**SAR Chapter 2, “Site Characteristics”**

**RAI NP-2.6-3:**

Provide justification for why soil boring to depths greater than 45 feet are not needed.

WCS CISF SAR Section 2.6.4 states that the WCS CISF subsurface conditions were explored with eighteen soil borings. Among the eighteen borings, four borings encountered auger refusal conditions at depths ranging from 37 to 45 feet below ground surface (bgs), and fourteen borings were terminated at 25 feet bgs. General industrial guidance for geotechnical investigations, such as US Army Corps of Engineering[1] and FHWA[2] manual/standard, recommends the boring depth, for example, (1) be at least to a depth where the increased stress due to the estimated footing load is less than 10% of the existing effective overburden stress, (2) be 1.5 times the minimum dimension of footing below the base of the footing, or (3) penetrate a minimum of 3 meters into the bedrock, if bedrock is encountered before other required depths.

**References:**

1. US Army Corps of Engineers “Geotechnical Investigations” (EM 1110-1-1804, 1 January 2001).

2. FHWA “GEOTECHNICAL ENGINEERING CIRCULAR NO. 5 Evaluation of Soil and Rock Properties” (April 2002)

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI NP-2.6-3:**

Four of the eighteen borings performed for the CISF project encountered auger refusal. The auger refusal depths ranged from 37 to 45 feet below the ground surface (bgs). Borings can be extended to a greater depth in order to obtain the soil parameters or shear wave velocities can be used to extend the soil parameters necessary for settlement analysis. In this case, shear wave surveys were performed in conjunction with the geotechnical exploration and shear wave velocities are provided to depths of 100 feet bgs. Additionally, multiple previous geotechnical investigations have been performed at the site as well as shear wave testing. The historical data outlined below were utilized to extend the soil profile and engineering parameters to a depth of 600 feet. This depth satisfies general industry guidance for settlement evaluation depth. The depth of 600 feet was selected as the termination depth due to encountering the Trujillo Sandstone Layer.

The sections below reference the previous studies that were performed along with the methodology for obtaining the necessary soil parameters to perform the settlement analyses.

Methodology:

The information from the eighteen borings and shear wave data included in the Report of Geotechnical Exploration (Attachment E to Chapter 2 of the SAR) was supplemented with data obtained from References [2], [3], and [4]. These data were used to produce a soil stratigraphic column to 600 feet along with the necessary engineering parameters required for settlement analysis. Figure NP-2.6-3-1 displays the locations of the historical borings provided.

Stratigraphy Development:

- The upper stratigraphy (to a depth of 45 feet) was based solely on the results of the eighteen soil test borings
- From a depth of 45 to 100 feet bgs, the stratigraphy was based on the Geologic Column of the CISF Area (Figure 7-30 of the SAR).
- From 100 feet to 600 feet bgs, the Geologic Column of the CISF Area (Figure 7-30 of the SAR), WCS (2007) Plate 2-2, and deeper historical borings were utilized to generate the stratigraphy.

The resulting stratigraphy, as utilized for settlement analysis at the site, is provided in Table NP-2.6-3-1.

**Table NP-2.6-3-1**
**Stratigraphy for Settlement Analysis**

| Top (feet) | Bottom (feet) | Layer Description |
|---|---|---|
| 0 | 2 | Cover Sands |
| 2 | 10 | Caliche with Sand Matrix - Moderately Hard |
| 10 | 20 | Caliche with Sand Matrix - Moderately Hard |
| 20 | 25 | Caliche - Very Hard |
| 25 | 35 | Caliche - Very Hard |
| 35 | 50 | Ogallala - Sand with Gravel |
| 50 | 80 | Ogallala - Sand with Gravel |
| 80 | 100 | Ogallala - Sand with Gravel |
| 100 | 130 | Dockum - Claystone and Siltstone |
| 130 | 230 | Claystone and Siltstone |
| 230 | 275 | Dockum - Claystone |
| 275 | 300 | Dockum - Silty Sands |
| 300 | 360 | Dockum - Claystone |
| 360 | 600 | Dockum - Claystone |

Soil Parameter Selection:

The settlement analysis that was utilized required the development of constrained modulus (elastic modulus) values. The constrained modulus values were calculated as follows:

- To a depth of 20 feet bgs, the constrained modulus was calculated using the standard penetration test (SPT) N-Values obtained in the borings. The SPT N-values were correlated to constrained modulus utilizing the method outlined in Reference [1]. This methodology was only used to a depth of 20 feet, as it is only applicable to soils with N-values up to 70 blows per foot.

- From 20 feet to 100 feet bgs, constrained modulus values were obtained by converting the shear wave velocities provided in the Report of Geotechnical Exploration to constrained modulus using the unit weight and Poisson's ratio.

- From 100 feet to 600 feet bgs, constrained modulus values were obtained by converting the shear wave velocities provided in Reference [2] to constrained modulus using the unit weight and Poisson's ratio. The unit weight and Poisson's ratio values were also obtained from Appendix A of Reference [2].

The resulting soil column is provided in Table NP-2.6-3-2.

**Table NP-2.6-3-2**
**WCS CISF Soil Column**

| Top (feet) | Bottom (feet) | N-Value (bpf) | Average Shear Wave Velocity (ft/s) | Layer Description | Constrained Modulus (ksf) |
|---|---|---|---|---|---|
| 0 | 2 | 33 | | Cover Sands | 890 |
| 2 | 10 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 10 | 20 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 20 | 25 | | 1,530 | Caliche - Very Hard | 35,815 |
| 25 | 35 | | 1,900 | Caliche - Very Hard | 55,232 |
| 35 | 50 | | 2,290 | Ogallala - Sand with Gravel | 80,233 |
| 50 | 80 | | 1,840 | Ogallala - Sand with Gravel | 53,870 |
| 80 | 100 | | 2,790 | Ogallala - Sand with Gravel | 123,857 |
| 100 | 130 | | 2,300 | Dockum - Claystone and Siltstone | 84,172 |
| 130 | 230 | | 2,755 | Claystone and Siltstone | 120,769 |
| 230 | 275 | | 2,755 | Dockum - Claystone | 120,769 |
| 275 | 300 | | 2,755 | Dockum - Silty Sands | 120,679 |
| 300 | 360 | | 2,755 | Dockum - Claystone | 120,679 |
| 360 | 600 | | 3,115 | Dockum - Claystone | 154,394 |

As shown in Table NP-2.6-3-2, the historical data available at the site, coupled with the eighteen borings and new shear wave study, has allowed the development of a stratigraphic column without additional new soil borings (to greater depths).

The soil column and parameters shown above have been utilized in the additional settlement analyses, which resulted from comments within the RAI process. The results of the settlement analyses are provided in the Revised Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR.

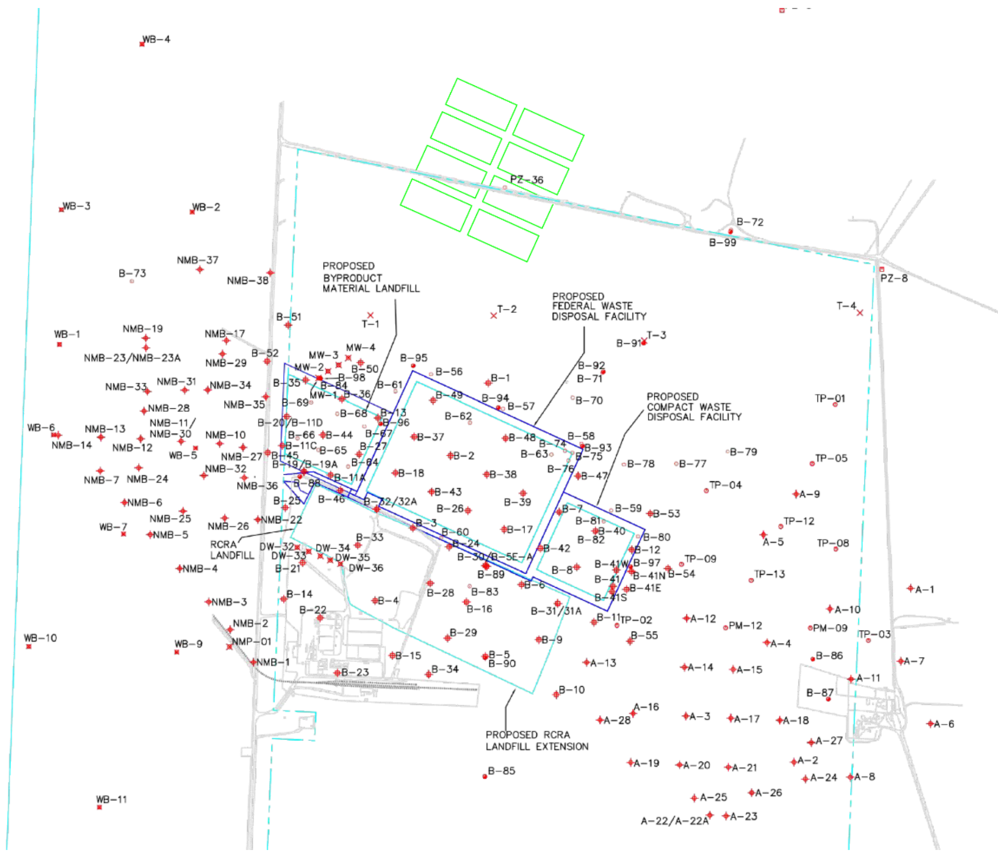



**Figure NP-2.6-3-1**
**Historical Borings at WCS Site**

**References:**

1. Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M., "Engineering Manual for Shallow Foundations," prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association of State Highway and Transportation Officials and Federal Highway Administration, Washington, D.C., Blacksburg, VA, 1991, 171 pp.

2. Waste Control Specialists LLC, "Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions," Attachment D to Chapter 2 of the SAR: AECOM, Centralized Interim Storage Facility Project, March 18, 2016.

3. Cook-Joyce, Inc., “Geology Report,” Revision 12c, Appendix 2.6.1, prepared for Waste Control Specialists, LLC, Austin, Texas, May 1, 2007.

4. Waste Control Specialists LLC, “Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste,” WCS CISF SAR Chapter 2, March 2007.

**Impact:**

SAR Attachment E to Chapter 2 has been revised as described in the response.

**RAI NP-2.6-4:**

Provide the following information with respect to the laboratory investigations:

a. Justify how the soil strength and deformation properties of the cohesive soils were determined and how the settlement potential of the clay stratum can be adequately evaluated given the absence of consolidated undrained triaxial tests and consolidated tests.

b. Provide results from the California Bearing Ratio (CBR) testing.

c. A description of the laboratory tests (including the test results) that were completed after the submittal of the Geotechnical Exploration Report (Attachment E to the SAR).

WCS CISF SAR Section 2.6.4 states the following tests were performed for this application: Atterberg Limits; Natural Moisture Content; Particle Size Analysis; Resistivity of Soil; Consolidated Undrained Triaxial Test; Standard Proctor Moisture-Density Tests; California Bearing Ratio; and Consolidation. However, Subsection 2.2 "Laboratory test program" of the Geotechnical Exploration Report (Attachment E to SAR) states that consolidated undrained triaxial tests and consolidation tests were not conducted because undisturbed Shelby tube samples could not be obtained due to the caliche. These tests are important for determining the shear strength parameters and consolidation characteristics of soil. Moreover, in the same subsection ISP indicated that one CBR test was performed. The staff reviewed ISP's soil data summary enclosed in Attachment E, Appendix B to the SAR, and the CBR testing results were not reported. Additionally, Subsection 2.2, "Laboratory test program," of the Geotechnical Exploration Report (Attachment E to SAR) states, "At the time this report was prepared, some of the laboratory testing was still on-going." In order for the NRC staff to perform a complete evaluation of the laboratory investigations, ISP should provide a complete description of the laboratory tests, including the test results.

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI NP-2.6-4:**

a) Four of the eighteen borings performed for the CISF project encountered auger refusal. The auger refusal depths ranged from 37 to 45 feet below the ground surface (bgs). In this case, shear wave surveys were performed in conjunction with the geotechnical exploration and shear wave velocities are provided to depths of 100 feet bgs. Additionally, multiple previous geotechnical investigations, as well as shear wave testing, have been performed at the site. The historical data outlined below was utilized to extend the soil profile and engineering parameters to a depth of 600 feet. This depth satisfies general industry guidance for settlement evaluation depth. The depth of 600 feet was selected as the termination depth due to encountering the Trujillo Sandstone Layer.

The sections below reference the previous studies that were performed, along with the methodology for obtaining the necessary soil parameters to perform the settlement analyses.

Methodology:

The information from the eighteen borings and shear wave data included in the Report of Geotechnical Exploration (Attachment E to Chapter 2 of the SAR) was supplemented with data obtained from References [2], [3], and [4]. These data were used to produce a soil stratigraphic column to 600 feet along with the necessary engineering parameters required for settlement analysis. Figure NP-2.6-4-1 displays the locations of the historical borings provided.

Stratigraphy Development:

- The upper stratigraphy (to a depth of 45 feet) was based solely on the results of the eighteen soil test borings
- From a depth of 45 to 100 feet bgs the stratigraphy was based on the Geologic Column of the CISF Area (Figure 7-30 of the SAR).
- From 100 feet to 600 feet bgs, the Geologic Column of the CISF Area (Figure 7-30 of the SAR), WCS (2007) Plate 2-2, and deeper historical borings were utilized to generate the stratigraphy.

The resulting stratigraphy as utilized for settlement analysis at the site is provided in Table NP-2.6-4-1.

**Table NP-2.6-4-1**
**Stratigraphy for Settlement Analysis**

| Top (feet) | Bottom (feet) | Layer Description |
|---|---|---|
| 0 | 2 | Cover Sands |
| 2 | 10 | Caliche with Sand Matrix - Moderately Hard |
| 10 | 20 | Caliche with Sand Matrix - Moderately Hard |
| 20 | 25 | Caliche - Very Hard |
| 25 | 35 | Caliche - Very Hard |
| 35 | 50 | Ogallala - Sand with Gravel |
| 50 | 80 | Ogallala - Sand with Gravel |
| 80 | 100 | Ogallala - Sand with Gravel |
| 100 | 130 | Dockum - Claystone and Siltstone |
| 130 | 230 | Claystone and Siltstone |
| 230 | 275 | Dockum - Claystone |
| 275 | 300 | Dockum - Silty Sands |
| 300 | 360 | Dockum - Claystone |
| 360 | 600 | Dockum - Claystone |

Soil Parameter Selection:

The settlement analysis that was utilized required the development of constrained modulus (elastic modulus) values. The constrained modulus values were calculated as follows:

Constrained Modulus up to 20 Feet BGS:

To a depth of 20 feet bgs, the constrained modulus was correlated to the standard penetration test (SPT) N-values obtained in the borings. The SPT N-Values were correlated to constrained modulus using the method outlined in Reference [1]. This methodology allows correlation of constrained modulus to N-value for N-values up to 70 blows per foot. The graphical representation is shown in Figure 2.6-4-2.

