E-55412

Book 1 of 2

Enclosure 3 to E-55412

Draft RAI Responses with associated SAR change pages

(Public Version)

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

<u>Soil Parameter Selection:</u>

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.

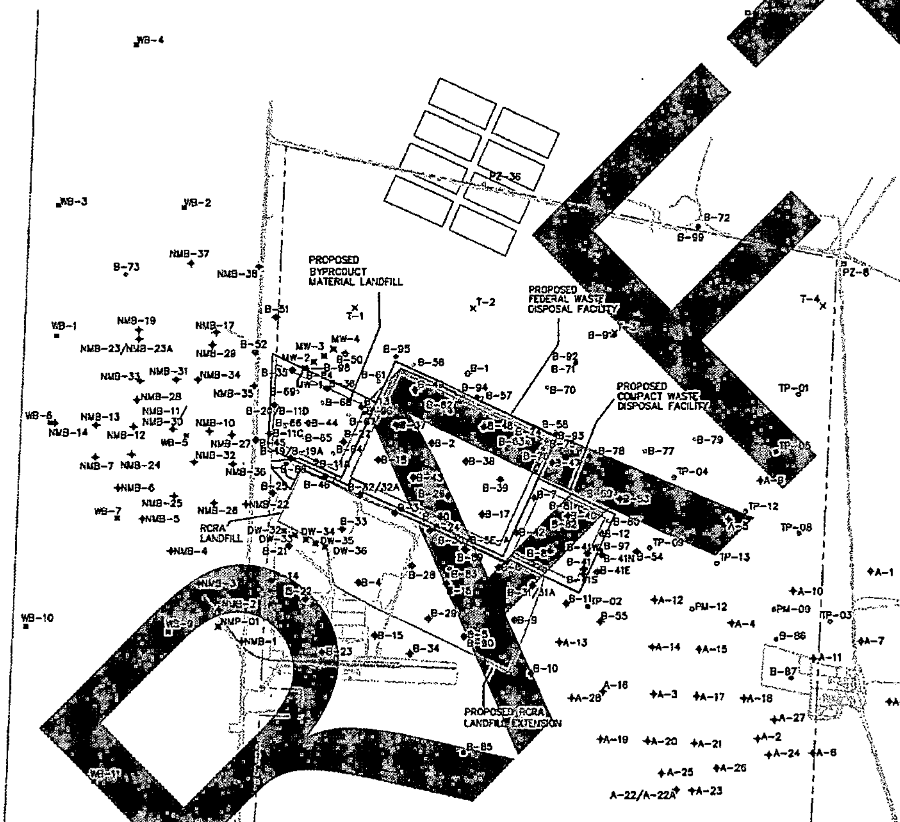

Figure NP-2.6-3-1
Historical Borings at WCS Site

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.

Attachment E

Geotechnical Investigation for WCS CISF

(347 pages)

All Indicated Changes are in response to RAI NP-2.6-3

GEOServices, LLC, Geotechnical and Materials Engineers

February 18, 2020

Waste Control Specialists, LLC
17101 Preston Road, Suite 15
Dallas, Texas 75248

ATTENTION: Mr. Ben Mason, Director of Engineering
bmason@wcstexas.com

Subject: **REPORT OF GEOTECHNICAL EXPLORATION**
Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No. 31-151247.R2

Dear Mr. Mason:

We are submitting the results of the geotechnical exploration performed for the proposed Consolidated Interim Storage Facility (CISF) in Andrews, Texas. The geotechnical exploration was performed in accordance with GEOServices' Proposal No. 13-151124Rev1 dated June 23, 2015 and authorized by you.

The following report presents our findings and recommendations for the proposed construction of the Consolidated Interim Storage Facility project. Should you have any questions regarding this report, or if we can be of any further assistance, please contact us at your convenience.

Sincerely,

GEOServices, LLC

Derek K. Kilday, P.E. (TN)
Vice President

STATE OF TEXAS
DENNIS A. HUCKABA II
99027
LICENSED
PROFESSIONAL ENGINEER
2-18-2020

Dennis A. Huckaba, P.E.
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DRAFT

REPORT OF GEOTECHNICAL EXPLORATION

Submitted to:

Waste Control Specialists, LLC
17101 Preston Road, Suite 15
Dallas, Texas 75248

CONSOLIDATED INTERIM STORAGE FACILITY (CISF)

ANDREWS, TEXAS

Submitted by:

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5559 N. Lee Highway
Cleveland, Tennessee 37312

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GEOServices, LLC, Geotechnical and Materials Engineers

GEOSERVICES, LLC
PROJECT NO. 31-151247.R2

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1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this geotechnical exploration was to characterize the subsurface conditions for the design and construction of the Consolidated Interim Storage Facility (CISF) in Andrews, Texas. This report provides recommendations for general site preparation, foundation design and slab-on-grade construction.

1.2 PROJECT DESCRIPTION

The project site is located at the existing Waste Control Specialists (WCS) Andrews facility located at 9998 Highway 176 West in Andrews, Texas. The proposed construction will consist of a 200-acre storage facility. The Consolidated Interim Storage Facility (CISF) will consist of eight consolidated interim storage facilities, transfer facility (Cask Handling Building), and administration building. The scope of this exploration was limited to one of the consolidated interim storage facilities, the transfer facility, and the administration building.

