R- 231795

ENERCON	CALCULATION COVER SHEET	Calc. No. NAC004-CALC-02
		Rev. 0
		Page No. 1 of 20

Title:	Liquefaction Potential and Elastic Settlement Evaluation for Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at WCS Site in Andrews, TX	**Client:** NAC International
		Project: NAC-004

Item	Cover Sheet Items	Yes	No
1	Does this calculation contain any open assumptions that require confirmation? (If **YES**, identify the assumptions.)______	☐	☒
2	Does this calculation serve as an "Alternate Calculation"? (If **YES**, identify the design verified calculation.) **Design Verified Calculation No.**______	☐	☒
3	Does this calculation Supersede an existing Calculation? (If **YES**, identify the superseded calculation.) **Superseded Calculation No.**______	☐	☒

Scope of Revision:

N/A

Revision Impact on Results:

N/A

Study Calculation ☐ **Final Calculation** ☒

Safety-Related ☐ **Non-safety-Related*** ☒

**ISFSI pad is important-to-safety with ENERCON QA requirements and design verification invoked.*

(Print Name and Sign)

Originator: Fikret Atalay / *Fikret Atalay*	**Date: 03/28/16**
Design Verifier: Joseph Smierciak / *Joseph M Smierciak*	**Date: 03/28/16**
Approver: Glenn Whritenour /	**Date:** 03.29.16

CALCULATION REVISION STATUS

REVISION	DATE	DESCRIPTION
0	03/29/16	Initial Issue

PAGE REVISION STATUS

PAGE NO.	REVISION	PAGE NO.	REVISION
1-20	0		

APPENDIX REVISION STATUS

APPENDIX NO.	PAGE NO.	REVISION	APPENDIX NO.	PAGE NO.	REVISION

Calculation Design Verification Plan:

Design inputs shall be verified by comparing the documented input with the source references and checking the validity of the references for their intended use. All assumptions shall be evaluated and verified to determine if they are based on sound engineering principles and practices. The methodology, results and conclusions shall be verified.

(Print Name and Sign for Approval - mark "N/A" if not required)

Approver: Glenn Whritenour /	**Date:** 3.29.2016

Calculation Design Verification Summary:

Calculation has been designated a safety related document as noted on the cover sheet.

Calculation has been reviewed for inputs, methodology, math, computations and completeness. The conclusions developed in the calculation satisfy the Purpose and Scope section.

Based on the above summary, the calculation is determined to be acceptable.

(Print Name and Sign)

Design Verifier: Joseph Smierciak / Joseph M. Smierciak	**Date: 03/28/16**
Others:	**Date:**

ENERCON	CALCULATION DESIGN VERIFICATION CHECKLIST	Calc. No. NAC004-CALC-02
		Rev. 0
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Item	CHECKLIST ITEMS	Yes	No	N/A
1	**Design Inputs** - Were the design inputs correctly selected, referenced (latest revision), consistent with the design basis and incorporated in the calculation?	X		
2	**Assumptions** - Were the assumptions reasonable and adequately described, justified and/or verified, and documented?	X		
3	**Quality Assurance** - Were the appropriate QA classification and requirements assigned to the calculation?	X		
4	**Codes, Standard and Regulatory Requirements** - Were the applicable codes, standards and regulatory requirements, including issue and addenda, properly identified and their requirements satisfied?	X		
5	**Construction and Operating Experience** - Have applicable construction and operating experience been considered?			X
6	**Interfaces** - Have the design interface requirements been satisfied, including interactions with other calculations?			X
7	**Methods** - Was the calculation methodology appropriate and properly applied to satisfy the calculation objective?	X		
8	**Design Outputs** - Was the conclusion of the calculation clearly stated, did it correspond directly with the objectives and are the results reasonable compared to the inputs?	X		
9	**Radiation Exposure** - Has the calculation properly considered radiation exposure to the public and plant personnel?			X
10	**Acceptance Criteria** - Are the acceptance criteria incorporated in the calculation sufficient to allow verification that the design requirements have been satisfactorily accomplished?	X		
11	**Computer Software** - Is a computer program or software used, and if so, are the requirements of CSP 3.02 met?			X

COMMENTS

Calculation has been designated a safety related document as noted on the cover page.