Constrained Modulus over 20 Feet BGS:

The borings performed for the WCS CISF site were only advanced to maximum depths of 45 feet. Additionally, the methodology outlined in Reference [1] is only valid up to N-values of 70 blows per foot. Based on the N-values obtained this methodology could only be extended to a depth of 20 feet below ground surface. Therefore, a second methodology had to be utilized to generate the constrained modulus from depths of 20 feet to 600 feet.

The Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions (Attachment D to Chapter 2 of the SAR) was used to supplement the information obtained in preparation of the Report of Geotechnical Exploration,. This document provided shear wave velocity profiles at the site to depths of approximately 1,200 feet.

The shear wave velocities were converted to constrained modulus using the following relationship:

$$V_S \xrightarrow{G=V_S^2*\rho} G \xrightarrow{M=\frac{2G(1-\nu)}{(1-2\nu)}} M$$

Where, $V_s$ = shear wave velocity
G = shear modulus
M = constrained modulus
$\nu$ = Poisson's ratio
$\rho$ = unit weight

From 20 feet to 100 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in the Report of Geotechnical Exploration to constrained modulus using the unit weight and Poisson's ratio.

- From 100 feet to 600 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in AECOM (2016) to constrained modulus using the unit weight and Poisson's ratio. The unit weight and Poisson's ratio values were also obtained from Appendix A of the AECOM (2016) report.

Results:

The methodology described above resulted in Table NP-2.6-4-2 soil column. This column will replace Appendix D in the revised Report of Geotechnical Exploration.

**Table NP-2.6-4-2**
**WCS CISF Soil Column**

| Top (feet) | Bottom (feet) | N-Value (bpf) | Average Shear Wave Velocity (ft/s) | Layer Description | Constrained Modulus (ksf) |
|---|---|---|---|---|---|
| 0 | 2 | 33 | | Cover Sands | 890 |
| 2 | 10 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 10 | 20 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 20 | 25 | | 1,530 | Caliche - Very Hard | 35,815 |
| 25 | 35 | | 1,900 | Caliche - Very Hard | 55,232 |
| 35 | 50 | | 2,290 | Ogallala - Sand with Gravel | 80,233 |
| 50 | 80 | | 1,840 | Ogallala - Sand with Gravel | 53,870 |
| 80 | 100 | | 2,790 | Ogallala - Sand with Gravel | 123,857 |
| 100 | 130 | | 2,300 | Dockum - Claystone and Siltstone | 84,172 |
| 130 | 230 | | 2,755 | Claystone and Siltstone | 120,769 |
| 230 | 275 | | 2,755 | Dockum - Claystone | 120,769 |
| 275 | 300 | | 2,755 | Dockum - Silty Sands | 120,679 |
| 300 | 360 | | 2,755 | Dockum - Claystone | 120,679 |
| 360 | 600 | | 3,115 | Dockum - Claystone | 154,394 |

As shown in Table NP-2.6-4-2, the historical data available at the site coupled with the eighteen borings and new shear wave study has allowed the development of a stratigraphic column without additional new soil borings to greater depths.

Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR has been updated to include the above information.

b) The results of the analyses are provided in the Revised Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR.

c) Two Standard Proctor Tests (ASTM D698), Two California Bearing Ratio tests (ASTM D1883), and two soil resistivity tests (ASTM G187) were performed after the submittal of the geotechnical report. These test results are attached to this document and will be reflected in the revised Report of Geotechnical Exploration.

The responses to RAIs NP-2.6-3, NP-2.6-4, NP-2.6-5, P-2.6-3, P-2.6-5 and P-2.6-6 all address the Report of Geotechnical Exploration. All of the required changes to this report (SAR Attachment E to Chapter 2) from the RAIs, are included as part of the response to RAI NP-2.6-3.

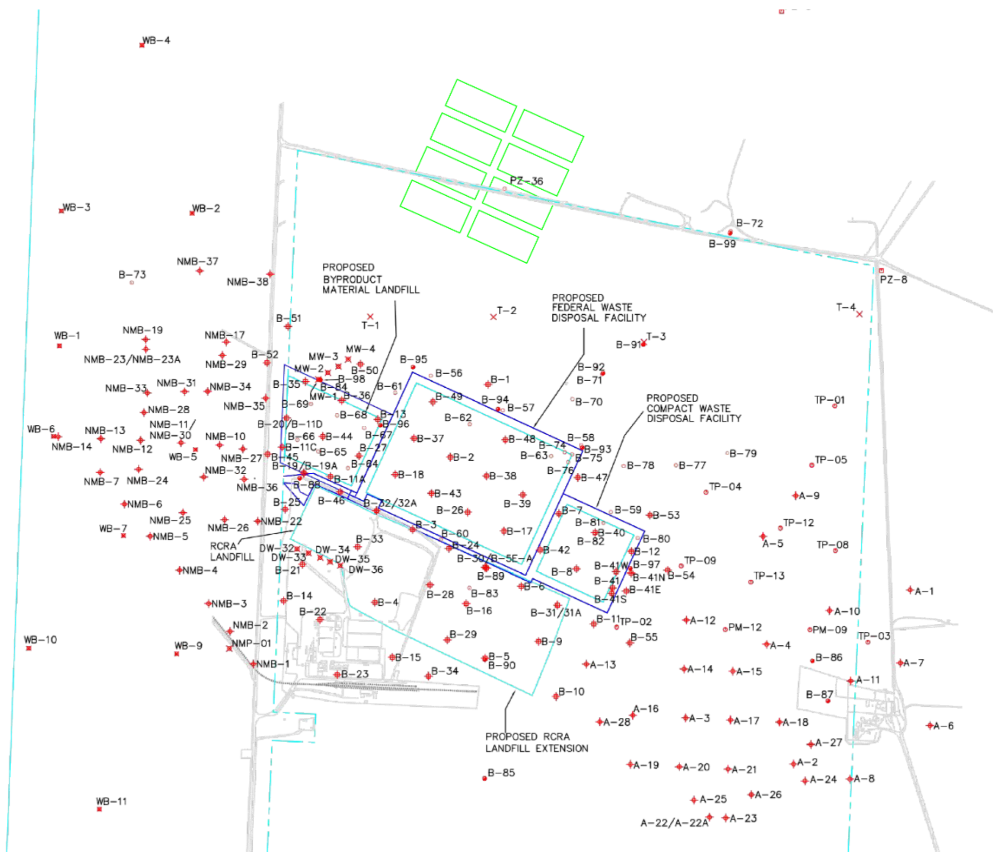



**Figure NP-2.6-4-1**
**Historical Borings at WCS Site**

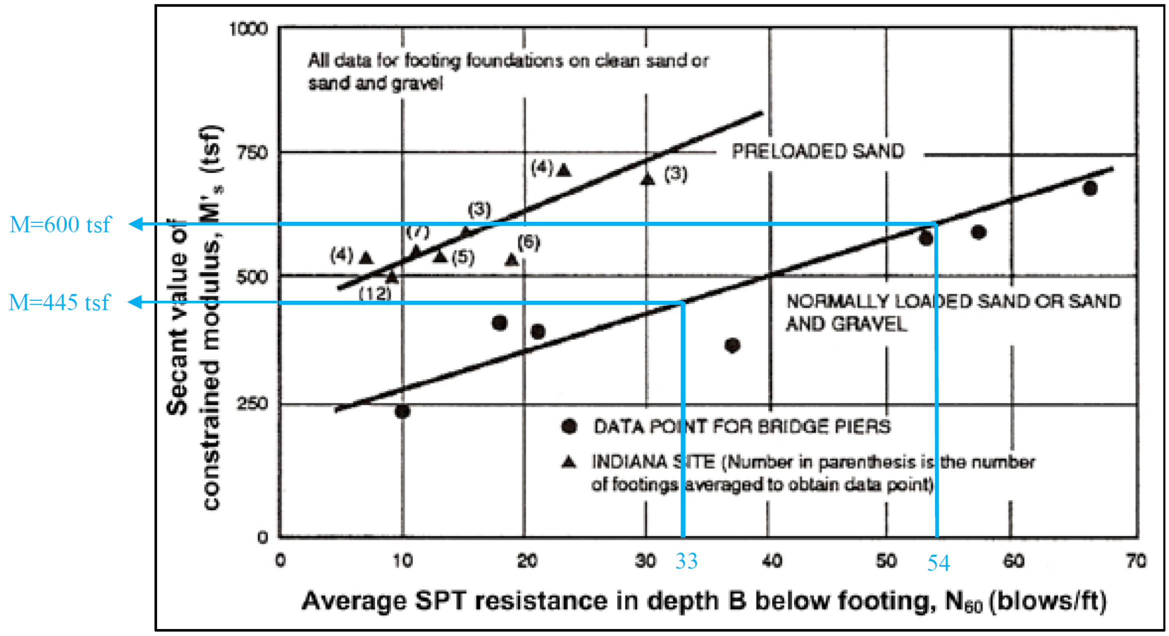



**Figure NP-2.6-4-2**
**Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]**

**References:**

1. Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M., "Engineering Manual for Shallow Foundations," prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association of State Highway and Transportation Officials and Federal Highway Administration, Washington, D.C., Blacksburg, VA, 1991, 171 pp.

2. Waste Control Specialists LLC, "Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions," Attachment D to Chapter 2 of the SAR: AECOM, Centralized Interim Storage Facility Project, March 18, 2016.

3. Cook-Joyce, Inc., "Geology Report," Revision 12c, Appendix 2.6.1, prepared for Waste Control Specialists, LLC, Austin, Texas, May 1, 2007.

4. Waste Control Specialists LLC, "Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste," WCS CISF SAR Chapter 2, March 2007.

**Impact:**

No additional changes as a result of this RAI.

**RAI NP-2.6-5:**

Provide the basis for using 20% of the dynamic modulus for the static elastic modulus as these values are considerably higher for similar soils.

Appendix D of the Geotechnical Exploration Report (Attachment E to SAR) provides the calculated static elastic moduli used for the design and analysis for a depth of 100 ft bgs. These calculated static elastic moduli are based on derived dynamic moduli from seismic wave values determined by the refraction micro-tremor (ReMi) method. Specifically, ISP used 20% of the dynamic modulus as the static elastic modulus for design and analysis. However, these elastic moduli exceed the typical range of values for similar soils reported by various engineering literatures.

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI NP-2.6-5:**

The static elastic modulus values presented in Appendix D of the Report of Geotechnical Exploration (Attachment E to Chapter 2 of the WCS CISF SAR) were utilized in the subsequent settlement calculations for the proposed foundations. In order to answer the other RAIs concerning settlement, the soil stratigraphy has been extended to a depth of 600 feet and the static elastic modulus values have been extended/revised based on additional historical information that was provided. The sections below outline the revised methodology and the resultant values that were utilized in the revised settlement analyses.

As mentioned above, it was determined that the settlement analysis would be extended to a depth of 600 feet (the top of the Trujillo Sandstone Formation). Therefore, the table provided in Appendix D needed to be extended and parameters needed to be selected for each of the stratigraphic layers. This was accomplished utilizing two distinct methodologies that were selected due to the information available from both the Report of Geotechnical Exploration and the available historical data.

Methodology 1:

To a depth of 20 feet bgs the constrained modulus was correlated to the Standard Penetration Test (SPT) N-values obtained in the borings. The SPT N-Values were correlated to constrained modulus (elastic modulus) using the method outlined in Reference [1]. This methodology allows correlation of constrained modulus to N-value for N-values up to 70 blows per foot. The graphical representation is provided in Figure NP-2.6-5.

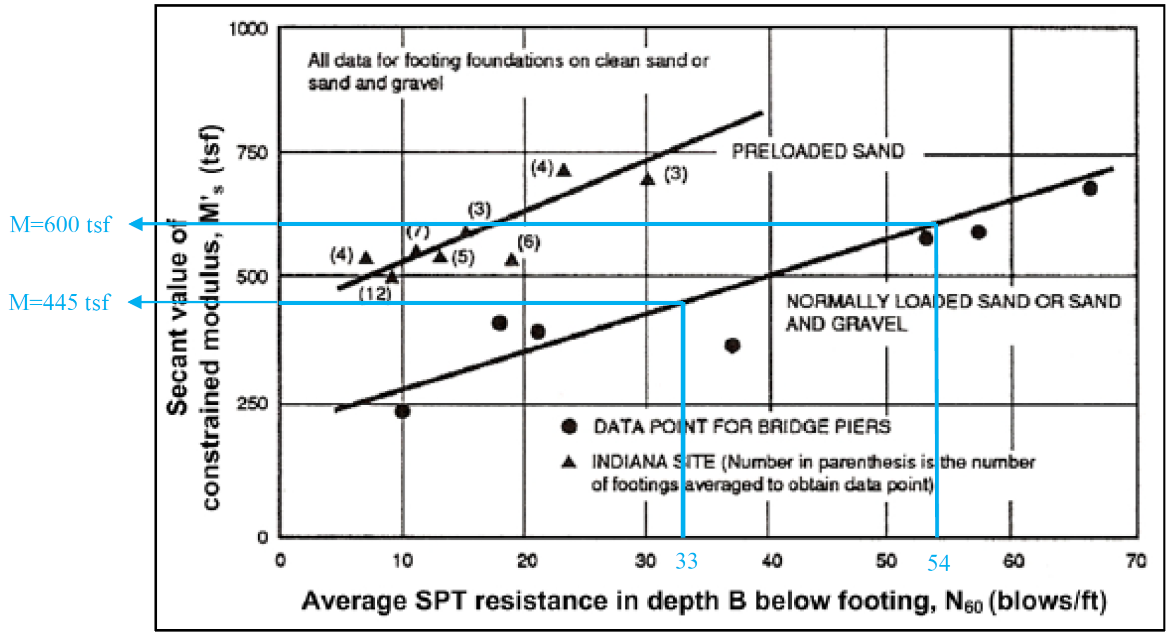



**Figure NP-2.6-5-1**
**Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]**

<u>Methodology 2:</u>

The borings performed for the WCS CISF site were advanced to maximum depths of 45 feet. Additionally, the methodology outlined in Reference [1] is only valid up to N-values of 70 blows per foot. Based on the N-values obtained this methodology could only be extended to a maximum depth of 20 feet below ground surface. A second methodology was utilized to generate the constrained modulus from depths of 20 feet to 600 feet.

To supplement the information obtained in preparation of the Report of Geotechnical Exploration, the Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, (Attachment D to Chapter 2 of the SAR) was used. This document provided shear wave velocity profiles at the site to depths of approximately 1,200 feet.

The shear wave velocities were converted to constrained modulus using the following relationship:

$$V_S \xrightarrow{G=V_S^2*\rho} G \xrightarrow{M=\frac{2G(1-\nu)}{(1-2\nu)}} M$$

Where, $V_s$ = shear wave velocity
G = shear modulus
M = constrained modulus
$\nu$ = Poisson's ratio
$\rho$ = unit weight

- From 20 feet to 100 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in the Report of Geotechnical Exploration to constrained modulus using the unit weight and Poisson's ratio.
- From 100 feet to 600 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in Attachment D to Chapter 2 of the WCS CSIF SAR to constrained modulus using the unit weight and Poisson's ratio. The unit weight and Poisson's ratio values were obtained from Appendix A of Attachment D to Chapter 2 of the WCS CSIF SAR.

Results:

The methodology described above resulted in the following soil column. This column will replace Appendix D in the revised Report of Geotechnical Exploration.