Each of the consolidated interim storage facilities (CISF) are planned to be 280,000 square feet (800 feet by 350 feet) in size. We understand that each CISF will consist of a gravel pad with a number of smaller cast-in-place reinforced concrete mat foundations that will each hold 24 (3x8 array) storage casks when full loaded. The individual mat foundations measure approximately 7,425 square feet (55 feet by 135 feet). Based on the loading provided by Enercon, each of the casks will have a diameter of 11'-4" and a height of just under 19 feet. The casks will have a maximum loaded weight of 360 kips. In addition to the weight of the casks, an operational and occupancy live load of 200 psf will be utilized. Based on the provided loading, the mats will impart a bearing pressure of 4,500 psf or less to the underlying subgrade.

The transfer facility (Cask Handling Building) is a two-bay Important to Safety (ITS) – Category B steel structure. The Cask Handling Building measures 175 feet by 193 feet in plan dimension and has a height of 72 feet. The structure will have rail access to facilitate cask unloading operations, canister transfer operations, and other maintenance activities. Two overhead bridge cranes will be utilized within the structure to facilitate rail car unloading. Based on information provided by AECOM, we understand that the foundations for the proposed cask handling building will bear at a depth of 10 to 11 feet below existing grade. Based on the loading information provided, we understand that maximum service level bearing pressure of less than 3.5 kips per square foot (ksf) are expected, while maximum limit state bearing pressures will approach 5.5 ksf.

The administration building will be traditional commercial construction and will consist of a single-story steel frame construction with a slab-on-grade. At the time this report was prepared, the administration building had not yet been designed. However, based on our experience with similar structures, we anticipate maximum column loads of less than 75 kips and maximum wall loads of 2 to 4 kips per linear foot.

The 200-acre tract of land is currently undeveloped with the exception of access roads that cross from one property to the adjacent property. Based on information obtained from internet research, site elevations range from approximately 3,505 feet Mean Sea Level (MSL) along the eastern property boundary to approximately 3,490 feet MSL along the western property boundary. Based on the provided grading plan, we anticipate average cuts and fills on the order of three feet or less will be required for this project.

1.3 SCOPE OF SERVICES

This geotechnical exploration involved a site reconnaissance, field exploration, laboratory testing, and engineering analysis. The following sections of this report present discussions of the field exploration, laboratory testing programs, site conditions, and conclusions and recommendations. Following the text of this report, figures, boring logs, and laboratory test results are provided in the appendices. Appendix A provides figures and boring logs. Appendix B provides laboratory tests performed and the results of these tests. Appendix C provides a summary table of the Site Soil Characteristics. Appendix D provides the static elastic modulus calculation. Appendix E provides the results of the on-site shear wave velocity study. Appendix F provides the seismic densification analysis calculations. Appendix G provides bearing capacity analyses and commentary. Appendix H provides the settlement analysis for the CISF pads and Cask Handling Building.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, subsurface water, or air, on, or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

2.0 EXPLORATION AND TESTING PROGRAMS

2.1 FIELD EXPLORATION

The site subsurface conditions were explored with eighteen soil test borings. The following is a breakdown of the boring layout:

STRUCTURE	Number of Borings	Boring Designation
Proposed Transfer Facility	5	TF-1 and TF-4
		TF-2, TF-3, and TF-5
CISF – Phase I	11	B-101 and B-111
		B-102 through B-110
Administration Building	2	AB-1 and AB-2

Table 1 – Boring Breakdown

The boring locations and depths were selected by GEOServices. The borings were surveyed in the field by WCS personnel. Drilling was performed between July 13th and July 21st, 2015. The soil test borings were advanced using a Cannon skid rig (air rotary) and a CME-55 track rig. The drill crew worked in general accordance with ASTM D6151 (HSA Drilling). Sampling of overburden soils was accomplished using the standard penetration test procedure (ASTM D1586). The borings were backfilled with soil cuttings prior to leaving the site.

In split–spoon sampling, a standard 2-inch O.D. split-spoon sampler is driven into the bottom of the boring with a 140 pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler the last 12 inches of the standard 18 inches of total penetration is recorded as the Standard Penetration Resistance (N-value). These N-values are indicated on the boring logs at the testing depth, and provide an indication of the relative density of granular materials and strength of cohesive materials.

2.2 LABORATORY TEST PROGRAM

Soil samples collected during drilling were transported to our laboratory for visual classification and laboratory testing. The following laboratory testing was performed on select samples to determine the various soil properties.

- Atterberg Limits (ASTM D4318): Three Atterberg Limits tests were performed. These tests help us to confirm our visual classifications according to the AASHTO Classification System and the Unified Soil Classification System (USCS). The plastic limit and liquid limit represent the moisture content at which a cohesive soil changes from a semi-solid to a plastic state and from a plastic state to liquid state, respectively.
- Natural Moisture Content (ASTM D2216): One-hundred thirty-four moisture content determinations were performed. The natural moisture content is defined as the ratio of the weight of water present in the soil to the dry weight of soil.
- 200 Wash Analysis (ASTM D1140): Nine particle size analyses were performed. The particle size analysis is used to determine the soil classification and determine drainage properties of the material.
- Resistivity of Soil (ASTM G187): Four soil resistivity tests were performed. The resistivity tests provide information related to corrosive properties of soil.
- Consolidated Undrained Triaxial Test (ASTM D4767): Consolidated undrained triaxial tests were planned, however, undisturbed Shelby tubes were not able to be performed due to the caliche

present. This test provides data useful in determining strength and deformation properties of cohesive soils.