(Print Name and Sign)	
Design Verifier: Joseph Smierciak / Joseph M. Smierciak	**Date:** 03/28/16
Others:	**Date:**

TABLE OF CONTENTS

1.0 PURPOSE AND SCOPE

The purpose of this calculation is to evaluate the liquefaction potential and elastic settlement of the away-from-reactor licensing-basis independent spent fuel storage installation (ISFSI) concrete pad located at the Waste Control Specialists, LLC (WCS) site in Andrews, Texas.

The scope of work included:

- Review of Drawing NAC004-C-001, Rev. 0 showing the dimensions and general arrangement of the ISFSI concrete pad [Ref. 2.1], and review of Drawing NAC004-C-002, Rev. 0 showing the structural concrete plan, sections, and details [Ref. 2.2].
- Review of "Report of Geotechnical Exploration" performed by GEOServices, LLC [Ref. 2.3].
- Liquefaction potential evaluation using the data from Ref. 2.3.
- Elastic settlement evaluation under static loading conditions using the data from Ref. 2.3.

2.0 REFERENCES

2.1 ENERCON (2016), Drawing NAC004-C-001, Rev. 0, ISFSI Pad Licensing Design General Arrangement & Geotechnical.

2.2 ENERCON (2016), Drawing NAC004-C-002, Rev. 0, ISFSI Pad Licensing Design Structural Concrete Plan, Sections, & Details

2.3 GEOServices, LLC (2015), Report of Geotechnical Exploration: Consolidated Interim Storage Facility (CISF) Andrews, Texas, Project No. 31-151247.

2.4 Nuclear Regulatory Commission (2003), Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites, Regulatory Guide 1.198.

2.5 Youd, T.L. et. al. (2001), Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, October 2001, pp. 817-833.

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

2.7 Mathcad Computer Program, PTC Inc., Version 15.

2.8 Excel Computer Program, Microsoft Inc., Version 15.0.4771.1004.

3.0 BASIS/ASSUMPTIONS

3.1 It is assumed that the upper 4 feet of soils at the site will be excavated and replaced with compacted, dense graded aggregate [Ref. 2.1], with shear and elastic modulus at least equal to or greater than shown in the Table in Appendix C of Ref. 2.3 for the material to be excavated.

3.2 Groundwater was not encountered within the explored depths [Ref. 2.3]. For conservatism, liquefaction potential calculations were performed assuming the groundwater to be at the ground surface.

3.3 Based on Ref. 2.3, the two deeper borings performed within the ISFSI pad footprint encountered auger refusal (indicating that the soils were too hard to be drilled through using a power auger) at a depth of 37 and 45 feet. Two other deeper borings performed near the ISFSI pad encountered auger refusal at a depth of 40 feet. For settlement purposes, the depth to the incompressible layer was assumed to be 40 feet below the ground surface, which is the average depth where auger refusal was encountered in the four deepest soil borings at the project site. Based on the assumed concrete pad embedment depth of 3 feet [Ref. 2.1 & 2.2], the resulting thickness of compressible soils below the concrete pad is 37 feet.

3.4 The ISFSI concrete pad is assumed to be flexible for settlement purposes (which results in larger settlement in comparison to assuming a rigid pad, therefore is more conservative).

Refer to the body of the calculations (Section 6 of this document) for additional calculation-specific assumptions.

4.0 DESIGN INPUTS

4.1 Soil Properties

Max. allowable bearing pressure = 3,000 psf	[Ref. 2.3, Section 4.3.2]
Elastic moduli = see Table 5.2-2	[Ref. 2.3, Appendix C]
Poisson's ratio = see Table 5.2-2	[Ref. 2.3, Appendix C]
Thickness of compressible soils below the pad = 37 feet	[Section 3.3 of this document]
SPT N-values for ISFSI Pad	[Ref. 2.3, Figures 7, 8, and 9]
Total unit weight = 125 pcf	[Ref. 2.3, Appendix C]

4.2 Relevant Concrete Pad Properties

Pad dimensions = 55 feet wide by 135 feet long	[Ref. 2.1]
Pad embedment (below adjacent grade) = 3 feet	[Ref. 2.1 & Ref. 2.2]

5.0 METHODOLOGY

5.1 Liquefaction Potential Evaluation

Liquefaction potential evaluation was based on NRC Regulatory Guide 1.198 [Ref. 2.4] and widely accepted empirical methodology using Standard Penetration Test (SPT) and laboratory test data [Ref. 2.5].