**Table NP-2.6-5-1**
**WCS CISF Soil Column**

| Top (feet) | Bottom (feet) | N-Value (bpf) | Average Shear Wave Velocity (ft/s) | Layer Description | Constrained Modulus (ksf) |
|---|---|---|---|---|---|
| 0 | 2 | 33 | | Cover Sands | 890 |
| 2 | 10 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1200 |
| 10 | 20 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 20 | 25 | | 1,530 | Caliche - Very Hard | 35,815 |
| 25 | 35 | | 1,900 | Caliche - Very Hard | 55,232 |
| 35 | 50 | | 2,290 | Ogallala - Sand with Gravel | 80,233 |
| 50 | 80 | | 1,840 | Ogallala - Sand with Gravel | 53,870 |
| 80 | 100 | | 2,790 | Ogallala - Sand with Gravel | 123,857 |
| 100 | 130 | | 2,300 | Dockum - Claystone and Siltstone | 84,172 |
| 130 | 230 | | 2,755 | Claystone and Siltstone | 120,769 |
| 230 | 275 | | 2,755 | Dockum - Claystone | 120,769 |
| 275 | 300 | | 2,755 | Dockum - Silty Sands | 120,679 |
| 300 | 360 | | 2,755 | Dockum - Claystone | 120,679 |
| 360 | 600 | | 3,115 | Dockum - Claystone | 154,394 |

Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR has been updated to include the above information.

The responses to RAIs NP-2.6-3, NP-2.6-4, NP-2.6-5, P-2.6-3, P-2.6-5 and P-2.6-6 all address the Report of Geotechnical Exploration. All of the required changes to this report (SAR Attachment E to Chapter 2) from the RAIs, are included as part of the response to RAIs NP-2.6-3.

**References:**

1. Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M., “Engineering Manual for Shallow Foundations,” prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association of State Highway and Transportation Officials and Federal Highway Administration, Washington, D.C., Blacksburg, VA, 1991, 171 pp.

**Impact:**

No additional changes as a result of this RAI.

**SAR Chapter 4, "Facility Design"**

**RAI NP-4-4:**

Describe the important-to-safety movement of a NAC fuel canister in its transportation cask from a railcar to the canister transfer system (CTS) and provide drawings of the major structures, systems, and components intended for this function.

The described movement of the NAC canisters from the railcar to the CTS using the vertical cask transporter (VCT) appears inconsistent with provided drawings of the cask handling building (CHB) and VCT. WCS CISF SAR Section 4.7.4, "NAC Cask Transfer System," describes that the VCT is used to unload the NAC transportation casks from the railcar in the following manner:

> …After the transportation cask has been received, including removal of the impact limiters, the VCT is driven over, essentially straddling the railcar, and is positioned to engage the transportation cask upper trunnions. The VCT then raises and moves towards the rear of the cask to raise and lift the transportation cask from the railcar. The VCT then lowers the transportation cask to 3-6" off the ground. The railcar is removed from the unloading area and the VCT moves the cask to the CTS. The VCT is shown in Figure 4-4.

WCS CISF SAR Section 7.5.2, "Vertical Cask Transporter (VCT)," describes that the VCT lift removing the transportation cask for vertical storage cask systems from the railcar within the CHB is considered important to safety. However, WCS CISF SAR Figure 4-4 [Proprietary] depicts a mobile, hydraulic gantry hoist with less than a 5-foot hoist range, which is insufficient to upright a transportation cask that is over 15-feet in height from a horizontal position. Furthermore, Figure 1-7, "Cask Handling Building Plan," and Figure 1-8, "Cask Handling Building Section View," depict train rails traversing the entire CHB with the rails approximately at the finish grade of the CHB floor, which appears to preclude positioning the U-shaped VCT frame depicted in Figure 4-4 such that it can move over the railcar "towards the rear of the cask."

This information is needed to determine compliance with the 10 CFR 72.24(d).

**Response to RAI NP-4-4:**

ISP has determined that it can use the Canister Handling Building (CHB) overhead cranes to receive and remove the NAC Transportation casks from the railcar. The transport cask off-loading and loading operations are described in updated Safety Analysis Report (SAR) Section 1.3.1.3 and will utilize lifting equipment designed in accordance with ANSI N14.6 for each of the three NAC transportation cask systems included in the SAR. Drawings for the three lifting yokes to be used to upright the NAC transportation casks have been added to Section 4.10 and applicable materials information has been added to Section 15.

In addition, SAR Sections 1.1, 1.2.3, 1.3.1.1, 3.3.1, 3.4.1, 4.1.2, 4.7.1, 4.7.2, 4.7.3, 4.7.4, 4.7.4.1, 5.1.3.1.2, 5.1.3.5, 5.2.1.1.1, 5.2.1.2.1, 5.2.1.3.2, 7.5, 7.5.2.4, 7.5.3, 7.5.3.1.3, 7.6.1.1, and 9.1.3 have been updated to be consistent with the use of the CHB overhead cranes to remove the casks from the railcars.

**Impact:**

SAR Sections 1.1, 1.2.3, 1.3.1.1, 1.3.1.3, 3.3.1, 3.4.1, 4.1.2, 4.7.1, 4.7.2, 4.7.3, 4.7.4, 4.7.4.1, 4.10, 5.1.3.1.2, 5.1.3.5, 5.2.1.1.1, 5.2.1.2.1, 5.2.1.3.2, 7.5, 7.5.2.4, 7.5.3, 7.5.3.1.3, 7.6.1.1, 9.1.3 and 15 have been revised as described in the response.

SAR Table 3-5, Table 7-1 and Table 7-25 have been revised as described in the response.

SAR Sections 15.1.6, 15.2.5 and 15.3.5 have been added as described in the response.

**SAR Chapter 7, "Installation Design and Structural Evaluation"**

**RAI NP-7-3:**

Make appropriate adjustments to the SASSI model to account for concrete cracking to ensure consistency with the GTSTRUDL model. Report these findings in WCS CISF SAR Section 7.6.1.5 and/or other appropriate sections of the WCS CISF SAR.

In the GTSTRUDL model used to evaluate all of the load combinations, the concrete pad flexural stiffness is reduced by 50% to account for concrete cracking. However, in the SASSI soil structure interaction (SSI) model the concrete pad is considered to be uncracked and the flexural stiffness is not reduced (ENERCON CALC NO. NAC004-CALC-04, Rev. 1, "Soil Structure Interaction Analysis of ISFSI Concrete Pad at Andrews, TX," Page 34). In the load combinations, safe shutdown earthquake (SSE) occurs with Dealload (D) and Liveload (L). If the concrete pad is cracked under D and L, then it must be cracked under SSE. The GTSTRUDL and SASSI models must be consistent in their assumptions regarding concrete cracking. In the SSI analysis it is conservative to consider the concrete cracked. Had the concrete been considered cracked, it is estimated that the acceleration at the center of gravity of the cask would be higher by approximately 10%. (Reference: G. Bjorkman, "Influence of ISFSI Design Parameters on the Seismic Response of Dry Storage Casks," PATRAM 2010, London.)

This information is needed to determine compliance with 10 CFR 72.24 (d)(2).

**Response to RAI NP-7-3:**

The SASSI model used in the soil-structure interaction (SSI) analysis has been modified in Revision 2 of Calculation NAC004-CALC-04 [3] to include cracked concrete properties consistent with the GTSRUDL model, per American Society of Civil Engineers (ASCE) 43-05 [1]. The flexural rigidity and shear rigidity have been reduced to half per Table 3-1 of ASCE 43-05 [1]. The damping ratio has been increased to 7 percent for cracked concrete per ASCE 43-05 Table 3-2, consistent with Response Level 2 [1]. In revision 2 of NAC004-CALC-04, the SSI analysis was performed using cracked concrete properties for the governing analysis cases to obtain the maximum cask time history acceleration, maximum cask sliding potential, and maximum cask overturning potential (as determined in Section 7 of Reference [2]. The results are documented in Attachment 4.1 of NAC004-CALC-04. SAR Sections 7.6.1 and 7.6.2, including applicable subsections; Tables 7-11 through 7-20; Figures 7-10 through 7-13 and 7-15 through 7-27 have been updated to be consistent with the revised analysis. In addition, Figures 7-64 through 7-67 have been added to the SAR.

Based on the results of the SSI analysis using cracked concrete properties, the accelerations for the governing cases for design increased by up to 6.5 percent compared to the uncracked models. Consequently, the accelerations obtained from the previously completed full (36 cases) SSI analysis documented in Reference [2] have been increased by 10 percent (multiplied by a factor of 1.1) and used in a design re-evaluation of the pad as documented in Reference [4]. The revised accelerations have been used in the GTSTRUDL analysis for the four (4) controlling cask configurations for design as presented in Reference [4]. For cask sliding and overturning and pad sliding evaluations, the design parameters extracted from SSI re-analyses (Reference [3]) are directly used (Reference [4]). The design of the pad was not impacted by the changes in acceleration inputs. While the factor of safety against overturning decreased by approximately 6.7 percent, it remained above the requirement of 1.1 and, while the sliding coefficient increased slightly, sliding distance has still been shown to be within acceptable limits.

The responses to RAI NP-7-3, NP-7-4 and NP-7-7 all address the evaluation of the Storage Pads for the NAC systems. All of the required changes to SAR Sections 7.6.1 and 7.6.2, including subsections, are included as part this response.

Similarly, SAR Attachment E to Chapter 2 updates are included as part of the response to NP-2.6-3.

**References:**

1. ASCE 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.”

2. Enercon Calculation No. NAC004-CALC-04, Revision 1, “Soil Structure Interaction Analysis of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at WCS Site in Andrews, TX.”

3. Enercon Calculation No. NAC004-CALC-04, Revision 2, “Soil Structure Interaction Analysis of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at WCS Site in Andrews, TX.”

4. Enercon Calculation No. NAC004-CALC-01, Revision 2, “Licensing Design of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at Andrews, TX.”

**Impact:**

SAR Sections 3.2.3.5, 7.6.1, 7.6.2 and 7.6.3, Tables 7-11 through 7-20,and Figures 7-9 through 7-13 and 7-15 through 7-27 been revised, and Figures 7-64 through 7-67 have been added as described in the response.

**RAI NP-7-4:**

Ensure the soil springs in the GTSTRUDL model reflect the behavior of the storage pad under applied loads. Make any changes to WCS CISF SAR Section 7.6.1.5 and/or other appropriate sections of the WCS CISF SAR.

In WCS CISF SAR Section 7.6.1.5, subheading “Nonlinear Soil Springs" it states:

> Nonlinear (compression only) springs are included at each storage pad node using the GTSTRUDL function…. The GTSTRUDL command uses the user input soil stiffness…. combined with the tributary area from each node's connecting element(s) to compute a spring stiffness in force per unit length.

The resulting soil springs are uncoupled and are commonly referred to as a "Winkler" foundation (M. Hetenyi, "Beams on Elastic Foundation," University of Michigan Press, 1946; and J. Bowles, "Foundation Analysis and Design," McGraw-Hill, Fourth Edition, 1988). Because of the way the soil spring stiffness is calculated, a uniformly distributed load applied to the storage pad will produce a uniform downward displacement everywhere. By contrast, if the storage pad were placed on an elastic half-space and a uniform load were applied, the displacement would not be uniform but concave downward, which is in agreement with measured test results (Bowles, 1988). One way to account for this using a Winkler foundation is to double the stiffness of the soil springs at and near the edges of the pad (Bowles, 1988).

This information is needed to determine compliance with 10 CFR 72.24 (d)(2).

**Response to RAI NP-7-4:**

<u>Background</u>

WCS CISF SAR Section 7.6.1.5, subheading “Nonlinear Soil Springs” states: “Nonlinear (compression only) springs are included at each storage pad node using the GTSTRUDL function….” The GTSTRUDL command uses the user input soil stiffness….combined with the tributary area from each node’s connecting element(s) to compute a spring stiffness force per unit length.

We agree the resulting soil springs are uncoupled and are commonly referred to as a “Winkler” foundation based on his 1867 paper which first proposed the concept of subgrade reaction. In its traditional form, the Winkler method makes the assumption that each “spring” is linear and acts independently from one another. It also assumes that all springs have the same value of $K_s$ (modulus of subgrade reaction). The Winkler method is an improvement over rigid analyses; however, it is still thought of as only a coarse representation of the actual interaction between soils and mat foundations.

The RAI references a mechanism for generating more accurate estimates of actual settlements, "One way to account for this is to double the stiffness of the soil springs at and near the edges of the pad (Bowles, 1988)." Bowles outlines multiple methods for coupling the springs from the Winkler method. One method is to double the stiffness of the soil at and near the edges of the pad or to double the quantity of springs at or near the edges of the pad and leave the $K_s$ value constant. These methods will result in a more accurate estimate of settlement under a structure that is uniformly loaded. However, Bowles goes on to state that this method is only valid if the plate or mat is uniformly loaded.

Unfortunately, the mat foundations for the CISF area will be loaded in stages and even when all casks are fully loaded, will not be uniformly loaded. As such, the methodology outlined by Bowles for coupling will not achieve the desired effect.

Methodology

As mentioned previously and referenced in other RAI's, the WCS CISF SAR Section 7.6.1.5 utilizes a single modulus of subgrade reaction ($K_s$) of 150 pounds per cubic inch for the entirety of the area beneath the mat foundation.

The modulus of subgrade reaction is a conceptual relationship between soil pressure and deflection. It is not a soil property. The modulus value is the direct relationship of pressure/deflection. Since the modulus value is a direct function of the load distribution on the mat (which is unknown until a preliminary modulus is selected), the modulus of subgrade reaction traditionally selected to begin with is based on a plate load test (ASTM D1196). As referenced in the RAI, by using the single subgrade modulus, the result was an uncoupled Winkler analysis.

Analysis Procedure

In order to obtain realistic deflections with complex loading, the subgrade modulus determined using the plate load test per ASTM D1196 needs to be adjusted for loads applied over a much larger area than the plate such as the mat foundations present at the WCS CISF. To address this issue, the geotechnical engineer must work with the structural engineer to adjust the subgrade modulus through an iterative process. Since the loading (pressure) on the mat can be calculated from the structural model, that pressure can be utilized to generate the associated settlements and ultimately, determine more realistic modulus of subgrade reaction ($K_s$) values at various points beneath the slab.

The analysis procedure for this project consisted of multiple iterations and proceeded as follows:

1. The first iteration of the settlement analysis was performed using mat pressures provided by the structural engineer. As indicated, a single value for modulus of subgrade reaction was used at all points below the slab for this first iteration.

2. These pressures were used to develop a Settle3 model (finite difference software) with the end goal of formulating values of subgrade modulus (k) that would align with the calculated settlements. The program calculates settlements at multiple points beneath the mat based on the pressures provided. The modulus values are calculated at distinct points by determining the pressure/settlement ratios.

3. The resulting new values of subgrade modulus were then submitted to the structural engineer to be integrated into the GTSTRUDL analysis.

4. The next iteration combined the applied loads with a much more accurate estimate of soil response (calculated k values) thus refining the mat pressure distribution.