- Standard Proctor Moisture-Density Tests (ASTM D698): One standard Proctor test was performed on a composite soil sample. This test provides information concerning the relationship between moisture content, compaction effort, and density.
- California Bearing Ratio (CBR) Tests (ASTM D1883): One CBR test was performed on a composite soil sample. This test provides a CBR value, which is used in pavement design to represent the support of the soil subgrade.
- Consolidation (ASTM D2435): Consolidation tests were originally planned, however, undisturbed Shelby tube samples could not be obtained due to the caliche. The test results are used to evaluate the settlement potential of the clay stratum.

The test results of the laboratory testing are presented in the Soil Data Summary enclosed in Appendix B.

3.0 SITE CONDITIONS

3.1 GEOLOGIC CONDITIONS

The WCS site is located over the north-central portion of a prominent subsurface structural feature known as the Central Basin Platform. The geologic formations of concern, beneath of the WCS facility comprise, from oldest to youngest, the Triassic Dockum Group, the Cretaceous Trinity Group Antlers Formation, the Late Tertiary Ogallala Formation, the Late Tertiary/Quaternary Gatuña Formation or Cenozoic Alluvium (note that the Gatuña Formation and Cenozoic Alluvium are sometimes used interchangeably), the Pleistocene windblown sands of the Blackwater Draw Formation, Holocene windblown sands and playa deposits. A regional hard caliche pedisol, termed the Caprock caliche, developed on all pre-Quaternary formations before the Blackwater Draw sands were deposited.

3.2 SUBSURFACE CONDITIONS

3.2.1 Encountered Soils

The geologic profile for the CISF area consists of loose cover sands which overlay caliche, the Blackwater Draw, and Ogallala Formation. A combination of these materials was encountered in each of the eighteen soil test borings to auger refusal and/or boring termination depths ranging from 25 to 45 feet below the existing ground surface elevation.

Cover sands were encountered in five of the eighteen soil test borings (AB-2, B-101, B-102, B-103, TF-2, and TF-5) to depths ranging from 2.5 to 6.5 feet. These sands were generally loose to very loose in consistency. We anticipate that the cover sands will be encountered in other area of the site as well. Clearing activities to access some of the boring locations likely removed some of the thinner layers of cover sands.

Beneath the cover sands, caliche with silty sands (SM) were encountered to auger refusal and/or boring termination depths ranging from 25 to 45 feet below the existing ground surface elevation. The N-values of the standard penetration resistance test (SPT) are used to evaluate the relative consistency or density of the subsurface soils. The N-values for the encountered soils ranged from 13 bpf to 100 blows per 1 inch of penetration, indicating a relative density of medium dense to very dense.

The natural moisture content of the sampled soils ranged from 2.5 to 9 percent. Atterberg limits testing on three selected residual samples revealed liquid limits (LL) ranging from 26 to 29 percent and each sample was non-plastic. Wash 200 tests performed on eight soil samples revealed 24 to 45 percent finer than the 200 sieve.

3.2.2 Subsurface Water

Subsurface water was not observed in any of the soil test borings either during or at the completion of drilling activities. Subsurface water levels may fluctuate due to seasonal changes in precipitation amounts or due to construction activities in the area. The groundwater information presented in this report is the information that was collected at the time of our field activities.

3.2.3 Auger Refusal Conditions

Auger refusal materials were encountered in four of the eighteen soil test borings (B-101, B-111, TF-1, and TF-4) at depths ranging from 37 to 45 feet below the existing ground surface elevation. The remaining soil test borings were terminated at a depth of 25 feet prior to encountering refusal materials. Refusal is a designation applied to any material that cannot be penetrated by the power auger. The following table presents the auger refusal depths. Auger refusals could indicate a number of materials, however, we understand that in the CISF area the altered portion of the Ogallala Formation, or the Caprock Caliche, is horizontally present across the entire CISF footprint. The caliche is present in various stages of development (Machette, 1985) both vertically and horizontally across the CISF site. The location of the well-developed (stage 3-5) caliche at the CISF is generally indicated by auger refusal in geotechnical borings. Based on this information, it appears the auger refusal materials consisted of stage 3-5 caliche. Rock coring was beyond the scope of this exploration, so the character and continuity of the refusal materials was not determined. The following table should be reviewed for auger refusal depths:

AUGER REFUSAL DEPTHS	
Location	Refusal Depth (feet)
B-101	45
B-111	37
TF-1	40
TF-4	40

Table 2 – Auger Refusal Depths

3.2.4 General

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in Appendix A should be reviewed for specific information at individual boring locations. The depth and thickness of the subsurface strata indicated on the test records were generalized from and interpolated between boring locations. The transition between materials will be more or less gradual than indicated and may be abrupt. Information on actual subsurface conditions exists only at the specific test locations and is relevant to the time the exploration was performed. Variations may occur and should be expected between boring locations. The stratification lines were used for our analytical purposes and, unless specifically stated otherwise, should not be used as the basis for design or construction cost estimates.