The SPT N-values used in the calculation were obtained by computing the mean and standard deviation of the field N-values at each sample interval below 4 feet (i.e., 6 feet, 9 feet, 14 feet, 19 feet, 24 feet, 29 feet, 34 feet, and 39 feet) for the ISFSI pad soil test borings (B-101 through B-111) as given on Figures 7 through 9 in Ref. 2.3, then using the mean value minus one standard deviation as the representative N-value for that interval. For soils exhibiting field N-values greater than 100 blows per foot (bpf), the N-value was taken as 100 bpf. The results are shown below in Table 5.2-1:

Table 5.2-1. SPT N-values from soil borings for liquefaction potential evaluation

Depth (ft)	*B-101*	*B-102*	*B-103*	*B-104*	*B-105*	*B-106*	*B-107*	*B-108*	*B-109*	*B-110*	*B-111*	Mean N-value (bpf)	Stdev N-value (bpf)	Mean - 1 x Stdev (bpf)
	Field SPT N-value (bpf)													
2														
4														
6	22	46	16	39	32	37	36	37	42	15	52	34	12	**22**
9	78	55	66	71	56	52	73	100	40	42	51	62	18	**45**
14	98	100	100	76	60	100	100	100	100	50	60	86	20	**66**
19	55	100	66	78	40	100	100	100	40	40	90	74	26	**47**
24	89	100	56	62	65	100	100	100	63	84	65	80	18	**62**
29	100										66	83	24	**59**
34	100										79	90	15	**75**
39	100											100	N/A	**100**

5.2 Elastic Settlement Evaluation

The elastic settlement of the ISFSI concrete pad under uniform, static loading conditions was evaluated using an analytical solution based on the Theory of Elasticity, as described in Chapter 5-6 of Ref. 2.6. Pad dimensions used in the analysis are given in Section 4.2 of this document. The soil profile and properties shown in Table 5.2-2 on the following page were used in the calculation. The values shown in this Table are based on static elastic moduli as presented in Appendix C of Ref. 2.3, assuming that the excavated soils will be replaced with material of equal elastic modulus (as discussed in Section 3.1 of this document), and using a pad embedment depth of 3 feet [Ref. 2.1 & 2.2].

The elastic settlement calculation was performed using the maximum allowable bearing pressure of 3,000 psf as given in Ref. 2.3. Actual pad pressures are likely to be lower.

Table 5.2-2. Soil profile and properties used for elastic settlement evaluation (from Ref. 2.3)

Depth below bottom of pad (ft)	Poisson's Ratio	Static Elastic Modulus, E_s (psi)
0 – 7.5	0.35	9,796
7.5 – 12.5	0.35	28,289
12.5 – 32.5	0.35	32,667
32.5 – 37	0.35	95,255

6.0 CALCULATIONS

6.1 Liquefaction Potential

As stated in Section 3.2 of this document, groundwater was not encountered within the explored depths. Ref. 2.4 states that soils that "(1) are currently unsaturated (e.g., are above the water table), (2) have not been saturated previously (e.g., are above the historic high water table), and (3) cannot reasonably be expected to become saturated" can be considered to pose no potential liquefaction hazard.

For conservatism, we performed a liquefaction potential calculation considering the possibility of site soils becoming fully saturated. The calculation was performed in accordance with the procedures outlined in Ref. 2.5. According to Section 3.2.3 of Ref. 2.3, auger refusal (indicating that the soils were too hard to be drilled through using a power auger) was encountered at depths ranging between 37 and 45 feet, with an average depth of about 40 feet below the grades at the time of the exploration. As such, soils below a depth 40 feet are considered non-liquefiable.