5. The results of the refined GTSTRUDL analysis were then provided and used to update the Settle3 model. The result was an updated set of subgrade modulus values for the entire mat for input back into the GTSTRUDL analysis.

6. This iterative process was continued until the models converged (calculated soil modulus values and displacements did not change more than 10 percent between consecutive iterations).

The analysis was performed on a single pad in four different loading configurations: fully loaded, quarter loaded, half loaded, and three quarters loaded. Plots showing the converged models and subsequent subgrade modulus values and anticipated settlements are included as Appendix H of the Report of Geotechnical Exploration, Revision 2, which is included in updated Attachment E to Chapter 2 of the WCS CISF SAR.

The pad design calculation was revised to reflect the new subgrade modulus in Revision 2 of NAC004-CALC-01. The design of the pad was not impacted by the changes in subgrade modulus.

The responses to RAI NP-7-3, NP-7-4 and NP-7-7 all address the evaluation of the Storage Pads for the NAC systems. All of the required changes to SAR Sections 7.6.1 and 7.6.2, including subsections, are included as part of the response to RAIs NP-7-3.

Similarly, SAR Attachment E to Chapter 2 updates are included as part of the response to NP-2.6-3.

**Impact:**

No additional changes as a result of this RAI.

**RAI NP-7-7:**

In WCS CISF SAR Sections 7.6.4.2 and 7.6.5.1, explain whether the concrete pad is assumed to be cracked or uncracked in the structural and SSI analyses.

Based on the value of Young's modulus used in the structural analysis and the SSI analysis, it appears that the concrete pad is considered to be uncracked. If this is correct, please explain the basis for this assumption.

This information is needed to determine compliance with and 72.24 (c) (d)

**Response to RAI NP-7-7:**

In the original SSI analysis of the pad (Reference [2]), the concrete was assumed to be uncracked. In response to NRC RAI NP-7-3, the SSI analysis has subsequently been revised (Reference [3]) and the SASSI model has been modified to include cracked concrete properties per ASCE 43-05 (Reference [1]). The results of this revised analysis have been incorporated into the evaluation of the concrete pad as shown in Reference [4]. The design of the pad was not impacted by the revised results and the sliding and overturning results remained within acceptable limits. See the response to RAI NP-7-3 for additional details.

**References:**

1. ASCE 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities.”

2. Enercon Calculation No. NAC004-CALC-04, Revision 1, “Soil Structure Interaction Analysis of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at WCS Site in Andrews, TX.”

3. Enercon Calculation No. NAC004-CALC-04, Revision 2, “Soil Structure Interaction Analysis of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at WCS Site in Andrews, TX.” (included in Enclosure 3 of submittal E-56508)

4. Enercon Calculation No. NAC004-CALC-01, Revision 2, “Licensing Design of Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at Andrews, TX.” (included in Enclosure 9 of submittal E-56415)

**Impact:**

No change as a result of this RAI.

**RAI NP-7-8:**

With respect to WCS CISF SAR Section 7.6.5.4, provide the proprietary settlement calculations for the NUHOMS storage pad for staff review.

Without reviewing the storage pad settlement calculations, the staff is unable to make a safety finding.

This information is needed to determine compliance with 10 CFR 72.24(d)(2).

**Response to RAI NP-7-8:**

The responses to RAIs NP-7-3, NP-7-4, and NP-7-7 all address the evaluation of the Storage Pads for the NAC systems. All of the required changes to SAR Sections 7.6.1 and 7.6.2, including subsections, have been included as part the response to RAI NP-7-3.

Similarly, SAR Attachment E to Chapter 2 updates have been included as part of the response to RAI NP-2.6-3.

The proprietary settlement calculations for the NUHOMS® storage pad are included in calculation AREVATN001-CALC-001, Revision 2 (included in Enclosure 3 of submittal E-56508), which has been updated to include the revised to reflect the changes in SAR Attachment E to Chapter 2 discussed above. In addition, calculation AREVATN001-CALC-001, Revision 2 has also been updated along with SAR Section 7.6.5 to be consistent with the NAC pad evaluations included in the response to RAI NP-7-3.

Finally, SAR Section 7.5.3.5 is also updated to reflect the revised soil bearing properties included in SAR Attachment E to Chapter 2 which has been updated as part of the response to RAI NP-2.6-3.

**Impact:**

SAR Sections 7.5.3.5, and 7.6.5, Tables 7-38 through 7-40, and Figures 7-49, 7-50, and 7-53 have been revised as described in the response.

**SAR Chapter 15, "Materials Evaluation"**

**RAI NP-15-10-S:**

Clarify bolting material listed on WCS SAR page 15-8.

SAR page 15-8 has a listing for ASTM A574 Grade 70, but the reference cited, "Structural and Thermal Material Properties – MAGNASTOR/MAGNATRAN Cask System," NAC Calculation 71160-2101 Rev. 9, NAC International, Atlanta, Georgia (Reference 15-3), does not contain information for ASTM A574 material.

There are two issues that need clarification:

1. ASTM A574 is not in Reference 15-3 but yield strength and tensile strength values listed on SAR Page 15-8 are correct according to ASTM A574.

2. ASTM A574 has multiple grades including: 4137, 4142, 4145, 4340, 8740, 5137M, and 51B37M, but no Grade 70. The Grades of ASTM A574 refer to alloy designations (i.e., 4340 Cr-Mo steel) rather than strength (e.g., A516 Grade 70). The yield strength and tensile strength of all grades of ASTM A574 is 135 ksi (minimum) and 170 ksi (minimum) respectively which is much stronger than a typical "Grade 70" steel which usually refers to an alloy with a tensile strength of 70 ksi.

This information is necessary to assure compliance with 10 CFR 72.24(c)(3) and (c)(4).

**Response to RAI NP-15-10-S:**

1. Section 15.3.2.5 has been revised to add new Table 15-3, which includes the applicable Material Properties from the 2001 Edition of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Materials Section, Part D – Properties, which is now cited as new Reference [15-6].

2. Section 15.3.2.5 has been revised to remove "Gr 70" and Reference [15-3] from the notation.

**Impact:**

SAR Section 15.3.2.5 has been revised and Table 15-3 has been added as described in the response.

**RAI NP-15-13-S**

Provide the following:

1. The location of the referenced tables in the RAI response: The response to RAI NP-15-13 refers to (1) SAR Table 15.3-1 comparing the FO, FC and FF DSCs to the DSC subcomponents evaluated in the 1004 renewal, (2) SAR Table 15.3-2 comparing the GTCC DSCs to the DSC subcomponents evaluated in the 1004 renewal, (3) SAR Table 15.3-3 comparing the 24PT1 DSC to the to the DSC subcomponents evaluated in the 1004 renewal and, (4) SAR Table 15.3-4 comparing the AHSM to the HSM subcomponents evaluated in the 1004 renewal. SAR tables corresponding to Tables RAI 15.13-1 through RAI 15.13-4 were not included with the SAR change pages provided with the RAI response.

2. The applicability of the CoC No. 1004 AMPs to the 24PT1 DSC and the AHSM in the response to RAI NP-15-13: In their RAI response, the applicant stated:

   *SAR Section B.13 has been added to Appendix B to require the AMPs in Appendix C, Section C.13, to be applied to the Standardized Advanced NUHOMS® System (i.e., the 24PT1 DSC and the AHSM). SAR Tables 15.3-3 and 15.3-4 review the subcomponents of the 24PT1 DSCs and AHSM, compare them to corresponding DSC and HSM subcomponents evaluated in the Renewed CoC 1004, and conclude that no AMA is required or that the AMPs in CoC 1004 are applicable. Therefore, the AMPs in Appendix C (SAR Section C.13) are applicable to the SSCs of the MP187 system proposed for storage at the WCS CISF.*

It appears that the underlined statement should refer to the Standardized Advanced NUHOMS® System and the 24PT1 DSC. The preceding paragraph in the RAI 15-13 response addresses the FO, FC, and FF DSCs of the MP187 system.

3. The CoC No. 1004 renewal time limited aging analyses (TLAAs), if any, which will be used to manage aging effects in the period of extended operation: Table RAI 15.13-1 through RAI 15.13-4 include a column titled CoC No. 1004 Aging Management Activity. The entries in this table only refer to aging management programs (AMPs). No TLAAs are listed in these tables. Several TLAAs in the CoC No. 1004 renewal that were incorporated into Rev. 17 of the CoC No. 1004 FSAR Section 12.2 would appear to be applicable including:

   - Fatigue Evaluation of the Dry Shielded Canisters
   - Horizontal Storage Module Concrete and Dry Shielded Canister Steel Support Structure Thermal Fatigue, Corrosion, and Temperature Effects Evaluation
   - Dry Shielded Canister Poison Plates Boron Depletion Evaluation
   - Evaluation of Neutron Fluence and Gamma Radiation on Storage System Structural Materials
   - Confinement Evaluation of 24P and 52B Non-Leaktight DSCs
   - Thermal Performance of Horizontal Storage Modules for the Period of Extended Operation
   - Evaluation of Additional Cladding Oxidation and Additional Hydride Formation Assuming Breach of Dry Shielded Canister Confinement Boundary

- Evaluation of Cladding Gross Rupture during Period of Extended Operation

4. Revisions to any TLAAs approved in the CoC No. 1004 renewal and incorporated into CoC No. 1004 UFSAR Revision 17 that do not consider the proposed actions and loadings associated with the transportation of the existing DSCs currently in service at other specifically licensed and generally licensed ISFSIs: The movement of DSC to the proposed ISP/WCS CISF facility should consider additional parameters associated with the transfer and transportation operations as necessary. For example, it appears that the Fatigue Evaluation of the Dry Shielded Canisters included in Section 12.2 of the CoC No. 1004 UFSAR Revision 17 does not address loading cycles associated with the movement of DSC to the proposed ISP/WCS CISF facility including: (1) DSC loading during removal from the existing HSM, (2) DSC loading and temperature cycles during transportation package leak testing prior to transportation, (3) loads during transportation, (4) temperature during transportation, (5) DSC loading and temperature cycles during transportation package testing upon receipt at the ISP/WCS CISF facility, and (6) DSC loading during placement into the HSM at the ISP/WCS CISF facility.

5. Additional information on the assessment of ITS components in Tables RAI 15.13-1 through RAI 15.13-4 where the comparison component for the CoC No. 1004 system was NITS: Entries in the columns of Table RAI 15.13-1 (page 38 of 93 of the RAI response) for the Stop Plate (2nd row) and the Bottom Shield Plug (6th row) are considered ITS for the FO, FC and FF DSCs currently located at the Rancho Seco ISFSI but are NITS for the CoC No. 1004 system. The assessment of the ITS components should consider the ITS function, the range of possible aging mechanisms and the operating environment. The applicant should also review Tables 15.13-2 thru 15.13-4 for similar entries.

6. Additional information on the NITS components for the FO, FC, FF in Table RAI 15.13-1, the GTCC DSC in Table RAI 15.13-2, the 24PT1 Component of Table RAI 15.13-3 and the AHSM components in Table RAI 15-13-4: Specifically, provide additional information on the screening assessment and the determination on whether these components might be screened in under category 2 in accordance with the guidance in NUREG-1927 Revision 1 Section 2.4.2.

7. Revised material information for the GTCC DSC and the DSCs from the CoC No. 1004 in Table RAI 15.13-2: The information provided in this Table RAI 15.13-2 appears to contain many errors on the materials used for the DSC components. For example, the Outer Bottom Cover Plate in Table RAI 15.13-2 is listed as SA-240 Type 304 for the GTCC Material and A240 Type 304 for the CoC No. 1004 Material. These appear to be reversed. The DSCs approved for spent fuel storage under the CoC No. 1004 system used SA-240 Type 304. The GTCC canisters used A240 Type 304.

8. Aging management reviews for the FO, FC, FF, GTCC and 24PT1 DSCs and the AHSM: Tables RAI 15.13-1, through RAI 15.13-4 provide a crosswalk to justify the application of the approved CoC No. 1004 AMPs to the FO, FC, FF, GTCC and 24PT1 DSCs and the AHSM. While Tables RAI 15.13-1 through RAI 15.13-4 identify the safety classification of the subcomponent parts, the safety function(s) of the subcomponent parts are not identified. The CoC No. 1004 renewal (along with other CoC and specific license renewals) have included an aging management review with the safety functions of the ITS SSCs identified. The staff has used the information in the aging management review to evaluate the adequacy of the proposed aging management activity. The information provided in previous renewals has been consistent with the guidance in NUREG-1927 Revision 1 Section 3.2.

Without information on the safety functions of the ITS SSCs, the staff cannot determine whether the proposed aging management activities are sufficient to maintain the safety function of the ITS SSCs throughout the period of extended operation.

9. The use of surrogate inspections identified in SAR Sections C.13.3.1 and C.13.4.1: The revised SAR pages in Appendix C state the following:

   *Interim Storage Partners (ISP) may use inspections results from other general or specific licensee inspections if it can be demonstrated that the other licensee inspections are bounding. Parameters to be considered in making a bounding determination include: similar or more benign environmental conditions, similar storage system design components, similar stored fuel parameters, heat load, and operational history.*

   The staff notes that Sections C.13.3.4 and C.13.4.4 state the following:

   *A minimum of one DSC from each originating ISFSI, is selected for inspection. The DSC(s) selected for inspection is based on the following considerations/criteria which provide the basis for selection of a bounding DSC(s): (1) Time in service, (2) Initial heat load, (3) DSC Fabrication and Design Considerations and (4) HSM array configuration relative to climatological and geographical features.*

   Sections C.13.3.4 and C.13.4.4 do not address the potential use of surrogate inspections.

   NUREG-1927, Revision 1, notes that the use of surrogate inspections may be acceptable only when substantial operating experience provides a basis for their use. Table B-1 notes that an approach of using surrogates would need to be justified on a case-by-case basis by an applicant, considering canister examination results for the susceptibility rankings.

   In addition, in the Response to December 21, 2016, Nuclear Energy Institute Submittal: NEI 14-03, “Format, Content and Implementation Guidance for Dry Cask Storage Operations-Based Aging Management,” Revision 2 (ML18325A207) the NRC clarified the additional information necessary for the use of surrogates for AMP inspections:

   *The NRC has not approved the use of surrogates for AMPs to date. There is not yet substantial operating experience for canister examinations for the various susceptibility rankings to understand how the susceptibility assessments may be applied, and surrogates used, across the Independent Spent Fuel Storage Installation fleet. There is not yet a technical basis for the use of surrogate inspections for canisters until the Code Case is developed and operating experience exists for canister examination results for the various susceptibility rankings. For other structures, systems, and components (SSCs) within the scope of renewal, there are limited AMP inspection results and no industry guidance for determining which SSCs may be appropriate for the use of surrogate inspections. Both a guidance document that considers the effects of environmental and operational parameters on aging effects and operational experience gained from conducting AMP inspections are necessary for identifying potential surrogates for SSCs other than storage canisters.*

This information is needed to determine compliance with 10 CFR 72.42(a) and 72.120(a).