3.2.5 Additional Provided Resources

As mentioned previously, of the eighteen borings performed for the CISF project only four of the borings encountered auger refusal. The auger refusal depths ranged from 37 to 45 feet below the ground surface (bgs). Industry standards would typically result in an extension of one or more of the borings to a greater depth. The purpose of the extension would be 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 have been performed on the WCS property as well as shear wave testing. 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 section below outlines the previous studies which were utilized to extend the soil column to a depth of 600 feet. Additional information regarding the soil column development and soil parameters obtained for use in the settlement analysis are provided in Appendices C, D and H.

<u>Provided Additional Documents:</u>

1. AECOM. (2016). Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, WCS Centralized Interim Storage Facility Project. Dated March 18, 2016.
2. WCS. (2007). (Waste Control Specialists LLC). Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste. Dated March 2007.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 SITE ASSESSMENT

The results of the subsurface exploration indicate that the site is adaptable for the proposed construction. However, as is the case with most sites, some inherent challenges are associated with this development of this site. These challenges include the presence of isolated zones of loose cover sands.

As mentioned previously, very loose to loose sands were encountered in five of the eighteen soil test borings (AB-2, B-101, B-102, B-103, TF-2, and TF-5) to depths ranging from 2.5 to 6.5 feet. While we anticipate that foundation excavations would penetrate the majority of the loose cover sands, we anticipate that some undercutting of cover sands will be required were encountered in structural and roadway areas. Where undercutting is required, the depths of undercut and replacement materials should be provided by the geotechnical engineer of record during constructions. Based on the proposed foundation loads and bearing elevations, we anticipate the undercuts can be backfilled with caliche compacted to the requirements outlined in Section 4.2.2.

4.2 SITE PREPARATION

4.2.1 Subgrade

All vegetation, organic soils, rock fragments greater than 6 inches, and other debris should be removed from the proposed construction area. The actual depth of removal should be determined by a representative of the geotechnical engineer at the time of construction.

After completion of stripping operations and any required excavations to reach planned subgrade elevation, we recommend that the subgrade be proofrolled with a fully-loaded, tandem-axle dump truck or other pneumatic-tired construction equipment of similar weight. The geotechnical engineer or his representative should observe proofrolling. Areas to receive structural soil fill should also be proofrolled prior to the placement of any fill. Based on the results of the drilling activities, very loose to loose sands were encountered in five of the 18 soil test borings (AB-2, B-101, B-102, B-103,TF-2, and TF-5) to depths ranging from 2.5 to 6.5 feet. We anticipate that these soils encountered will perform unsatisfactorily during proofrolling activities. The project budget should include a contingency for required undercutting of the upper loose soils within the proposed building footprints and roadway sections and replacement with properly compacted fill.

4.2.2 Structural Soil Fill

Characteristics of recommended fill soils and the placement and compaction criteria for fill are provided in the table on the following page. The results of our limited laboratory testing indicate that **SOME** of the on-site materials **DO** meet the criteria for reuse as structural fill. However, we recommend that the near surface silty sands NOT be reused as compacted fill. Therefore, dependent on grading requirements, some fill materials may need to be imported during grading. The grading contractor should include provisions in their bid for importing new soil materials and exporting excess materials.

The near surface fill materials consists of sands that contain more than 15 percent fines. Experience indicates these materials can be moisture sensitive and degrade rapidly under heavy rubber-tired equipment. Therefore, the contractor should be aware that if these materials will be reused as fill or are present at the subgrade level, some repairs of subgrades that degrade during the construction may be required prior to pavement construction.

Prior to initiating grading activities, samples of proposed fill soils should be submitted for Atterberg limits and moisture-density relationship determination testing (i.e., standard Proctor). This testing typically requires at least 3 to 4 days to complete. To avoid delays during grading, samples of proposed fill materials (both on-site and off-site) should be collected during site preparation activities.

SUMMARY OF RECOMMENDED FILL CRITERIA

MATERIAL TYPE	CHARACTERISTICS	COMPACTION PROCEDURES	COMPACTION CONTROL
COARSE-GRAINED SOILS (CALICHE)	▪ Maximum gravel size – 1 inch ▪ Maximum gravel content – 30 percent retained on a ¾-inch sieve ▪ Maximum allowable organic content – 5 percent by weight, but no large roots should be allowed ▪ USCS Classification SP, SC, SM	▪ Maximum loose lift thickness – 6 inches **Compaction requirement[1]:** ▪ The fill should be compacted by making multiple passes with an appropriately sized sheepsfoot roller. ▪ Compaction should be at least 95 percent of the standard Proctor maximum (ASTM D 698) **Moisture content for fill:** ▪ At time of compaction – within minus 2 and plus 2 percent of the optimum moisture content	**Building and pavement areas:** ▪ One test every 2,500 to 5,000 square feet per lift, with a minimum of two tests per lift **Trench areas:** ▪ One test every 100 linear feet per lift **Minimum requirement:** ▪ Two tests per lift ***(for preliminary planning only, our technician or engineer should determine the actual test frequency)***
[1] In addition, the fill must be stable under the influence of the compaction equipment. After the soil fill is properly placed and compacted, it will be advisable to limit the amount of heavy construction traffic on the soil subgrade.			

