines content, and were conservatively assigned as 0 and 1.0, respectively, for clean sands (less than 5 percent fines).

Once the CRR at each depth of interest was calculated based on the measured SPT N value, the MSF, CCRR, CSR, and FS against liquefaction were calculated as presented in Section 1.0.

3.2 Summary of Results

Following are summaries of results from the soil texture-based screening and empirical evaluations of native soils, respectively.

3.2.1 Soil Texture-Based Screening of Native Soils

Site soils encountered in boreholes near the AP1000 structures at HAR 2 and HAR 3 are residual lean clays and sands formed by in-place weathering of parent siltstone and sandstone. Liquefaction is not commonly observed in residual soils due to the high percentage of plastic fine-grained material that makes up this type of soil and relatively high in-situ density. Residual soils are not included among the types of soils listed in RG 1.198 in which liquefaction is commonly observed (fluvial-alluvial deposits, eolian sands and silts, beach sands reclaimed land, and uncompacted hydraulic fills). Further, Table 3-1 of Youd, 1998 indicates that residual soils of Pleistocene age or older have a "very low" likelihood for susceptibility to liquefaction. This screening provides a general indication that liquefaction potential of native soils at the HAR sites is low.

Laboratory index tests were performed on 18 soil samples collected from BPA-series boreholes at HAR 2 and HAR 3, as reported in HAR FSAR Table 2.5.4-208. Fifteen of the 18 samples have fines fraction (passing the No. 200 sieve) greater than 35 percent. The LL, PI, and moisture content of these 15 fine-grained samples have been further evaluated for liquefaction susceptibility using the Seed et al., 2003 screening method, as summarized below:

- Three of the 18 samples have fines fraction less than 35 percent. These three granular soil samples represent materials that could be subject to classic liquefaction under certain in-situ conditions, and this potential has been further evaluated as summarized in Section 3.2.2 below.

- Two of the 15 fine-grained soil samples are characterized as "Zone C" soils based on the LL and PI results, and as such are not susceptible to liquefaction.

- Thirteen of the 15 fine-grained soil samples are characterized as "Zone A" or "Zone B" soils based on the LL and PI results. Test results indicate that each of these samples has a moisture content less than or equal to 0.5LL. Since moisture contents for samples in "Zone A" and "Zone B" are significantly less than 0.8LL for each fine-grained sample, none of the fine-grained samples tested are susceptible to liquefaction per the criteria in

Seed et al., 2004. These soils are further confirmed as not susceptible to liquefaction per the criteria in Bray and Sancio (2006) and Boulanger and Idriss (2006).

3.2.2 Empirical Evaluation of Native Granular Soils – Based on SPT Blowcounts:

Soil SPT samples field-classified as silt (ML) or as a granular soil (SW, SC, SM, GW, etc.) were evaluated for liquefaction susceptibility based on the SPT N and associated FS. Soil SPT samples field-classified as clay (CL or CH) were not considered susceptible to liquefaction, as confirmed by the screening assessment presented in Section 3.2.1.

Of the 369 soil SPT intervals evaluated, 11 indicate a low or intermediate FS against liquefaction based on PGA of 0.173g and a magnitude of 7.1, and are located below approximate site grade elevation of 260 feet mean sea level (MSL). Most of these represent the shallowest SPT sample collected at the respective borehole (upper few feet bgs), and each is located above elevation 250 feet MSL. Recommendations for application of these results to the construction specifications are provided in Section 5.0.

4.0 Liquefaction Resistance of Compacted Cohesive Backfill

Compacted cohesive backfill may be placed adjacent to the HAR nuclear islands in areas that are not underneath adjacent structures. As summarized in HAR FSAR Table 2.5.4-212, cohesive backfill will consist of fine-grained soils excavated on-site and placed to at least 95 percent relative compaction (modified Proctor method).

4.1 Evaluation Method

A soil texture-based screening of cohesive backfill was performed based on criteria described in Section 3.1.1. In this evaluation, the expected, as-placed moisture content of the cohesive backfill was compared to the average liquid limit of the potential backfill material (site soils). The expected, as-placed moisture content is based on the optimum moisture content from compaction tests on five fine-grained soil samples reported in the HNP (Unit 1) FSAR. No Atterberg limits were reported for these materials; however, based on the sample descriptions the Atterberg limits would likely be consistent with the HAR site samples reported in HAR FSAR Table 2.5.4-208.

4.2 Summary of Results

Figures 2.5.4-106 through 2.5.4-110 of the HNP (Unit 1) FSAR show the results of five compaction tests on fine-grained soil samples (standard Proctor method). The average optimum moisture content for the five samples is 18 percent. If one of the samples is not included in the average (Sample BC-158, S-2, with an anomalously low maximum dry density), the optimum moisture content is 15 percent.

Backfill would likely be compacted at or slightly drier than the optimum moisture content, and would not be expected to be compacted at more than 4 percent above optimum moisture content. Compaction testing using the modified Proctor method is also expected to result in slightly lower optimum moisture content than using the standard Proctor method. Based on the above information, the average as-compacted moisture content would be 22 percent or less.