Further, the following assumptions were made for liquefaction potential evaluation purposes:

- The energy efficiency of the hammer used for standard penetration testing was not provided in Ref. 2.3. An efficiency (denoted "ER" in the calculation) of 60 percent (which is typical) was assumed for hammer energy correction purposes.
- It was assumed that the compacted, dense graded aggregate that will be used to replace the excavated soils per Ref. 2.1 is not susceptible to liquefaction. Based on our experience, this is a reasonable assumption because the high relative density of compacted dense graded aggregate would preclude it from liquefaction.
- According to Section 3.2.1 and Appendix B of Ref. 2.3, the residual soils above the auger refusal depths which were tested for their percent fines indicated a fines content between 24 and 45 percent, with 7 out of 9 samples tested indicating a fines content of 34 percent or greater. A fines content of 35 percent was assumed for the calculation.
- The groundwater (GWT) was assumed to be at the ground surface.
- Total unit weight (γ_{total}) of soils was taken as 125 pcf. Unit weight of water (γ_w) was taken as 62.4 pcf. Atmospheric pressure (p_{atm}) was taken as 2.1 ksf.

The SPT N-values (mean minus one standard deviation) at each depth interval below 4 feet are as follows (from Table 5.2-1 of this document):

Sample depth:	SPT N-value (bpf):

$$\text{depth} := \begin{pmatrix} 6 \\ 9 \\ 14 \\ 19 \\ 24 \\ 29 \\ 34 \\ 39 \end{pmatrix} \cdot \text{ft} \qquad N := \begin{pmatrix} 22 \\ 45 \\ 66 \\ 47 \\ 62 \\ 59 \\ 75 \\ 100 \end{pmatrix}$$

The total and effective stresses in the soils can be calculated as follows (GWT = 0 feet):

$\gamma_{total} := 125 \cdot \frac{lbf}{ft^3}$ Total unit weight $\qquad \gamma_w := 62.4 \frac{lbf}{ft^3}$ Unit weight of water

Total stresses:

$$i := 0 .. \text{last}(\text{depth})$$

$$\sigma_{v_i} := \text{if}\left[\text{depth}_i \le GWT, \gamma_{total} \cdot \text{depth}_i, GWT \cdot \gamma_{total} + \left(\text{depth}_i - GWT\right) \cdot \left(\gamma_{total}\right)\right]$$

Hydrostatic stresses:

$$u_{0_i} := \text{if}\left[\text{depth}_i \le GWT, 0, \gamma_w \cdot \left(\text{depth}_i - GWT\right)\right]$$

Effective stresses:

$$\sigma_{v\_eff} := \sigma_v - u_0$$

The calculated stresses are summarized below:

--- Summary of stresses ---

$$\sigma_v = \begin{pmatrix} 0.750 \\ 1.125 \\ 1.750 \\ 2.375 \\ 3.000 \\ 3.625 \\ 4.250 \\ 4.875 \end{pmatrix} \cdot ksf \qquad u_0 = \begin{pmatrix} 0.374 \\ 0.562 \\ 0.874 \\ 1.186 \\ 1.498 \\ 1.810 \\ 2.122 \\ 2.434 \end{pmatrix} \cdot ksf \qquad \sigma_{v\_eff} = \begin{pmatrix} 0.376 \\ 0.563 \\ 0.876 \\ 1.189 \\ 1.502 \\ 1.815 \\ 2.128 \\ 2.441 \end{pmatrix} \cdot ksf$$

The overburden stress, rod length, borehole diameter, sampling method and hammer energy correction factors, as well as the calculated corrected SPT N-values are shown below:

Overburden stress correction factor:

$p_{atm} := 2.1ksf$ Atmospheric pressure

$$C_{N_i} := \left(\frac{2.2}{1.2 + \frac{\sigma_{v\_eff_i}}{p_{atm}}} \right) \qquad C_N = \begin{pmatrix} 1.596 \\ 1.498 \\ 1.360 \\ 1.245 \\ 1.149 \\ 1.066 \\ 0.994 \\ 0.931 \end{pmatrix}$$