Table 3 – Summary of Fill Criteria

4.3 FOUNDATION AND STRUCTURAL RECOMMENDATIONS

4.3.1 Administration Building Foundations

The security and administration building will be traditional commercial construction and will consist of a single-story steel frame construction with a slab-on-grade. Foundations for the proposed construction will be supported on the underlying caliche with sand and/or properly compacted structural fill materials. The recommended preliminary allowable bearing capacity for design of the foundations is 3,000 pounds per square foot (psf) or less. A one-third increase in the allowable bearing capacity for all load conditions that include transient loads (wind, seismic, other short term loads) is permitted. The 33% increase in allowable bearing capacity (stress) can be applied to load combinations that consider transient loads in conjunction with dead loads. This increase in allowable stress cannot be applied solely to dead loads. We recommend that continuous foundations be a minimum of 18 inches wide and isolated spread footings be a minimum of 24 inches wide to reduce the possibility of a localized punching shear failure. All exterior footings should be designed to bear at least 36 inches below finished exterior grade.

Foundation excavations should be opened, the subgrade evaluated, remedial work performed, and concrete placed in an expeditious manner. Exposure to weather often reduces foundation support capabilities, thus necessitating remedial measures prior to concrete placement. It is also important that proper surface drainage be maintained both during construction (especially in terms of maintaining dry footing trenches) and after construction.

4.3.1.1 Administration Building Settlement

As mentioned previously, at the time this report was prepared the Administration Building was still being designed. Therefore detailed loading and foundations sizes were not available. Based on the conditions encountered in our borings, and the anticipated loading (maximum column loads of 75 kips or less) we anticipate that total settlements will be less 0.5 inches for the administration building. This is based on the assumption the foundations will bear in the caliche and sand matrix or newly placed structural soil fill. Once the building design is finalized these settlement calculations can be updated to include the actual foundation loads and sizes.

4.3.1.2 Slabs-on-Grade (Administration Building)

For slab-on-grade construction for the administration building, the site should be prepared as previously described. If moisture mitigation through the slab is a concern, we recommend that the subgrade be topped with a minimum 6-inch layer of crushed stone. A polyethylene vapor barrier is not required if the designer utilizes a dense graded aggregate base. If a dense aggregate base is not utilized a vapor barrier should be placed beneath the slab. The vapor barrier material should be in compliance with ASTM E 1745 and have a thickness of at least 10 mils (0.3 mm), as recommended by ACI 302.1R-04 "Guide for Concrete Floor and Slab Construction". The vapor barrier material should be of sufficient strength and durability to resist puncture during reinforcing steel and concrete placement. Placement of the vapor barrier should be in accordance with manufacturer's recommendations.

The subgrade should be proofrolled and approved prior to the placement of the crushed stone. Based on the conditions encountered on this site, we recommend that the floor slabs be designed using a subgrade modulus of 150 pounds per cubic inch (pci). This subgrade modulus value is for small diameter loads (i.e., a 1 foot by 1 foot plate) and should be adjusted for wider loads such as large mat foundations.

4.3.2 Transfer Facility (Cask Handling Building) Foundations

The transfer facility (Cask Handling Building) is a two-bay ITS – Category B steel structure. The Cask Handling Building measures 175 feet by 193 feet in plan dimension and has a height of 72 feet. Based on information provided by AECOM, we understand that the foundations for the proposed cask handling building will bear at a depth of 10 to 11 feet below existing grade. Foundations for the cask handling building will bear in the Caliche with Sand Matrix. The recommended allowable bearing capacity for the service level design of the foundations is 4,000 pounds per square foot (psf) or less. An allowable bearing pressure for limit state loadings of 6,000 psf can be utilized. This bearing pressure is based on a foundation bearing depth of 10 feet below grade. Should the foundation elevations be changed, the geotechnical engineer should be contacted to evaluate the bearing capacity and settlement at the new foundation elevation. Bearing capacity calculations are provided in Appendix G of this report.

4.3.2.1 Transfer Facility (Cask Handling Building) Settlement Analysis

Settlement analysis for the transfer facility was performed using the soil column outlined in Appendix D. The settlement calculation was performed utilizing Settle3 a finite difference software produced by RocScience. Settle3 allows for the input of the foundation loads for the entire footprint so that any stress overlaps between adjacent foundations can be analyzed. For the Cask Handling Building the service level loads shown on AECOM drawing WCS01-13-2001 dated December 24, 2019 were utilized for the analysis. Both a dead load sustained case and a seismic case were analyzed. The gross bearing pressures provided were used for the analysis. It should be noted that if the bearing pressure or bearing depth changes, the geotechnical engineer should be contacted to update the calculations. The results of the analysis are shown below. Detailed settlement calculations are provided in Appendix H.

Load Combination	Maximum Total Settlement
Dead (1.0D)	Less Than 0.25 inches
Seismic (1.0D + 0.7E)	Less Than 0.50 inches

Table 4 – Summary of Cask Handling Building Settlement Results

4.3.2.2 Slabs-on-Grade (Cask Handling Building)

For slab-on-grade construction for the administration building, the site should be prepared as previously described. If moisture mitigation through the slab is a concern, we recommend that the subgrade be topped with a minimum 6-inch layer of crushed stone. A polyethylene vapor barrier is not required if the designer utilizes a dense graded aggregate base. If a dense aggregate base is not utilized a vapor barrier should be placed beneath the slab. The vapor barrier material should be in compliance with ASTM E 1745 and have a thickness of at least 10 mils (0.3 mm), as recommended by ACI 302.1R-04 "Guide for Concrete Floor and Slab Construction". The vapor barrier material should be of sufficient strength and durability to resist puncture during reinforcing steel and concrete placement. Placement of the vapor barrier should be in accordance with manufacturer's recommendations.