**Response to RAI NP-15-13-S:**

1. Tables NP-15-13-1 through NP-15-13-4 summarize the results of the aging management reviews (AMRs) performed for the various subcomponents and provides an explanation why the certificate of compliance (CoC) No. 1004 AMPs are applicable to the FO, FC, FF, Greater than Class C (GTCC), and 24PT1 dry shielded canisters (DSCs) and the advanced horizontal storage module (AHSM). A review of the renewed CoC 1004 UFSAR and the renewal submittals for CoC No. 1029 (NP-15-13-S Item 2) determined that these renewal submittals did not include detailed aging management review (AMR) results tables. To be consistent with previous renewal submittals, ISP did not intend to include Tables NP-15-13-1 through NP-15-13-4 in the safety analysis report (SAR). However, ISP has revised Sections A.13, B.13, C.13, and D.13 to clearly reference the submittals that document the AMRs performed for each structure, system, and component (SSC).

2. The initial response to RAI NP-15-13 erroneously referenced the MP187 System when the rest of the paragraph was discussing the Standardized Advanced NUHOMS® System. The sentence has been corrected to reference the Standardized Advanced NUHOMS® System proposed for storage at the WCS CISF.

3. Tables NP-15-13-1 through NP-15-13-4 have been revised to identify when an aging effect is being managed via a TLAA for the FO, FC, FF, GTCC, and 24PT1 DSCs. These TLAAs were identified in the AMRs for the MP187 System and Standardized Advanced NUHOMS® System in References [1] and [2]. In addition, subsections have been added to Chapters A.13 and B.13 listing the TLAAs identified during the AMR of these systems. A statement has also been added to Chapters C.13 and D.13 stating the TLAAs in the CoC No. 1004 renewal application [3] are applicable to the 61BT and 61BTH canisters, respectively.

   Please note that stainless steel pressure boundary components have two aging mechanisms that are evaluated, Stress Corrosion Cracking (SCC) and thermal fatigue. SCC is managed via an AMP and thermal fatigue is managed via a TLAA. As SCC is not applicable to carbon steel pressure boundary components, these carbon steel components are managed via a TLAA only.

4. New subsections have been added to Chapters A.13, B.13, C.13, and D.13 of the SAR to summarize the TLAAs that were identified in the renewal submittals for the various DSCs and horizontal storage module (HSM) (i.e., References [1], [2], and [3]) to manage selected aging effects. Of these TLAAs, only the fatigue evaluations required revising to account for the proposed actions and loadings associated with the transportation of the existing DSCs currently in service to the WCS CISF. A single evaluation was performed that bounds all the DSCs to be transported to and stored at the WCS CISF. This revised fatigue evaluation is summarized in the new SAR Section C.13.2.

5. Tables NP-15-13-1 through Table NP-15-13-4 have been revised (see below) to include the results of the aging management review (AMR) that had been performed for the subcomponents from the previous renewal submittals. The tables originally listed the results of the AMR for the CoC 1004 subcomponents. Revised Table NP-15-13-1 (for the FO, FC, FF DSCs) and Table NP-15-13-2 (for the GTCC DSC) include AMR results from the Sacramento Municipal Utility District (SMUD) Rancho SECO Independent Spent Fuel Storage Installation (ISFSI) License (SNM 2510) Renewal Application [1]. Revised Table NP-15-13-3 (for the 24PT1 DSC) and Table NP-15-13-4 (for the AHSM) included the AMR results from the CoC No. 1029 renewal submittal [2]. These revised tables include the intended functions, operating environments and aging effects that require management for the FO, FC, FF, GTCC, and 24PT1 DSCs and the AHSM ITS subcomponents where the corresponding CoC 1004 subcomponents are classified as NITS. The last column in the tables has also been revised to use the identified aging effects as the basis for determining applicability of the CoC 1004 AMP to the FO, FC, FF, GTCC, and 24PT1 DSCs and the AHSM.

6. Tables NP-15-13-1 through NP-15-13-4 have been revised to include footnotes explaining why the not important-to-safety (NITS) items for FO, FC, FF, GTCC, and 24PT1 DSCs and the AHSM do not screen in under Scoping Criterion #2. These explanations come directly from scoping evaluations performed in References [1] and [2].

7. Tables NP-15-13-1 through NP- 15-13-4 have been revised to correct the materials used for the various subcomponents. Note that the CoC 1004 material for the outer bottom cover plate for the 24PT2S and 24PT2L DSCs in Table NP-15-13-2 was correctly listed as A240 Type 304.

8. Tables NP-15-13-1 through NP-15-13-4 have been revised to include the intended function for each subcomponent. These are the intended functions listed in the respective renewal submittals for the various DSCs and the AHSM (i.e., References [1] and [2], and the listed CoC 1004 DSC and AMR Results Tables from Reference [3]) subcomponents.

9. After reconsidering the level of operating experience needed to provide a basis for the use of a surrogate inspection, and the likelihood that such experience will not be available in the immediate future, ISP has revised the AMPs to remove the option to use the inspection results from other general or specific licensee inspections to manage the DSC aging effects.

**References:**

1. Letter from Dan Tallman (SMUD) to Wendy A. Reed (NRC), DPG 19-087, “Response to Request for Clarification of Response to Additional Information for the Technical Review of the Application for Renewal of the Rancho Seco Independent Spent Fuel Storage Installation License No. SNM-2510 (CAC/EPID NOS. 001028/L-2018-RNW-0005; 000993/L-2018-LNE-0004),” dated July 12, 2019.

2. Letter from Prakash Narayanan (TN Americas LLC) to NRC Document Control Desk, E-55203, “Response to Request for Supplemental Information for the Technical Review of the Application for Certificate of Compliance No. 1029 (Docket No. 72-1029, CAC/EPID Nos. 001028/L-2019-RNW-0014),” dated December 4, 2019.

3. Letter E-46190 from Jayant Bondre (AREVA Inc.) to Document Control Desk (NRC), C.1329 "Response to Re-Issue of Second Request for Additional Information – AREVA Inc. Renewal application for Standardized NUHOMS® System – CoC 1004 (Docket No. 72-1004, CAC No. L24964)," September 29, 2016, (ADAMS Accession Number ML16279A367).

**Impact:**

SAR Chapters A.13, B.13, C.13, and D.13 have been revised as described in the response.

Proprietary Information on Pages 32 to 50
Withheld Pursuant to 10 CFR 2.390.

**SAR Appendix E, "NAC-MPC"**

**RAI NP-E-1:**

Revise the discussion in WCS CISF SAR Section E.3.1.1.3, "Seismic Design," on the seismic response of the NAC-MPC to recognize that the storage pad peak earthquake motions are based on the WCS CISF SAR Section 7.6.3 SSI analysis. On the basis of the SSI analysis results, which show markedly higher accelerations at cask center of gravity than those seismic motions used in the quasi-static analysis to demonstrate cask seismic stability, revise the Section E.3.1.1.3 discussion on the seismic response of the NAC-MPC at the proposed WCS CISF site.

SAR Section E.3.1.1.3 notes that Section 11.2.2 of the NAC-MPC FSAR demonstrates cask seismic stability for the peak pad seismic motion of 0.25 g horizontal and 0.167 g vertical in a quasi-static analysis. These seismic motions are seen markedly lower than those calculated at the cask center of gravity in the site-specific SSI analysis in Section 7.6.3. Section 7.6.3 also notes that cask sliding is likely to occur. Thus, the cask seismic performance discussion should be based on the storage pad seismic motions evaluated in SAR Section 7.6.3 for the WCS CISF site. [Note: This request applies also to Section E.3.2.1.3 for the MPC-LACBWR storage system.]

This information is needed to determine compliance with 10 CFR 72.24(c), 72.24(d)(1) and (2), and 72.122(b)(2)(i).

**Response to RAI NP-E-1:**

WCS CISF Section E.3.1.1.3, "Seismic Design" has been revised to point to the site-specific seismic evaluation provided in Section 7.6.3 of the WCS CISF SAR. Section 7.6.3 demonstrates that the NAC-MPC and NAC-LACBWR systems are bounded by the MAGNASTOR system for sliding and tip-over. The MAGNASTOR system does not tip-over in the design basis seismic event for the WCS CSIF site and experiences minimal sliding (Maximum 1.20 inches).

**Impact:**

SAR Sections E.3.1.1.3 and E.3.2.1.3 have been revised as described in the response.

**SAR Appendix F, “NAC-UMS”**

**RAI NP-F-1:**

Revise the NAC-UMS Seismic Ground Motion Design Criteria listed in WCS CISF SAR Table F.3.1,”Summary of WCS CISF Principal Design Criteria, which states, “[T]he maximum allowable ground acceleration for the NAC-UMS system is 0.26 g horizontal and 0.29 g vertical.”

The staff notes that Section 11.2.8 of the NAC-UMS FSAR defines the design basis peak pad seismic motions at 0.26 g and 0.29 g for two orthogonal horizontal components and 2/3 of the horizontal resultant for the vertical.

This information is needed to determine compliance with 10 CFR 72.24(c), 72.24(d)(1) and (2), and 72.122(b)(1).

**Response to RAI NP-F-1:**

As discussed in the responses to RAIs NP-E-1, NP-F-2, and NP-G-1 Sections E.3.1.1.3, E.3.2.1.3, F.3.1.1.3, and G.3.1.1.3 have been revised to point to the site-specific seismic evaluation provided in Section 7.6.3 of the WCS CISF SAR. Tables E.3-1, F.3-1, and G.3-1 have also been updated to point to the site-specific seismic evaluation provided in Section 7.6.3 of the WCS CISF SAR.

**Impact:**

SAR Tables E.3-1, F.3-1, and G.3-1 have been revised as described in the response.

**RAI NP-F-2:**

Revise the discussion in WCS CISF SAR Section F.3.1.1.3, “Seismic Design,” on the seismic stability of the NAC-UMS to recognize that the storage pad peak earthquake motions are based on the WCS CISF SAR Section 7.6.3 SSI analysis. On the basis of the SSI analysis results, which show markedly higher accelerations at cask center of gravity than those seismic motions used in the quasi-static analysis to demonstrate cask seismic stability, revise the last two sentences of Section F.3.1.1.3, which state:

> “The existing analysis bounds the WCS CISF site pad design limits for accelerations at the top pad surface. Therefore, no further evaluations are required.”

SAR Section F.3.1.1.3 notes that Section 11.2.8 of the NAC-UMS FSAR demonstrates cask seismic stability for the peak pad seismic motions of 0.25 g and 0.29 g horizontal components and 2/3 of the horizontal resultant for the vertical in a quasi-static analysis. These seismic storage pad motions are less severe than the ones resulting from the SSI analysis in SAR Section 7.6.3 for the WCS CISF site. Section 7.6.3 also notes that cask sliding is likely to occur. Thus, the cask seismic performance discussion needs to be revised based on the storage pad seismic motions evaluated in SAR Section 7.6.3 for the WCS CISF site.

This information is needed to determine compliance with 10 CFR 72.24(c), 72.24(d)(1) and (2), and 72.122(b)(1).

**Response to RAI NP-F-2:**

Section F.3.1.1.3, “Seismic Design” has been revised to point to the site-specific seismic evaluation provided in Section 7.6.3 of the WCS CISF SAR. Section 7.6.3 demonstrates that the NAC-UMS system is bounded by the MAGNASTOR system for sliding and tip-over. The MAGNASTOR system does not tip-over in the design basis seismic event for the WCS CSIF site and experiences minimal sliding (Maximum 1.20 inches).

**Impact:**

SAR Section F.3.1.1.3 has been revised as described in the response.

**SAR Appendix G, "NAC-MAGNASTOR"**

**RAI NP-G-1:**

Revise the discussion in WCS CISF SAR Section G.3.1.1.3, "Seismic Design," on the seismic stability of the MAGNASTOR to recognize that the storage pad peak earthquake motions are based on the SSI analysis of SAR Section 7.6.3. On the basis of the SSI analysis results, which show markedly higher accelerations at cask center of gravity than those seismic motions used in the quasi-static analysis to demonstrate cask seismic stability, revise the last two sentences of Section G.3.1.1.3, which state:

> "The existing analysis bounds the WCS CISF site pad design limits for accelerations at the top pad surface. Therefore, no further evaluations are required."

SAR Section G.3.1.1.3 notes that Section 11.2.8 of the MAGNASTOR FSAR demonstrates that the cask is stable during a 0.37 g horizontal storage pad motion. The vertical acceleration for this evaluation is defined as 2/3 of the horizontal motion. These storage pad accelerations are less severe than the ones resulting from the SSI analysis in SAR Section 7.6.3 for the WCS CISF site. Section 7.6.3 also notes that cask sliding is likely to occur. Thus, the cask seismic performance discussion needs to be revised based on the storage pad seismic motions evaluated in SAR Section 7.6.3 for the WCS CISF site.

This information is needed to determine compliance with 10 CFR 72.24(c), 72.24(d)(1) and (2), and 72.122(b)(1).

**Response to RAI NP-G-1:**

Section G.3.1.1.3, "Seismic Design" has been revised to point to the site-specific seismic evaluation provided in Section 7.6.3 of the WCS CISF SAR. Section 7.6.3 demonstrates that the MAGNASTOR system does not tip-over in the design basis seismic event for the WCS CSIF site and experiences minimal sliding (Maximum 1.20 inches).

**Impact:**

SAR Section G.3.1.1.3 has been revised as described in the response.

**SAR Chapter 2, "Site Characteristics"**

**RAI P-2.6-3:**

Provide the following information regarding the WCS CISF settlement evaluation and associated material properties:

a. Consistent with proprietary RAI 2.6-1 and non-proprietary RAI 2.6-5, provide the stratigraphic information by depth and associated material properties, including the static elastic modulus values, for the stratum depth causing settlement, and justify the basis of the material properties assigned.

b. Provide a settlement evaluation for consolidation and secondary compression.

The material composition and thicknesses of the subsurface layers at the site vary throughout WCS CISF SAR as described in proprietary RAI 2.6-1. In addition, as described in the non-proprietary RAI 2.6-5, the values calculated for the static elastic moduli which are provided in Appendix D of the Geotechnical Exploration Report (Attachment E to SAR) exceeds the typical range of values for similar soils reported by various engineering literatures. For example, Bowles (1996) Table 2-8 presents that a typical range value for a silty sand is 725 psi to 2,900psi, for a dense sand 7,251 psi to 11,748 psi, for dense sand and gravel 14,503 psi to 29,007 psi and for hard clay 7,251 psi to 14,503 psi. However, Table 9 of Attachment E to the SAR the applicant provided static elastic modulus values from 9,796 psi to 95,255 psi. The stability of a site is safety significant and the engineering evaluations highly depends on the subsurface material properties. SAR Figure 7-30 presents the soil characterization by depth and estimated the depth to the red beds (clay) of 40 ft to 85 ft. The NRC staff reviewed the settlement evaluation to ensure the stability of the subsurface materials but did not find a discussion concerning the evaluation of consolidation and secondary compression settlement which is a typical geotechnical behavior for clay material.

**Reference:**

Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York.