Rod length correction factor:

$$C_{R_i} := \begin{vmatrix} 0.75 \text{ if } depth_i < 3m \\ 1.0 \text{ if } (depth_i \geq 3m) \end{vmatrix} \qquad C_R = \begin{pmatrix} 0.750 \\ 0.750 \\ 1.000 \\ 1.000 \\ 1.000 \\ 1.000 \\ 1.000 \\ 1.000 \end{pmatrix}$$

Sampler correction factor
(standard sampler)

$C_S := 1.0$

Borehole diameter correction factor
(Borehole diameter 65–115 mm)

$C_B := 1.0$

Hammer energy correction factor:

$ER := 60$ Hammer energy ratio

$C_E := \frac{ER}{60} = 1.00$

Corrected SPT N value:

$N_{1\_60} := \overrightarrow{\left(N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S\right)}$

$$N_{1\_60} = \begin{pmatrix} 26.3 \\ 50.6 \\ 89.8 \\ 58.5 \\ 71.2 \\ 62.9 \\ 74.5 \\ 93.1 \end{pmatrix}$$

At this point, further calculations were deemed unnecessary because according to Ref. 2.5, soils with a fines content of 35 percent and exhibiting a corrected blow count of about 21 or greater are not susceptible to liquefaction (see Figure 6.1-1 below; points to the right of the curves shown are considered not susceptible to liquefaction). It should also be noted that Figure 6.1-1 is for a moment magnitude (M) 7.5 earthquake. For earthquakes with M less than 7.5 (as expected for this site), the blow count threshold would be even lower. Therefore, overall the soils beneath the ISFSI concrete pad are deemed not susceptible to liquefaction, even if they should become saturated all the way to the ground surface (which is highly unlikely).

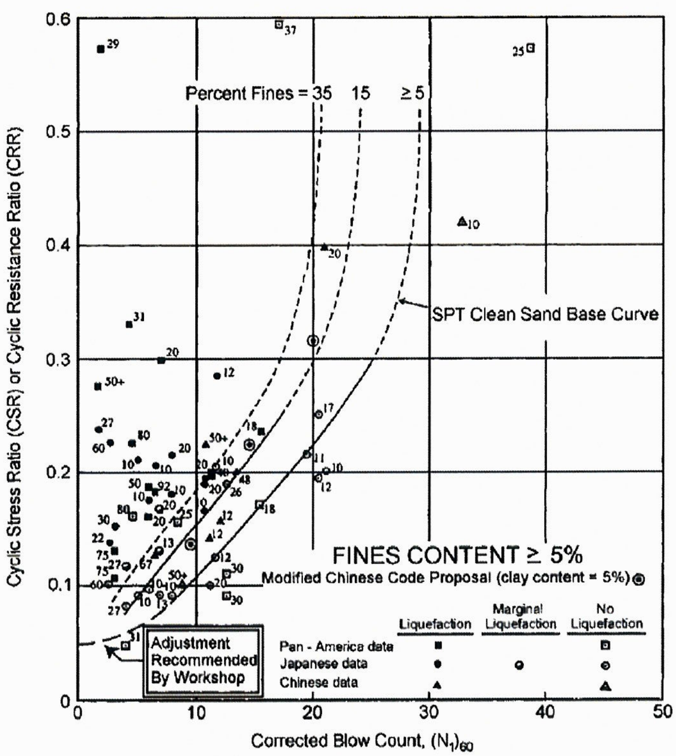


Figure 6.1-1. SPT-based liquefaction curves for M = 7.5 earthquakes (from Ref. 2.5)

6.2 Elastic Settlement

The settlement of the center and corner of a flexible rectangular base on the surface of a elastic half space can be computed from the Theory of Elasticity as follows (from Ref. 2.6):

$$\Delta H_{center} = q B' \frac{1-\mu^2}{E_s}(4 I_s) I_f \qquad \text{Center of footing}$$

$$\Delta H_{corner} = q B' \frac{1-\mu^2}{E_s} I_s I_f \qquad \text{Corner of footing}$$

Where:

ΔH = settlement (in consistent units)
q = contact pressure (in consistent units)
B' = least dimension (width) of rectangular area (in consistent units)
μ = Poisson's ratio
E_s = Elastic soil modulus (in consistent units)
I_s, I_f = influence factors (dimensionless) which depend on foundation dimensions, thickness of compressible stratum (h), Poisson's ratio, and embedment depth (d).