The subgrade should be proofrolled and approved prior to the placement of the crushed stone. Based on the conditions encountered on this site, we recommend that the floor slabs be designed using a subgrade modulus of 150 pounds per cubic inch (pci). This subgrade modulus value is for small diameter loads (i.e., a 1 foot by 1 foot plate) and should be adjusted for wider loads such as large mat foundations. Once preliminary slab pressures are provided we can assist the structural engineer in adjusting the subgrade modulus values to account for wider loads. The procedure outlined in section 4.3.3.1 of this report can be utilized.

4.3.3 CISF Pad Foundations

Each of the consolidated interim storage facilities (CISF) are planned to be 280,000 square feet (800 feet by 350 feet) in size. We understand that each CISF will consist of a gravel pad with a number of smaller cast-in-place reinforced concrete mat foundations that will each hold 24 (3x8 array) storage casks when full loaded. The individual mat foundations measure approximately 7,425 square feet (55 feet by 135 feet). The concrete mat foundation will measure 36 inches in thickness and bear at a depth of 4 feet below grade. The concrete mat foundation will be based on a minimum of 12 inches of dense graded aggregate.

As mentioned previously in the site assessment, loose cover sands are prevalent beneath the CISF site to depths of up to depths of up to 7.5 feet. Where encountered, these soils will have to be undercut and replaced with properly compacted caliche or crushed stone to provide adequate support of the proposed CISF pads. Provided the recommendations in the site assessment are followed, an allowable bearing pressure of 5 ksf can be utilized for design. Bearing capacity calculations are provided in Appendix G of this report.

4.3.3.1 CISF Mat Foundation Recommendations

The CISF Pads experience a series of complex loadings due to the number of casks on the pads and the fact the casks are loaded individually onto the pads. The use of a single modulus of subgrade reaction (k_s) for a mat with a loading of this complexity will not generate realistic deflections. In order to obtain realistic deflections with the complex loading, the subgrade modulus values must be adjusted to account for wider loads. To address this issue, GEOServices has worked with the structural engineer (Enercon) to adjust the subgrade modulus through an iterative process. The process proceeded as follows:

1) The first iteration of the settlement analysis was performed using mat pressures provided by Enercon.
2) These pressures were used to develop a Settle3 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 pressure/settlement.
3) The resulting new 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 a much more accurate estimate of soil response (calculate k-values) thus refining the mat pressure distribution.
5) The results of the refined GTSTRUDL analysis were then provided and used to update the pressures in the Settle3 model. The result was an updated set of subgrade modulus for the entire mat.
6) This iterative process was continued until model convergence (calculated soil modulus values and displacements did not change more than 10 percent between consecutive iterations) was achieved.

The analysis was performed on the four loading configurations shown on Figure 7-9 in Chapter 7 of the SAR. These configurations include fully loaded, quarter loaded, half loaded, and three quarter loaded. Plots showing the converged models and the subsequent subgrade modulus values are provided in Appendix H of this document.

4.3.3.2 CISF Settlement Analysis

Settlement analysis for the CISF pads was performed using the soil column outlined in Appendix D. The settlement calculation was performed utilizing Settle3 a finite difference software produced by RocScience. Settle3 allows for the input of the foundation loads for the entire footprint so that any stress overlaps between adjacent foundations can be analyzed. For the CISF pads, the final pressures from the iterative process (designed to provide an accurate estimate of soils response beneath the mat) were utilized. For the CISF pads, a settlement distribution for each of the four loading configurations shown of Figure 7-9 in Chapter of SAR is provided. Additionally, due to the number of pads which will be installed in the CISF Area (18 total) and the spacing between pads (20 feet edge to edge) an additional case including four pads was analyzed. This analysis was performed to take into account any stress overlap between adjacent pads and provide the impacts in terms of settlement. In each instance the maximum total settlements were less than 0.75 inches. The comprehensive results of the analysis are provided in Appendix H of this document.

4.3.4 Shear Wave Testing Results

We evaluated the site seismic class of the upper 100 feet to determine the seismic site class per the criteria in Table 1613.5.2 of the International Building Code (IBC, 2006/2012). The on-site shear wave velocity was determined using the refraction micro-tremor (ReMi) method. The testing used a Seismic Source DAQ Link II 24 seismograph and 10 Hz vertical geophones. The geophones were deployed along an approximately 300-foot long linear array and spaced on approximately 26-foot centers. Once the field data was collected, a computer model was used to determine the subsurface shear wave velocity profile. The test results are attached to this report.

The attached seismic velocity model displays the shear-wave velocity profiles for the upper 100 feet. The results of the models revealed the following shear wave velocities.

Depth (feet)	Run 1 Shear Wave Velocity (feet/sec)	Run 2 Shear Wave Velocity (feet/sec)	Run 3 Shear Wave Velocity (feet/sec)	Run 4 Shear Wave Velocity (feet/sec)	Average Shear Wave Velocity (feet/sec)
0 - 5	820	1020	989	843	918
5 - 15	1107	985	978	1036	1027
15 - 25	1498	1302	1549	1432	1445
25 - 35	1498	2253	2120	1889	1940
35 - 55	2558	2731	2252	2058	2400
55 - 75	2228	1231	1417	2153	1757
75 - 100	2228	3205	3383	3322	3035

Table 5 – Summary of Shear Wave Velocity Results

The location of each of the shear wave arrays as well as the plot for each individual run is provided in Appendix E of this report.