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI P-2.6-3:**

**Response to Part a:**

Four of the eighteen borings performed for the CISF project encountered auger refusal. The auger refusal depths ranged from 37 to 45 feet below the ground surface (bgs). Often one of the borings would be extended to a greater depth in order to obtain the soil parameters necessary for settlement analysis. In this case, shear wave surveys were performed in conjunction with the geotechnical exploration and shear wave velocities are provided to depths of 100 feet bgs. Additionally, multiple previous geotechnical investigations, as well as shear wave testing, have been performed at the site. The historical data outlined below were utilized to extend the soil profile and engineering parameters to a depth of 600 feet. This depth satisfies general industry guidance for settlement evaluation depth. The depth of 600 feet was selected as the termination depth due to encountering the Trujillo Sandstone Layer.

The sections below reference the previous studies that were performed along with the methodology for obtaining the necessary soil parameters to perform the settlement analyses.

Methodology:

The information from the eighteen borings and shear wave data included in the Report of Geotechnical Exploration (Attachment E to Chapter 2 of the SAR) was supplemented with data obtained from References [2], [3], and [4]. This data was used to produce a soil stratigraphic column to 600 feet along with the necessary engineering parameters required for settlement analysis. Figure NP-2.6-3-2 displays the locations of the historical borings provided.

Stratigraphy Development:

- The upper stratigraphy (to a depth of 45 feet) was based solely on the results of the eighteen soil test borings

- From a depth of 45 to 100 feet bgs the stratigraphy was based on the Geologic Column of the CISF Area (Figure 7-30 of the SAR).

- From 100 feet to 600 feet bgs, the Geologic Column of the CISF Area (Figure 7-30 of the SAR), WCS (2007) Plate 2-2, and deeper historical borings were utilized to generate the stratigraphy.

The resulting stratigraphy as utilized for settlement analysis at the site is provided in Table P-2.6-3.

**Table P-2.6-3-1**
**Stratigraphy for Settlement Analysis**

| Top (feet) | Bottom (feet) | Layer Description |
|---|---|---|
| 0 | 2 | Cover Sands |
| 2 | 10 | Caliche with Sand Matrix - Moderately Hard |
| 10 | 20 | Caliche with Sand Matrix - Moderately Hard |
| 20 | 25 | Caliche - Very Hard |
| 25 | 35 | Caliche - Very Hard |
| 35 | 50 | Ogallala - Sand with Gravel |
| 50 | 80 | Ogallala - Sand with Gravel |
| 80 | 100 | Ogallala - Sand with Gravel |
| 100 | 130 | Dockum - Claystone and Siltstone |
| 130 | 230 | Claystone and Siltstone |
| 230 | 275 | Dockum - Claystone |
| 275 | 300 | Dockum - Silty Sands |
| 300 | 360 | Dockum - Claystone |
| 360 | 600 | Dockum - Claystone |

Soil Parameter Selection:

The settlement analysis that was utilized required the development of constrained modulus (elastic modulus) values. The constrained modulus values were calculated as follows:

Constrained Modulus up to 20 Feet BGS:

To a depth of 20 feet bgs the constrained modulus was correlated to the standard penetration test (SPT) N-values obtained in the borings. The SPT N-Values were correlated to constrained modulus using the method outlined in Reference [1]. This methodology allows correlation of constrained modulus to N-value for N-values up to 70 blows per foot. The graphical representation is shown in Figure P-2.6-3-1 (Figure 5.4 of Reference [1]).

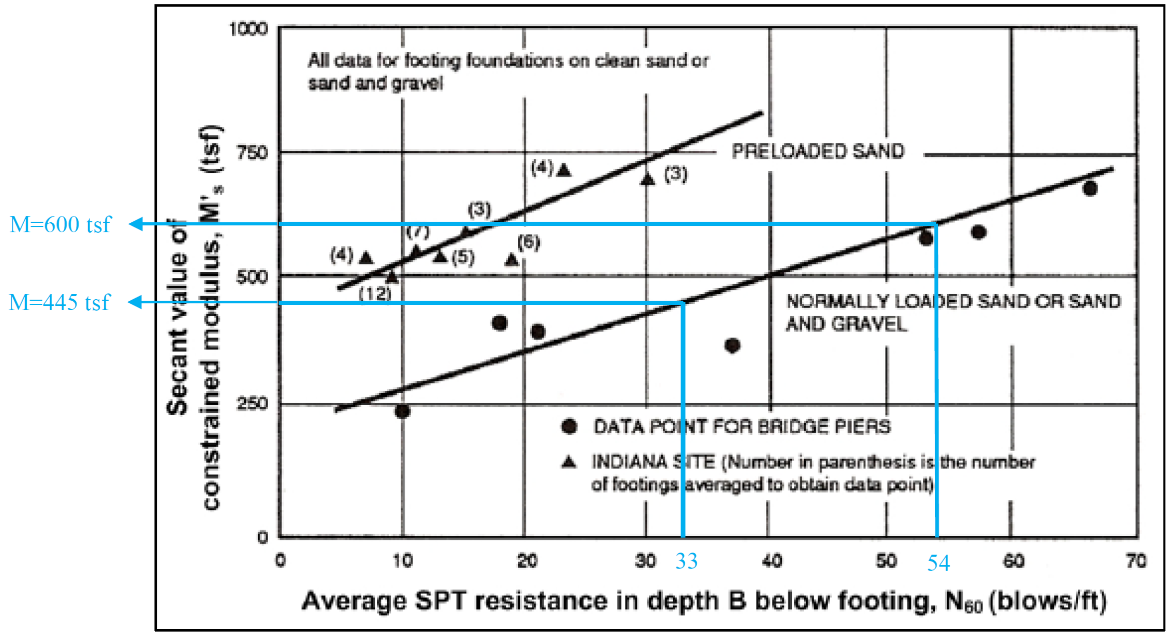



**Figure P-2.6-3-1**
**Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]**

<u>Constrained Modulus over 20 Feet BGS:</u>

The borings performed for the WCS CISF site were only advanced to maximum depths of 45 feet. Additionally, the methodology outlined in Reference [1] is only valid up to N-values of 70 blows per foot. Based on the N-values obtained this methodology could only be extended to a depth of 20 feet below ground surface. Therefore, a second methodology had to be utilized to generate the constrained modulus from depths of 20 feet to 600 feet.

To supplement the information obtained in preparation of the Report of Geotechnical Exploration, the Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, (Attachment D to Chapter 2 of the SAR) was used. This document provided shear wave velocity profiles at the site to depths of approximately 1,200 feet.

The shear wave velocities were converted to constrained modulus using the following relationship:

$$V_S \xrightarrow{G=V_S^2*\rho} G \xrightarrow{M=\frac{2G(1-\nu)}{(1-2\nu)}} M$$

Where, $V_s$ = shear wave velocity
G = shear modulus
M = constrained modulus
ν = Poisson's ratio
ρ = unit weight

- From 20 feet to 100 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in the Report of Geotechnical Exploration to constrained modulus using the unit weight and Poisson's ratio.
- From 100 feet to 600 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in AECOM (2016) to constrained modulus using the unit weight and Poisson's ratio. The unit weight and Poisson's ratio values were also obtained from Appendix A of the AECOM (2016) report.

<u>Results:</u>

The methodology described above resulted in the Table P-2.6-3-2 soil column. This column will replace Appendix D in the revised Report of Geotechnical Exploration.

**Table P-2.6-3-2**
**WCS CISF Soil Column**

| Top (feet) | Bottom (feet) | N-Value (bpf) | Average Shear Wave Velocity (ft/s) | Layer Description | Constrained Modulus (ksf) |
|---|---|---|---|---|---|
| 0 | 2 | 33 | | Cover Sands | 890 |
| 2 | 10 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 10 | 20 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 20 | 25 | | 1,530 | Caliche - Very Hard | 3,5815 |
| 25 | 35 | | 1,900 | Caliche - Very Hard | 55,232 |
| 35 | 50 | | 2,290 | Ogallala - Sand with Gravel | 80,233 |
| 50 | 80 | | 1,840 | Ogallala - Sand with Gravel | 5,3870 |
| 80 | 100 | | 2,790 | Ogallala - Sand with Gravel | 12,3857 |
| 100 | 130 | | 2,300 | Dockum - Claystone and Siltstone | 84,172 |
| 130 | 230 | | 2,755 | Claystone and Siltstone | 120,769 |
| 230 | 275 | | 2,755 | Dockum - Claystone | 120,769 |
| 275 | 300 | | 2,755 | Dockum - Silty Sands | 120,679 |
| 300 | 360 | | 2,755 | Dockum - Claystone | 120,679 |
| 360 | 600 | | 3,115 | Dockum - Claystone | 154,394 |

As shown above, the historical data available at the site coupled with the eighteen borings and new shear wave study has allowed the development of a stratigraphic column without additional new soil borings to greater depths.

Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR has been updated to include the above information.

**Response to Part b:**

The soil column and subsequent constrained modulus values shown above were utilized in the settlement analysis for the foundations.

The settlement analysis (consolidation) is provided in Appendix H of the Revised Report of Geotechnical Exploration dated February 21, 2020 (Attachment E to Chapter 2 of the WCS CISF SAR). The settlement analysis includes a series of four pad loading conditions and explores the effects of adjacent pads on total and differential settlement.

Please note that clay was not encountered in the borings at the WCS CISF site. The clay/claystone was referenced as part of the Dockum Group at depths ranging from 230 to 275 feet and 300 to 600 feet. Based on the constrained modulus values obtained from the shear wave velocities, these layers exhibited constrained modulus values ranging from 120,679 to 154,394 ksf (5,778 to 7,392 MPa). The modulus values obtained from the shear waves are significantly higher than the published values in Bowles for selected soils, and are characteristic of very hard rock. As such secondary compression was not included as part of the analysis.

The responses to RAIs NP-2.6-3, NP-2.6-4, NP-2.6-5, P-2.6-3, P-2.6-5 and P-2.6-6 all address the Report of Geotechnical Exploration. All of the required changes to this report (SAR Attachment E to Chapter 2) from the RAIs, are included as part of the response to RAI NP-2.6-3.

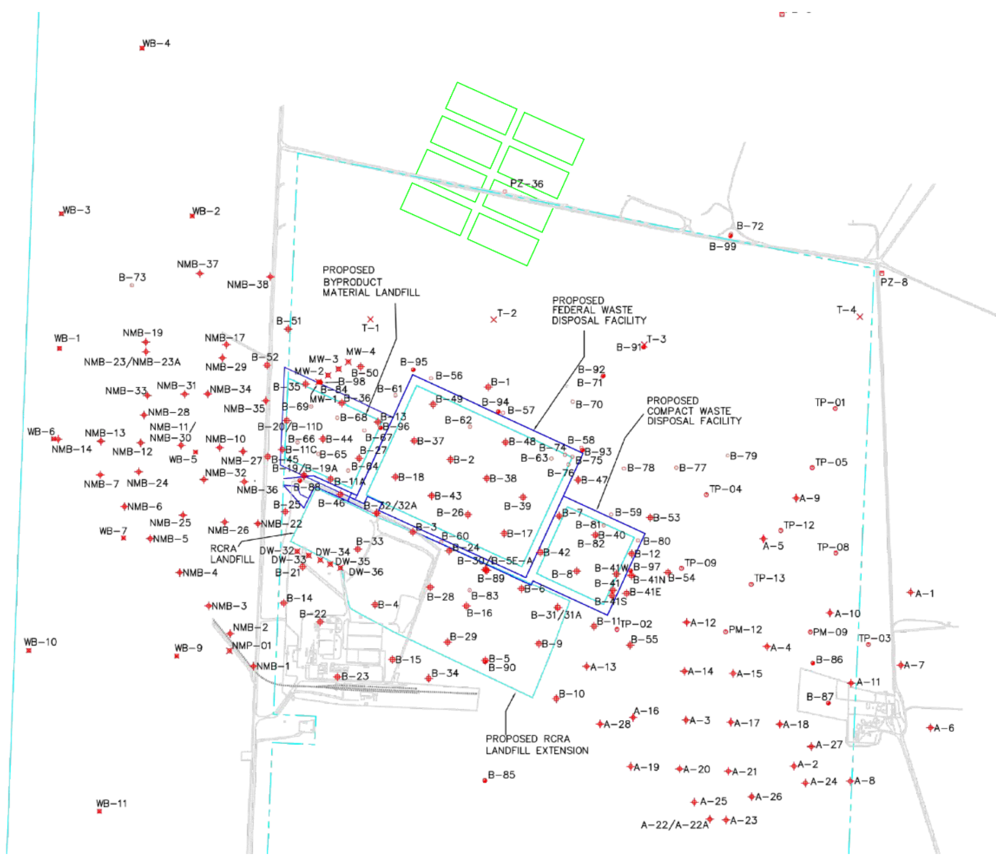



**Figure P-2.6-3-2**
**Historical Borings at WCS Site**

Additional Information and Clarification

1. Revision 2 of the Report of Geotechnical Exploration provides bearing capacity calculations for both the cask handling building and the CISF pads. The bearing capacity calculations are provided in Appendix G. The Vesic Bearing capacity formula was selected because it provides more accurate bearing values and it applies to a much broader range of loading and geometry conditions. The calculations utilized the Vesic Bearing Capacity Formula shown below.

$$q_{ult} = c'N_c s_c d_c i_c b_c g_c + \sigma'_{ZD} N_q s_q d_q i_q b_q g_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

In this equation the terms $s_c$, $s_q$, and $s_\gamma$ are shape factors and defined as follows:

$$s_c = 1 + (B/L)(N_q/N_c)$$

$$s_q = 1 + (B/L)\tan\Phi'$$

$$s_\gamma = 1 - 0.4(B/L)$$

As shown in the supplied calculations in Appendix G, shape factors were utilized in the analysis.

2. In Revision 0 of the Report of Geotechnical Exploration, there is a comment in Section 3.2.1 that states the N-values may be inflated due to the amount of caliche encountered during the test interval. ISP was asked to expound on this note and discuss if this was considered when utilizing the N-values for calculation of constrained modulus and to comment on how the methodology impacted the conservativeness of the settlement calculation.

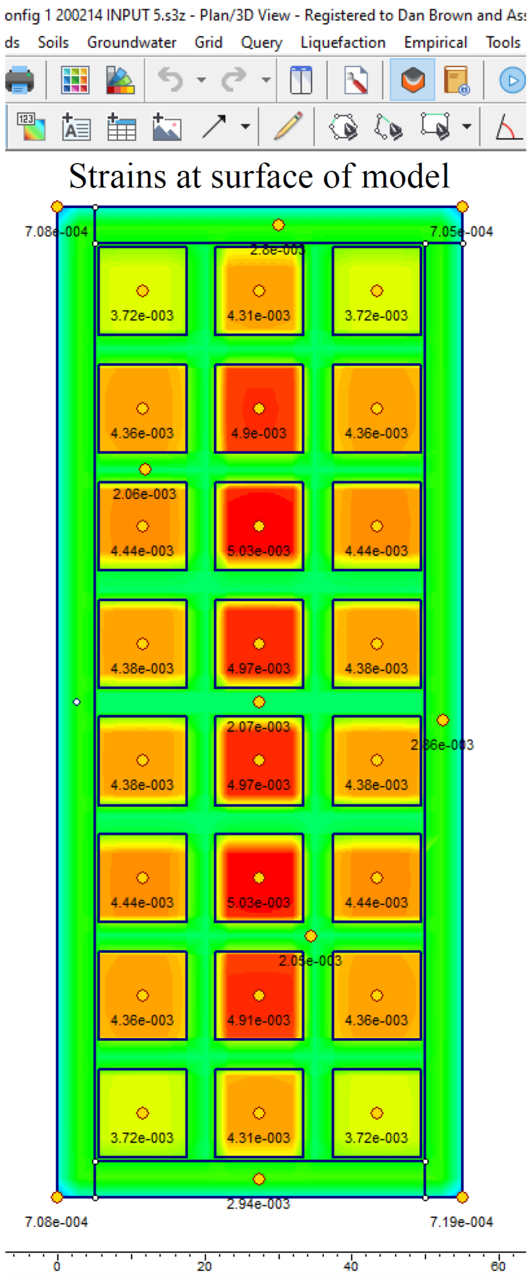
   The use of the word “inflated” creates ambiguity in the discussion of the presence of caliche with the samples; therefore, this statement has been removed in Revision 1 and 2 of the report. Essentially, the N-values obtained are representative of the materials encountered at the exact boring locations, and the amount of caliche in the borings has the potential to influence the N-values. ISP did not intend for it to appear that these N-values were not indicative of the subsurface materials at the site.