The influence factor I_s can be calculated as follows:

$$I_s = I_1 + \frac{1-2\mu}{1-\mu} I_2$$

$$I_1 = \frac{1}{\pi}\left[M \ln\left[\frac{\left(1+\sqrt{M^2+1}\right)\sqrt{M^2+N^2}}{M\left(1+\sqrt{M^2+N^2+1}\right)}\right] + \ln\left[\frac{\left(M+\sqrt{M^2+1}\right)\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right]\right]$$

$$I_2 = \frac{N}{2\pi}\operatorname{atan}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right)$$

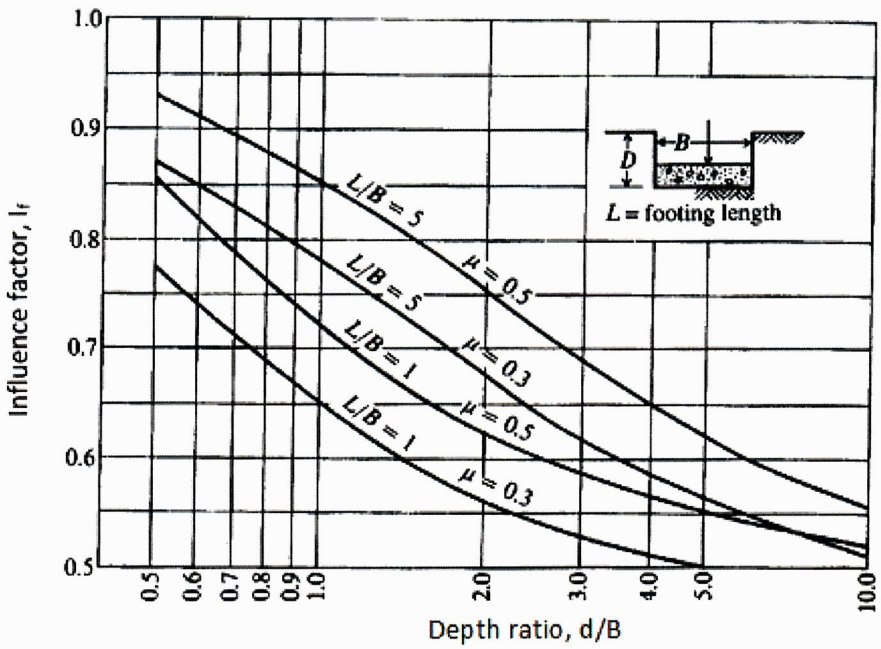
Where:

$$M = \frac{L'}{B'} \qquad N = \frac{h}{B'}$$

$B'_{center} = \frac{B}{2}$ Center of footing $\qquad$ $B'_{corner} = B$ Corner of footing

$L'_{center} = \frac{L}{2}$ Center of footing $\qquad$ $L'_{corner} = L$ Corner of footing

The influence factor I_f can be obtained graphically from the chart below using linear interpolation:



For this problem, from Section 4.1 and 4.2 of this document:

$B := 55ft$ $\qquad h := 37ft$ $\qquad \mu := 0.35$

$L := 135ft$ $\qquad d := 3ft$

The length to width ratio (L/B) and depth ratio (d/B) for the concrete pad can be calculated as follows:

$\frac{L}{B} = 2.45$ $\qquad \frac{d}{B} = 0.05$

From the chart above, for a Poisson's ratio of μ = 0.35, the influence factor I_f is approximately:

$I_f := 0.9$

For conservatism, use $I_f = 0.95$ (higher value results in larger settlement, hence more conservative):