4.3.5 Liquefaction Potential

Liquefaction occurs when soil, primarily saturated cohesionless soils, undergo a loss in strength due to monotonic, transient, or repeated disturbance that commonly occurs during a seismic event (Kramer 1996). This loss of strength occurs due to increased pore water pressures caused by an undrained condition. The increase in pore water pressure decreases the effective stress in the soil, thus reducing the soils ability to support any applied loads. For liquefaction to occur, there must be an increase in poor pressure meaning the soil must be saturated and be able to behave in an undrained condition. According to the NHI 2011 Reference Manual on LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, if any of the following criteria are satisfied then a significant liquefaction hazard does not exist:

- The geologic materials underlying the site are either bedrock or have very low liquefaction susceptibility according to the relative susceptibility ratings shown in the Estimated Susceptibility of Sedimentary Deposits to Liquefaction During Strong Ground Motion table presented by Youd and Perkins in 1978.
- The soils below the groundwater table at the site are one of the following
  - Clayey soils which have a clay content greater than 15%, liquid limit greater than 35%, or natural water content less than 90% of the liquid limit.
  - Sand with a minimum corrected SPT $(N_1)_{60}$ value of 30 blows/foot.
  - The water table is deeper than 50 feet below the ground surface or proposed finished grade at the site.

Since groundwater was not encountered in any of the eighteen soil test borings and given that some of the borings penetrated as deep as 45 feet below the ground surface, it can be concluded that a liquefaction hazard does not exist for the subject development.

4.3.6 Seismic Densification Analysis

While a liquefaction hazard does not exist for the subject project, there is a potential for settlement of the loose sands that exist in some areas of the CISF. According to Kramer (1996), the tendency of sands to densify when subjected to earthquake shaking is well documented and occurs very rapidly. This densification is usually completed by the end of the earthquake.

Calculations were performed to determine the magnitudes of settlements/densification that could occur during an earthquake event using the Pradel method. The calculations show that the seismic densification for the design earthquake will be negligible (on the order of 0.02 inches or less). Detailed information regarding the calculation, results, and procedure can be found in Appendix F.

4.4 LATERAL EARTH PRESSURES

At this time, we are not aware of planned retaining walls for the CISF project, however, we understand that some foundations may bear as deep as 10 to 11 feet below grade. Therefore, we are providing soil parameters and earth pressure coefficients for the materials we expect to be encountered on site as well as the potential backfill materials.

Earth Pressure Condition	Backfill Type	Unit Weight (pcf)	Friction Angle (deg.)	Earth Pressure Coefficient
Active (Ka)	Silty Sands	95	27	0.376
	Caliche	130	35	0.271
At-Rest (Ko)	Silty Sands	95	27	0.546
	Caliche	130	35	0.426
Passive (Kp)	Silty Sands	95	27	2.663
	Caliche	130	35	3.690

Table 6 – Earth Pressure Summary

Note: In each instance the earth pressure coefficients provided are unfactored.

For rigid, cast-in-place concrete walls, a friction factor of 0.45 between foundation concrete and the bearing soils may be used when evaluating friction. If a stone leveling course is utilized beneath the foundation, a friction factor of 0.55 between foundation concrete and the dense graded aggregate base may be used when evaluating friction.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 EXCAVATIONS

Auger refusal materials were encountered in four of the 18 soil test borings (B-101, B-111, TF-1, and TF-4) at depths ranging from 37 to 45 feet below the existing ground surface elevation. Typically, soils penetrated by augers can be removed with conventional earthmoving equipment. However, excavation equipment varies, and field refusal conditions may vary. Some of the very dense caliche may require difficult excavation techniques such as ripping, prior to excavation.

Excavations should be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards. The contractor is usually solely responsible for site safety. This information is provided only as a service and under no circumstances should GEOServices be assumed to be responsible for construction site safety.

5.2 FOUNDATION CONSTRUCTION

Foundation excavations should be opened, the subgrade evaluated, remedial work performed, and concrete placed in an expeditious manner. Exposure to weather often reduces foundation support capabilities, thus necessitating remedial measures prior to concrete placement. It is also important that proper surface drainage be maintained both during construction (especially in terms of maintaining dry footing trenches) and after construction. Soil backfill for footings should be placed in accordance with the recommendations for structural fill presented herein.

Foundation subgrade observations should be performed by a GEOServices geotechnical engineer, or his qualified representative, so that the recommendations provided in this report are consistent with the site conditions encountered. A dynamic cone penetrometer (DCP) is commonly utilized to provide information that is compared to the data obtained in the geotechnical report. Where unacceptable materials are encountered, the material should be excavated to stiff, suitable soils or remediated at the geotechnical engineer's direction. Typical remedial measures consist of undercutting, overexcavation, or combinations thereof.

5.3 MOISTURE SENSITIVE SOILS

The upper fine-grained soils encountered at this site will be sensitive to disturbances caused by construction traffic and changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. Construction traffic patterns should be varied to prevent the degradation of previously stable subgrade.

In addition, soils which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather. Climate data for Andrews, Texas, obtained from Weatherbase indicate in the following table the average monthly precipitation. The average amount of precipitation does not vary much throughout the year.