   Though it is considered that a reduction in these N-values is not necessary, additional measures were incorporated within the process that lead to a conservative calculation of settlement. These measures included ignoring the refusal blow count measurements when calculating the average N-value, and utilizing a large strain modulus correlation in the zone below the footing that will encounter the largest strains. This process is outlined in more detail in the following paragraph.

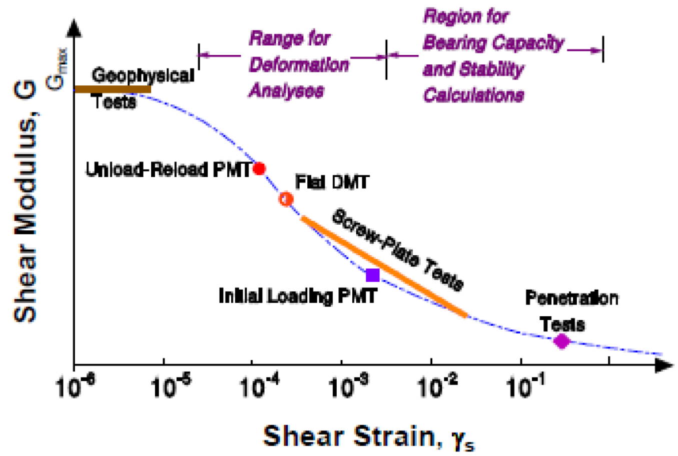
As discussed in Part a of this RAI response, to a depth of 20 feet bgs, the constrained modulus was calculated using the SPT N-values obtained in the borings. The SPT N-values were correlated to constrained modulus utilizing the method outlined in the "Engineering Manual for Shallow Foundations" [1]. The SPT correlated modulus was utilized for soil and intermediate geomaterial. It is important to note that the blow counts in the intermediate geomaterial included numerous refusal blow counts. Average blow counts in these materials were reduced by ignoring refusal blow count measurements in the top 20 feet, which had the effect of reducing the average blow count and the associated modulus to bound the settlement calculation. Additional conservatism is included considering the strain range of SPT measurements (shown in the "Evaluating axial drilled shaft response by seismic cone" chart [6]. See Figure P-2.3-3-3) is higher than the typical strain range of footing analysis. Namely, the modulus is reduced to a large strain modulus correlation in the zone below the footing that will encounter the largest strains in an effort to avoid overreliance on the cementation of the caliche materials.

3. As discussed in Part a of this RAI response, from a depth of 20 feet to a depth of 600 feet, the constrained modulus was obtained by converting the available shear wave velocities using Poisson's ratio and unit weight. Wave-based geophysical tests are performed in what is typically classified as the small strain range (1E-5 to 1E-6). The shear wave velocity from wave-based geophysical testing can therefore be used to calculate the small-strain modulus, $G_{max}$. Modulus reduction curves are often utilized in typical soil conditions (clays and saturated sands) to account for the change in modulus relative to strain as the strain range of typical geotechnical engineering problems exceeds the strain range of the measurement used to calculate the modulus. However, this approach is not one-size-fits-all. For the purposes of this analysis, ISP did not use modulus reduction curves for the reasons described below.

As shown in Figure P-2.6-3-3, the model strains are provided from Configuration 1 (fully loaded) of the Consolidated Interim Storage Facility (CISF) pads. This loading condition results in the largest magnitude strains at the center of the footing. As shown in the inset included in Figure P-2.6-3-3, the largest strains fall within the range of expected strain from Mayne and Schneider [6].




Mayne and Schneider (2001)




Results of analysis in strain range for deformation analysis from Burland (1989) and Mayne and Schneider (2001)

1

**Figure P-2.6-3-3**
**Model Strains from Configuration 1 (Fully Loaded) for the CISF Pads**

For clarity, the maximum strain at the midpoint of each layer is provided in Table P-2.6-3-3.

**Table P-2.6-3-3**
**Maximum Strains by Layer**

**Strain with Depth from highest loaded case**

Config 1 200214 input 5 File

| | Layer | Thickness | Depth of Layer Bottom | Midpoint Depth | Max Strain @ midpoint |
|---|---|---|---|---|---|
| SPT Blowcounts | Cover Sands | 2 | 2 | 1 | 4.85E-03 |
| | Caliche Sand 1 | 8 | 10 | 6 | 2.90E-03 |
| | Caliche Sand 2 | 10 | 20 | 15 | 2.25E-03 |
| Shear Wave Velocity | Caliche Hard 1 | 5 | 25 | 22.5 | 6.49E-05 |
| | Caliche Hard 2 | 10 | 35 | 30 | 3.71E-05 |
| | Ogallala 1 | 15 | 50 | 42.5 | 2.09E-05 |
| | Ogallala 2 | 30 | 80 | 65 | 2.24E-05 |
| | Ogallala 3 | 20 | 100 | 90 | 6.96E-06 |
| | Dockum Claystone/Silstone | 30 | 130 | 115 | 7.59E-06 |
| | Claystone and Siltstone | 100 | 230 | 180 | 2.77E-06 |
| | Dockum Clay/Claystone 1 | 45 | 275 | 252.5 | 1.57E-06 |
| | Dockum Sily/Sands | 25 | 300 | 287.5 | 1.24E-06 |
| | Dockum Clay/Claystone 2 | 60 | 360 | 330 | 9.71E-07 |
| | Dockum Clay/Claystone 3 | 240 | 600 | 480 | 3.79E-07 |

As shown in Table P-2.6-3-3, the largest model strains are in the top 20 feet before a sharp contrast at the transition from soil/intermediate geomaterial behavior and rock material behavior. Soils/Intermediate geomaterials are defined as materials that are able to be sampled with SPT. As previously mentioned, the SPT correlated modulus was utilized for soil and intermediate geomaterial. It is important to note that the blow counts in the intermediate geomaterial included numerous refusal blow counts. Average blow counts in these materials were reduced by ignoring refusal blow count measurements in the top 20 feet, which had the effect of reducing the average blow count and the associated modulus to bound the settlement calculation. Additional conservatism is included considering the strain range of SPT measurements (shown in the Mayne and Schneiderchart [6]) is higher than the typical strain range of footing analysis. That is to say that the modulus is reduced to a large strain modulus correlation in the zone below the footing that will see the largest strains in an effort to avoid overreliance on the cementation of the caliche materials.

“Rock” was defined as material with refusal blow counts that could not be used to estimate modulus. Below twenty feet, the constrained modulus was obtained by converting the available shear wave velocities using Poisson’s ratio and unit weight. Modulus reduction was not applied to these materials because no site-specific modulus reduction curves are available. This is not a shortcoming of the exploration, because materials this dense/hard are not expected to strain beyond the range of $G_{max}$ at this depth under these loads. ISP contractor, Dan Brown and Associates, has experience with modulus reduction curves in soft rock materials (see Figure P-2.6-3-4) showing our basis for applicability of $G_{max}$ for settlement problems in these materials.

Small strain $G_{max}$ can be converted to G using modulus reduction curves in soft rock at Oak Ridge National Laboratory under the direction of Tim Siegel.

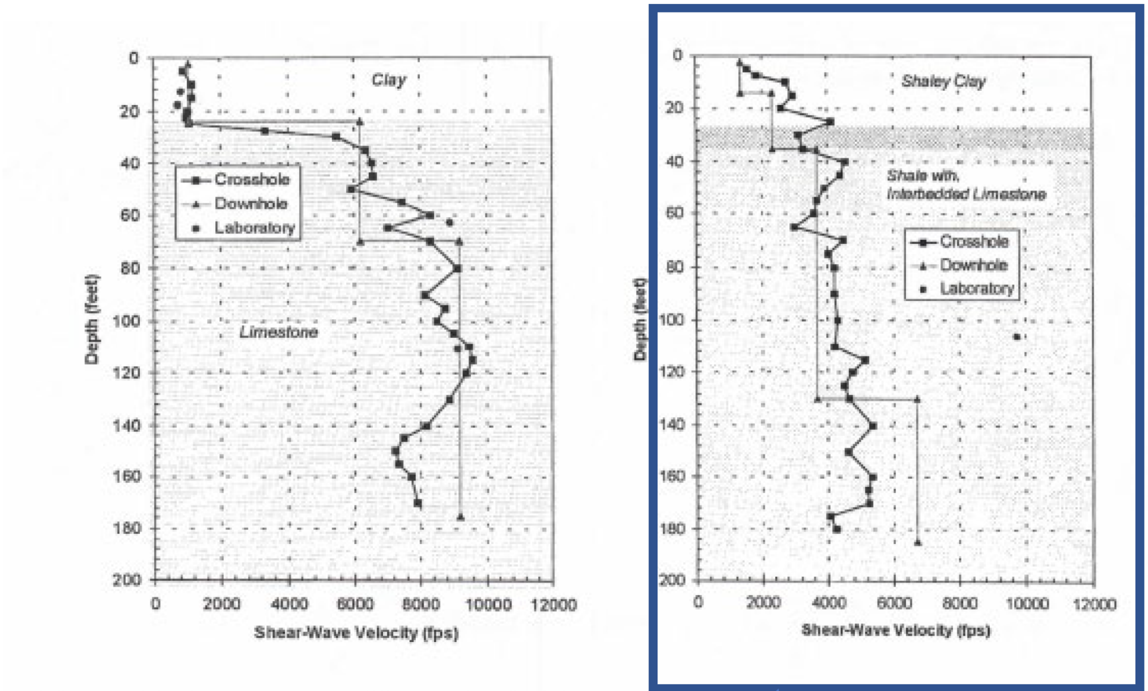


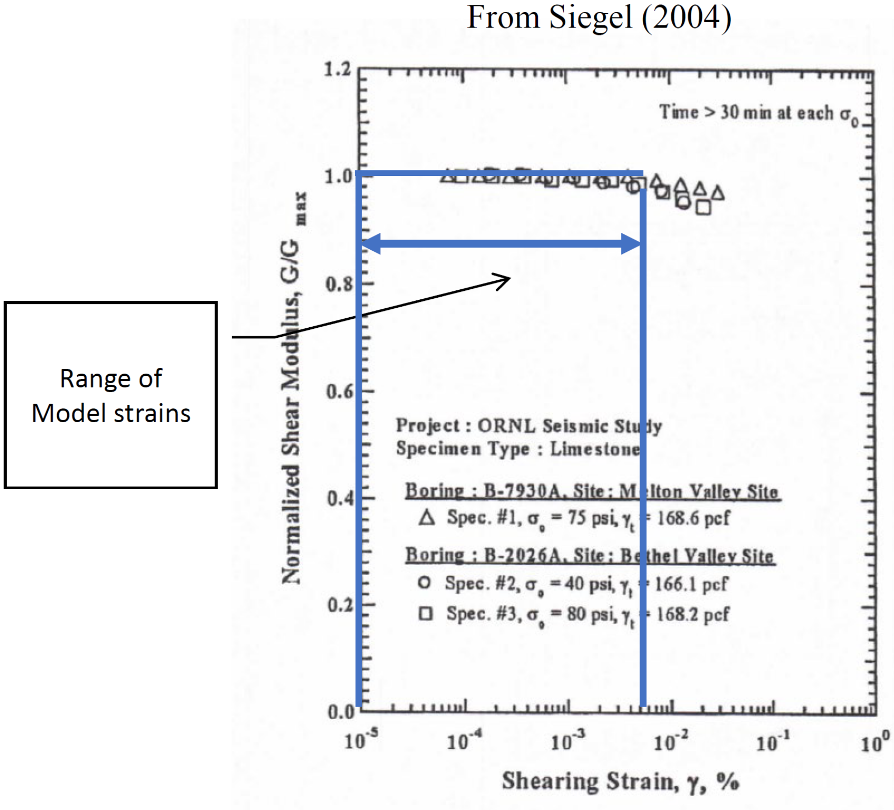

**Figure 4: Shear-wave velocity profiles for East Tennessee. (L) Clay overburden above limestone. (R) Weathered shale**

Data presented above collected at Oak Ridge National Laboratory. Soft Rock. Velocity magnitudes most similar to this site.

**Figure P-2.6-3-4**
**Modulus Reduction Curves In Soft Rock Materials from Siegel (2004) [8]**

In the model for the CISF site low strain values are predicted even in zones with large strain modulus assigned. Mayne and Schneider [6] and Burland [7] reports the range of strain for deformation analysis to be approximately 0.0003% to 0.003%. Analysis from the current project exhibits strains from 0.5E-03 to 0.1E-07. As shown in Figure P-2.6-3-5, taken from Siegel [8], at this range of strain, there is essentially no reduction for soft rock materials in this strain range.

From Siegel (2004)




Essentially no reduction for these materials in these strain ranges.

From Oak Ridge National Laboratory investigation by Siegel (2004). Sedimentary rock and residuum. Shear Wave Velocity profile similar to this site.

**Figure P-2.6-3-5**
**Normalized Shear Modulus vs. Shearing Strain from Siegel (2004) [8]**

**References:**


1. Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M., “Engineering Manual for Shallow Foundations,” prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association of State Highway and Transportation Officials and Federal Highway Administration, Washington, D.C., Blacksburg, VA, 1991, 171 pp.

2. Waste Control Specialists LLC, “Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions,” Attachment D to Chapter 2 of the SAR: AECOM, Centralized Interim Storage Facility Project, March 18, 2016.

3. Cook-Joyce, Inc., “Geology Report,” Revision 12c, Appendix 2.6.1, prepared for Waste Control Specialists, LLC, Austin, Texas, May 1, 2007.

4. Waste Control Specialists LLC, “Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste,” WCS CISF SAR Chapter 2, March 2007.

5. Bowles, Joseph E., “Foundation Analysis and Design,” 5th Edition, McGraw Hill Education, Peoria, Illinois, 1997.

6. Mayne, P.W. and Schneider, J.A. (2001). “Evaluating axial drilled shaft response by seismic cone”. Foundations and Ground Improvement, GSP No. 113, ASCE, Reston, Virginia, 655-669

7. Burland, J.B. (1989). "Small is beautiful: The stiffness of soils at small strains." Canadian Geotechnical Journal 26 (4), 499-516.