$I_f := 0.95$

For the center of the footing:

$$B'_{center} := \frac{B}{2} = 27.5\,ft \qquad L'_{center} := \frac{L}{2} = 67.5\,ft$$

$$M := \frac{L'_{center}}{B'_{center}} = 2.45 \qquad N := \frac{h}{B'_{center}} = 1.35$$

$$I_1 := \frac{1}{\pi}\left[M\ln\left[\frac{\left(1+\sqrt{M^2+1}\right)\sqrt{M^2+N^2}}{M\left(1+\sqrt{M^2+N^2+1}\right)}\right] + \ln\left[\frac{\left(M+\sqrt{M^2+1}\right)\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right]\right] = 0.182$$

$$I_2 := \frac{N}{2\pi}\operatorname{atan}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right) = 0.118$$

$$I_{s\_center} := I_1 + \frac{1-2\mu}{1-\mu}I_2 = 0.236$$

For the corner of the footing:

$$B'_{corner} := B = 55\,ft \qquad L'_{corner} := L = 135\,ft$$

$$M := \frac{L'_{corner}}{B'_{corner}} = 2.45 \qquad N := \frac{h}{B'_{corner}} = 0.67$$

$$I_1 := \frac{1}{\pi}\left[M\ln\left[\frac{\left(1+\sqrt{M^2+1}\right)\sqrt{M^2+N^2}}{M\left(1+\sqrt{M^2+N^2+1}\right)}\right] + \ln\left[\frac{\left(M+\sqrt{M^2+1}\right)\sqrt{1+N^2}}{M+\sqrt{M^2+N^2+1}}\right]\right] = 0.065$$

$$I_2 := \frac{N}{2\pi}\operatorname{atan}\left(\frac{M}{N\sqrt{M^2+N^2+1}}\right) = 0.099$$

$$I_{s\_corner} := I_1 + \frac{1-2\mu}{1-\mu}I_2 = 0.111$$

Ref. 2.6 suggests that the weighted average of the elastic modulus be used for settlement calculations. In this case, the weighted average can be computed using the values in Table 5.2-2 as follows:

$$E_s := \frac{7.5\text{ft} \times 9796\text{psi} + 5\text{ft} \times 28289\text{psi} + 20\text{ft} \times 32667\text{psi} + 4.5\text{ft} \times 95255\text{psi}}{7.5\text{ft} + 5\text{ft} + 20\text{ft} + 4.5\text{ft}}$$

$$E_s = 35051\,\text{psi}$$

$$E_s = 5.047 \times 10^6\,\text{psf}$$

The settlement at the center and corner of the concrete pad for a uniform pressure of q=3,000 psf can then be calculated as follows :

$$q := 3000\text{psf}$$

$$\Delta H_{center} := qB'_{center}\frac{1-\mu^2}{E_s}\left(4I_{s\_center}\right)I_f = 0.15\,\text{in}$$

$$\Delta H_{corner} := qB'_{corner}\frac{1-\mu^2}{E_s}I_{s\_corner}I_f = 0.04\,\text{in}$$

The estimates above are based on the "best estimate" elastic moduli as given in Ref. 2.2. To assess the sensitivity of the settlements to variations in elastic moduli, we can assume a coefficient of variation (COV) of 1.0, and obtain a lower-bound (denoted by subscript "lb") estimate of the weighted average elastic modulus as follows:

$$COV := 1.0$$

$$E_{s\_lb} := \frac{E_s}{1 + COV} = 2.524 \times 10^6\,\text{psf}$$

Using this value, the estimated settlements are:

$$\Delta H_{center} := qB'_{center}\frac{1-\mu^2}{E_{s\_lb}}4I_{s\_center}I_f = 0.31\,\text{in}$$

$$\Delta H_{corner} := qB'_{corner}\frac{1-\mu^2}{E_{s\_lb}}I_{s\_corner}I_f = 0.07\,\text{in}$$

7.0 SUMMARY OF RESULTS AND CONCLUSIONS

Based on the evaluation contained in Section 6.1, it is concluded that overall the soils below the ISFSI pad are not susceptible to liquefaction.

Based on the evaluation contained in Section 6.2, the estimated settlement at the center of the ISFSI pad (assuming the pad to be flexible for settlement purposes) for a uniform bearing pressure of 3,000 psf is on the order of 0.15 to 0.3 inch, with a differential settlement (between the corner and center of the concrete pad) on the order of ¼ inch or less.