As indicated in HAR FSAR Table 2.5.4-208, the average LL of native soil samples from BPA-series and BGA-series boreholes at HAR 2 and HAR 3 is 33 percent. The typical as-compacted moisture content would therefore be only 0.67(LL) for the HAR site soils. This indicates that the compacted cohesive backfill will not be susceptible to liquefaction, per the criteria summarized in Section 3.1.2 above (moisture content would be less than 0.8 [LL]). Recommendations for application of these results to the construction specifications are provided in Section 5.0.

5.0 Conclusions and Recommended Supplemental Construction Requirements

The following conclusions and recommendations are based on the results presented in Sections 2.0 through 4.0 above. Pending NRC's concurrence with the approach described in this technical memorandum, these conclusions and recommendations will be included in a future update to the HAR FSAR Sections 2.5.4.5 and 2.5.4.8.

5.1 Granular Backfill

A. The empirical evaluation of compacted granular backfill based on shear-wave velocity, as summarized in Section 2.0, indicates that this material will not be susceptible to liquefaction under a PGA of 0.173g and M = 7.1 so long as the as-compacted V_s is greater than 500 fps.

B. The secondary empirical evaluation of compacted granular backfill based on equivalent SPT N, summarized in Section 2.0, further confirms that granular backfill placed at 95 percent relative compaction (modified Proctor method) will not be susceptible to liquefaction.

C. Based on the above conclusions, future granular backfill material and construction specifications should include the following requirements:

1. The as-compacted granular backfill V_s should be 500 fps or greater at the design post-construction effective stress state (i.e., after buildings have been constructed, site grading is complete, and groundwater elevation is at site grade).
2. A program of SASW V_s testing should be conducted during construction to characterize the in-situ V_s of the compacted granular fill and to develop the relationship between in-situ measured V_s versus the mean effective confining pressure [$\sigma_m' = \sigma_v' (1 + 2K_o)/3$].
3. Details of the SASW testing program should be established prior to construction. The program is anticipated to consist of 4 to 8 SASW profiles spaced across the extents of each of the HAR Turbine Buildings. The SASW profiles should be performed at the final granular backfill surface (Turbine Building foundation subgrade).
4. The minimum mean effective stress that will confine the granular backfill should be calculated based on overburden pressure, building and surrounding fill surcharge loads, and the design groundwater elevation (at site grade). This resulting minimum mean effective confining stress should then be compared to the SASW V_s versus mean confining pressure relationship (as described in item 2 above) to demonstrate that the minimum V_s to resist liquefaction (i.e., 500 fps) is achieved.

5. A program of in-situ density tests should be performed to demonstrate that relative compaction of 95 percent or greater is achieved wherever granular backfill is placed under or adjacent to HAR structures. This program should include granular backfill outside the Turbine Building extents and adjacent to the nuclear-island sidewalls. Details of this program should be established prior to construction.

6. If granular backfill is placed adjacent to but outside the footprints of the HAR structures, where no surface building surcharge is applied, cohesive backfill may be placed to a specified depth below site grade over the granular fill to provide adequate confinement to prevent liquefaction. This depth may be determined based on the SASW V_s versus mean confining pressure relationship described above.

D. Cementaceous backfill may be used in lieu of granular backfill in some areas. This backfill would be specified to meet or exceed strength requirements in HAR FSAR Table 2.5.4-212 for granular backfill. If cementaceous backfill is used, there would be no liquefaction concern, and the V_s criteria and testing described above would not be required.

5.2 Cohesive Backfill

A. The screening evaluation of compacted fine-grained (cohesive) backfill summarized in Section 4.0 indicates that this material would not be susceptible to liquefaction after compaction. Laboratory compaction tests on the material to be used as backfill are not currently available, though the data from the HNP (Unit 1) FSAR indicate that site fine-grained soils will be acceptable backfill materials.

B. The cohesive backfill specification should require that the as-compacted material have a PI greater than 7 percent and an as-compacted moisture content less than 0.8(LL) to prevent liquefaction. Based on available test data, the HAR site fine grained soils are expected to satisfy these criteria.

C. A program of in-situ density tests should be performed to demonstrate that relative compaction of 95 percent or greater is achieved wherever cohesive backfill is placed adjacent to HAR structures.

5.3 Native Soils

A. The results of screening and empirical evaluations for native soils summarized in Section 3.0 indicate that native soils are not susceptible to liquefaction, except at isolated locations in the upper 5 to 10 feet below planned site grade (elevation 260 feet MSL).

B. It is recommended that granular backfill not be placed adjacent to these potentially liquefiable native soils. Where such soils are encountered adjacent to planned granular backfill extents, the potentially liquefiable shallow native soil should be removed and replaced with a non-liquefiable backfill (i.e., compacted cohesive backfill or very dense granular backfill meeting V_s requirements) to a horizontal distance sufficient to prevent loss of lateral confinement of the granular backfill under structures. The specific horizontal extent of such material replacement should be developed prior to construction.

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