8. Siegel, Timothy C. (2004). “Geotechnical Earthquake Engineering for Bridge and Highway Projects in East Tennessee.

**Impact:**

No additional changes as a result of this RAI.

**RAI P-2.6-5:**

Justify why the selected depth of 37 ft is adequate for evaluating settlement at the WCS CISF site.

In the settlement calculation, it is necessary to consider additional stresses due to the foundation load up to the influence depth of the settlement. Subsection 7.6.2.5 of the SAR presents a calculation of elastic settlements based on the theory of elasticity. The staff noted that the stratum depth causing settlement was assumed to be 37 feet below the concrete pad in the calculation. The SAR referenced Bowles (1996) for using a weighted average elastic modulus for elastic settlement evaluation. In the same reference, Bowles also recommends that the depth used for evaluation should be either five times the width of the foundation or the depth where a hard stratum is encountered. A hard layer is defined as ten times the static elastic modulus of the adjacent upper layer. The influence depth is very important to the settlement evaluation as it relates to the stability of subsurface materials which is safety significant.

**Reference:**

Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York.

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI P-2.6-5:**

The original calculation in Section 7.6.2.5 of the SAR utilized the average auger refusal depth encountered in the borings and subtracted the proposed mat thickness to obtain the compressible layer. The assumption made in the initial analysis was that any materials that could not be augered through were incompressible.

Since the Report of Geotechnical Exploration was performed historical data of past geotechnical explorations, well installation logs, and shear wave studies at the property have been provided. Based on a review of the documents listed below, it is deemed that settlement analysis should be extended to greater depths.

The historical data outlined below were utilized to extend the soil profile and engineering parameters to a depth of 600 feet. This depth satisfies general industry guidance for settlement evaluation depth. The depth of 600 feet was selected as the termination depth due to encountering the Trujillo Sandstone Layer.

The sections below reference the previous studies which were performed along with the methodology for obtaining the necessary soil parameters to perform the settlement analyses.

Methodology:

The information from the eighteen borings and shear wave data included in the Report of Geotechnical Exploration (Attachment E to Chapter 2 of the SAR) was supplemented with data obtained from References [2], [3], and [4]. This data was used to produce a soil stratigraphic column to 600 feet along with the necessary engineering parameters required for settlement analysis. Figure P-2.6-5-1 displays the locations of the historical borings provided.

Stratigraphy Development:

- The upper stratigraphy (to a depth of 45 feet) was based solely on the results of the eighteen soil test borings
- From a depth of 45 to 100 feet below ground surface (bgs) the stratigraphy was based on the Geologic Column of the CISF Area (Figure 7-30 of the SAR).
- From 100 feet to 600 feet bgs, the Geologic Column of the CISF Area (Figure 7-30 of the SAR), WCS (2007) Plate 2-2, and deeper historical borings were utilized to generate the stratigraphy.

The resulting stratigraphy as utilized for settlement analysis at the site is provided in Table P-2.6-5-1.

**Table P-2.6-5-1**
**Stratigraphy for Settlement Analysis**

| Top (feet) | Bottom (feet) | Layer Description |
|---|---|---|
| 0 | 2 | Cover Sands |
| 2 | 10 | Caliche with Sand Matrix - Moderately Hard |
| 10 | 20 | Caliche with Sand Matrix - Moderately Hard |
| 20 | 25 | Caliche - Very Hard |
| 25 | 35 | Caliche - Very Hard |
| 35 | 50 | Ogallala - Sand with Gravel |
| 50 | 80 | Ogallala - Sand with Gravel |
| 80 | 100 | Ogallala - Sand with Gravel |
| 100 | 130 | Dockum - Claystone and Siltstone |
| 130 | 230 | Claystone and Siltstone |
| 230 | 275 | Dockum - Claystone |
| 275 | 300 | Dockum - Silty Sands |
| 300 | 360 | Dockum - Claystone |
| 360 | 600 | Dockum - Claystone |

Soil Parameter Selection:

The settlement analysis which was utilized required the development of constrained modulus (elastic modulus) values. The constrained modulus values were calculated as follows:

- To a depth of 20 feet bgs, the constrained modulus was calculated using the Standard Penetration Test (SPT) N-Values obtained in the borings. The SPT N-values were correlated to constrained modulus utilizing the method outlined in Reference [1]. This methodology was only used to a depth of 20 feet as it is only applicable to soils with N-values up to 70 blows per foot.

- From 20 feet to 100 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in the Report of Geotechnical Exploration to constrained modulus using the unit weight and Poisson's ratio.

- From 100 feet to 600 feet bgs, constrained modulus values were obtained from converting the shear wave velocities provided in Reference [2] to constrained modulus using the unit weight and Poisson's ratio. The unit weight and Poisson's ratio values were also obtained from Appendix A of Reference [2].

The resulting soil column is provided in Table P-2.6-5-2.

**Table P-2.6-5-2**
**WCS CISF Soil Column**

| Top (feet) | Bottom (feet) | N-Value (bpf) | Average Shear Wave Velocity (ft/s) | Layer Description | Constrained Modulus (ksf) |
|---|---|---|---|---|---|
| 0 | 2 | 33 | | Cover Sands | 890 |
| 2 | 10 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 10 | 20 | 54 | | Caliche with Sand Matrix - Moderately Hard | 1,200 |
| 20 | 25 | | 1,530 | Caliche - Very Hard | 35,815 |
| 25 | 35 | | 1,900 | Caliche - Very Hard | 55,232 |
| 35 | 50 | | 2,290 | Ogallala - Sand with Gravel | 80,233 |
| 50 | 80 | | 1,840 | Ogallala - Sand with Gravel | 53,870 |
| 80 | 100 | | 2,790 | Ogallala - Sand with Gravel | 123,857 |
| 100 | 130 | | 2,300 | Dockum - Claystone and Siltstone | 84,172 |
| 130 | 230 | | 2,755 | Claystone and Siltstone | 120,769 |
| 230 | 275 | | 2,755 | Dockum - Claystone | 120,769 |
| 275 | 300 | | 2,755 | Dockum - Silty Sands | 120,679 |
| 300 | 360 | | 2,755 | Dockum - Claystone | 120,679 |
| 360 | 600 | | 3,115 | Dockum - Claystone | 154,394 |

As can be seen above, the historical data available at the site, coupled with the eighteen borings and new shear wave study, has allowed the development of a stratigraphic column without additional new soil borings (to greater depths).

The soil column and parameters shown above have been utilized in the additional settlement analyses that resulted from comments within the RAI process. The results of the settlement analyses are provided in Appendix H of the Revised Attachment E (Report of Geotechnical Exploration Consolidated Interim Storage Facility (CISF)) to Chapter 2 of the WCS CISF SAR.

The responses to RAIs NP-2.6-3, NP-2.6-4, NP-2.6-5, P-2.6-3, P-2.6-5 and P-2.6-6 all address the Report of Geotechnical Exploration. All of the required changes to this report (SAR Attachment E to Chapter 2) from the RAIs, are included as part of the response to RAIs NP-2.6-3.

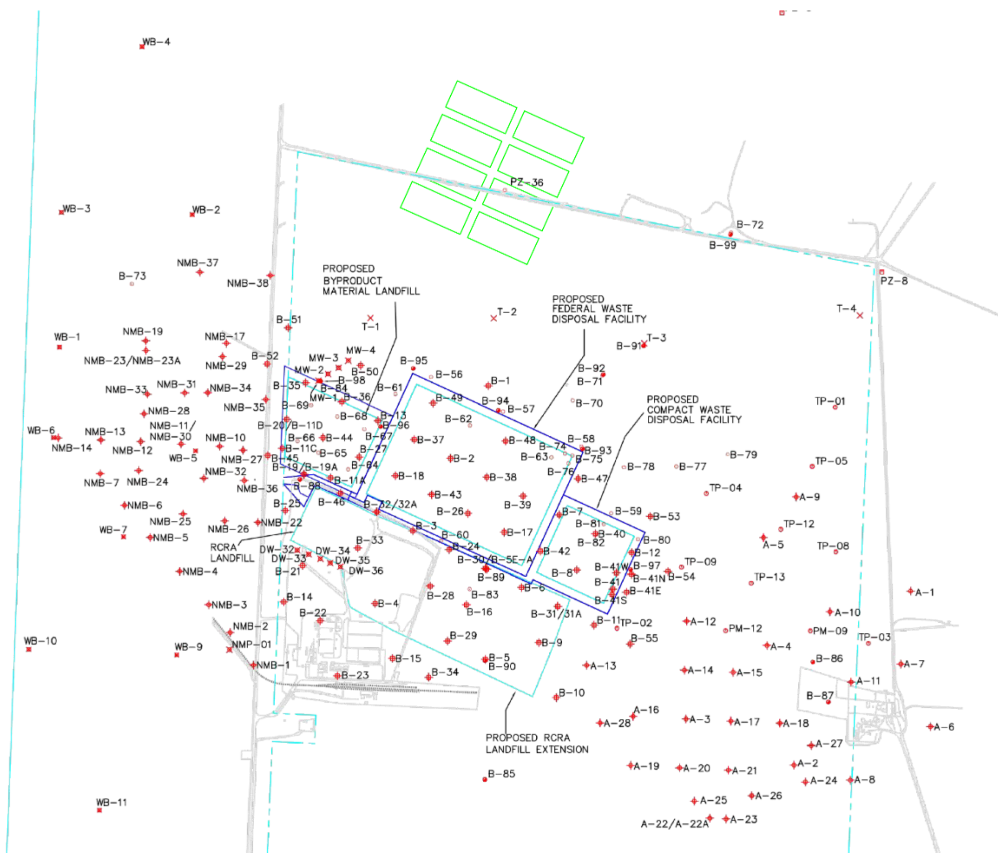



**Figure NP-2.6-3-1**
**Historical Borings at WCS Site**

**References:**

1. Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M., “Engineering Manual for Shallow Foundations,” prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association of State Highway and Transportation Officials and Federal Highway Administration, Washington, D.C., Blacksburg, VA, 1991, 171 pp.

2. Waste Control Specialists LLC, “Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions,” Attachment D to Chapter 2 of the SAR: AECOM, Centralized Interim Storage Facility Project, March 18, 2016.

3. Cook-Joyce, Inc., “Geology Report,” Revision 12c, Appendix 2.6.1, prepared for Waste Control Specialists, LLC, Austin, Texas, May 1, 2007.

4. Waste Control Specialists LLC, “Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste,” WCS CISF SAR Chapter 2, March 2007.

**Impact:**

No additional changes as a result of this RAI.

**RAI P-2.6-6:**

Provide the following information related to the settlement evaluation of the WCS CISF site and justify the selection and determination of the parameters and properties used.

a. The geotechnical engineering basis for how a subgrade modulus of 150 pounds per cubic inch (pci) was determined.

b. The WCS CISF Phase 1 storage pad layout is presented in SAR

Figure 1-6. The proposed storage facility area is about 350 ft x 800 ft. Provide the total and differential settlement evaluation influenced by the layout and construction sequence.

In Subsection 4.3.2 "Mat Foundations (Storage Building)" of Attachment E to the SAR, ISP recommended a subgrade modulus of 150 pounds per cubic inch (pci). This modulus is assigned to the GTSTRUDL model used for the structural analysis that calculated a maximum elastic vertical foundation displacement of 0.125408 inches.

The calculated elastic settlement is significantly less than that estimated by the Federal Highway Administration (FHWA) empirical method based on Standard Penetration test (SPT) N-values because the SPT N-values were likely inflated due to the presence of caliche (in other words, the settlement estimated by FHWA empirical method may not be on conservative side due to likely inflated SPT N-values). It appears that the settlement analyses are only from a single pad, not from multiple adjacent pads as shown in SAR Figure 1-6. Also, the construction sequence of the pads is not considered in total and differential settlement evaluation. The modulus of subgrade is an important safety parameter for the structural analyses. The layout and construction sequence are important for the settlement evaluation performed to assess the stability of the site.

This information is needed to determine compliance with 10 CFR 72.103(f)(1) and 10 CFR 72.103(f)(2)(iv).

**Response to RAI P-2.6-6:**

**Response to Part a:**

The modulus value is the direct relationship of stress/deflection. Since the modulus value is a direct function of the load distribution on the mat (which is unknown until a preliminary modulus is selected), the modulus of subgrade reaction that is traditionally selected to begin with is based on a plate load test. In the Report of Geotechnical Exploration, a preliminary modulus of subgrade reaction of 150 pci was given based on our experience with similar soils (value obtained from literature based on a 1 foot by 1 foot plate load test). The report goes on to state "as with all non-rigid method solutions, the process is iterative and requires the close coordination between the geotechnical engineer and the structural engineer during design. Once a pressure distribution is determined, we can utilize finite element methods to more accurately predict the settlement and provide detailed modulus calculations."

The 150 pci modulus was utilized in a GTSTRUDL model in absence of the specified iterative process described in the Report of Geotechnical Exploration. The use of a single modulus of subgrade reaction ($K_s$) for a mat with a loading of this complexity will not (and did not) generate realistic deflections.

In order to obtain realistic deflections with complex loading, the subgrade can be adjusted to account for wider loads such as the mat foundations present at the WCS CISF site. To address this issue, the geotechnical engineer has worked with the structural engineer to adjust the subgrade modulus through the iterative process.

1. The first iteration of the settlement analysis was performed using mat pressures provided by Enercon.

2. These pressures were used to develop a Settle3D model (finite difference software) with the end goal of formulating values of subgrade modulus (k). The program calculates settlements beneath the mat based on the pressures provided. The modulus values are calculated at distinct points by dividing the pressure/settlement.

3. Values of subgrade modulus were then submitted to Enercon to be integrated into the GTSTRUDL analysis.

4. The next iteration combined the applied loads with an estimate of soil response (calculated k values) thus refining the mat pressure distribution.

5. The results of the refined GTSTRUDL analysis were then provided and used to update the Settle3D model. The result was an updated set of subgrade modulus values for the entire mat.

6. This iterative process was continued until the models converged.

The analysis was performed on a single pad in four different loading configurations: fully loaded, quarter loaded, half loaded, and three quarters loaded. Plots showing the converged models and subsequent subgrade modulus values and anticipated settlements are included in Appendix H of the Revised Report of Geotechnical Exploration Revision 2, which is WCS CSIF SAR Attachment E to Chapter 2.

**Response to Part b:**

After the iterative process outlined above was completed, the resulting stress distributions were utilized in conjunction with SAR Figure 1-6 to perform additional settlement models. By utilizing multiple pads the settlement models encompass stress overlap between pads and the resulting potential for differential settlement depending on the construction/loading sequence of the pads. The resulting Settlement Analysis is included in Appendix H of the Revised Report of Geotechnical Exploration Revision 2, which is WCS CSIF SAR Attachment E to Chapter 2.

The responses to RAIs NP-2.6-3, NP-2.6-4, NP-2.6-5, P-2.6-3, P-2.6-5 and P-2.6-6 all address the Report of Geotechnical Exploration. All of the required changes to this report (SAR Attachment E to Chapter 2) from the RAIs, are included as part of the response to RAIs NP-2.6-3.

**Impact:**

No additional changes as a result of this RAI.