PRECIPITATION AVERAGES

Month	Monthly Precipitation Average (Inches)	Month	Monthly Precipitation Average (Inches)
January	0.7	July	1.9
February	0.5	August	1.5
March	0.3	September	1.5
April	0.9	October	1.8
May	2.1	November	0.4
June	1.6	December	0.6

Table 7 – Average Monthly Precipitation

5.4 DRAINAGE AND SURFACE WATER CONCERNS

To reduce the potential for undercut activities, water should not be allowed to collect in the foundation excavations, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, subsurface water, or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the buildings and beneath the floor slab. The grades should be sloped away from the buildings and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the buildings.

6.0 LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. This report is for our geotechnical work only, and no environmental assessment efforts have been performed. The conclusions and recommendations contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the exploration. The nature and extent of variations between the borings will not become evident until construction. We recommend that GEOServices be retained to observe the project construction in the field. GEOServices cannot accept responsibility for conditions which deviate from those described in this report if not retained to perform construction observation and testing. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing. Also, if the scope of the project should change significantly from that described herein, these recommendations may have to be re-evaluated.

GEOServices, LLC, Geotechnical and Materials Engineers

APPENDICES

GEOServices, LLC, Geotechnical and Materials Engineers

APPENDIX A

Figures and Boring Logs

Notes:

1) Aerial Provided by: Google Earth Pro, (02/12/2014)

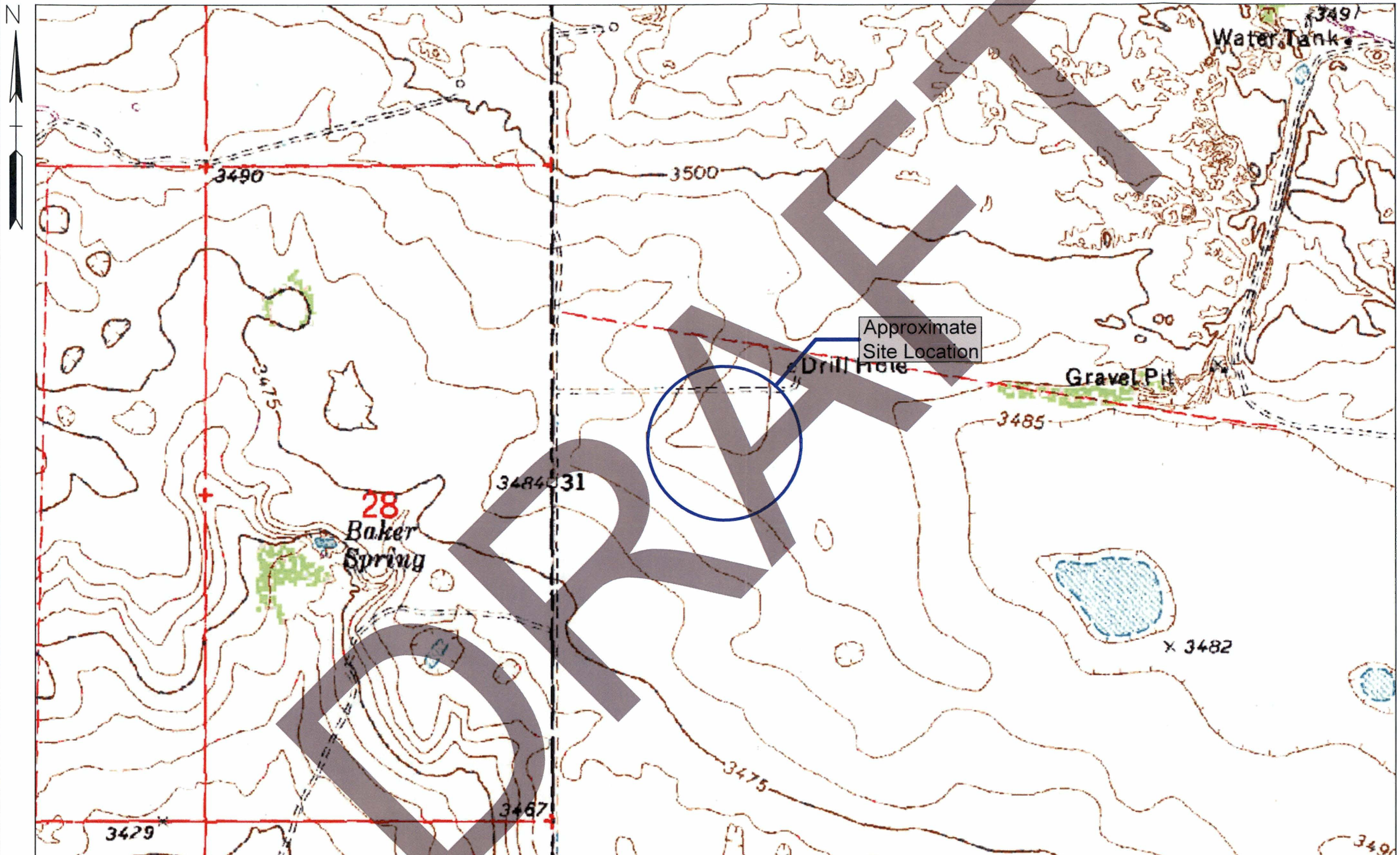
GEOServices, LLC-Geotechnical and Materials Engineers

Site Vicinity Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE:
08/10/2015

GEOS Project No.
31-151247

FIGURE:
1



Source Provided by: MYTOPO

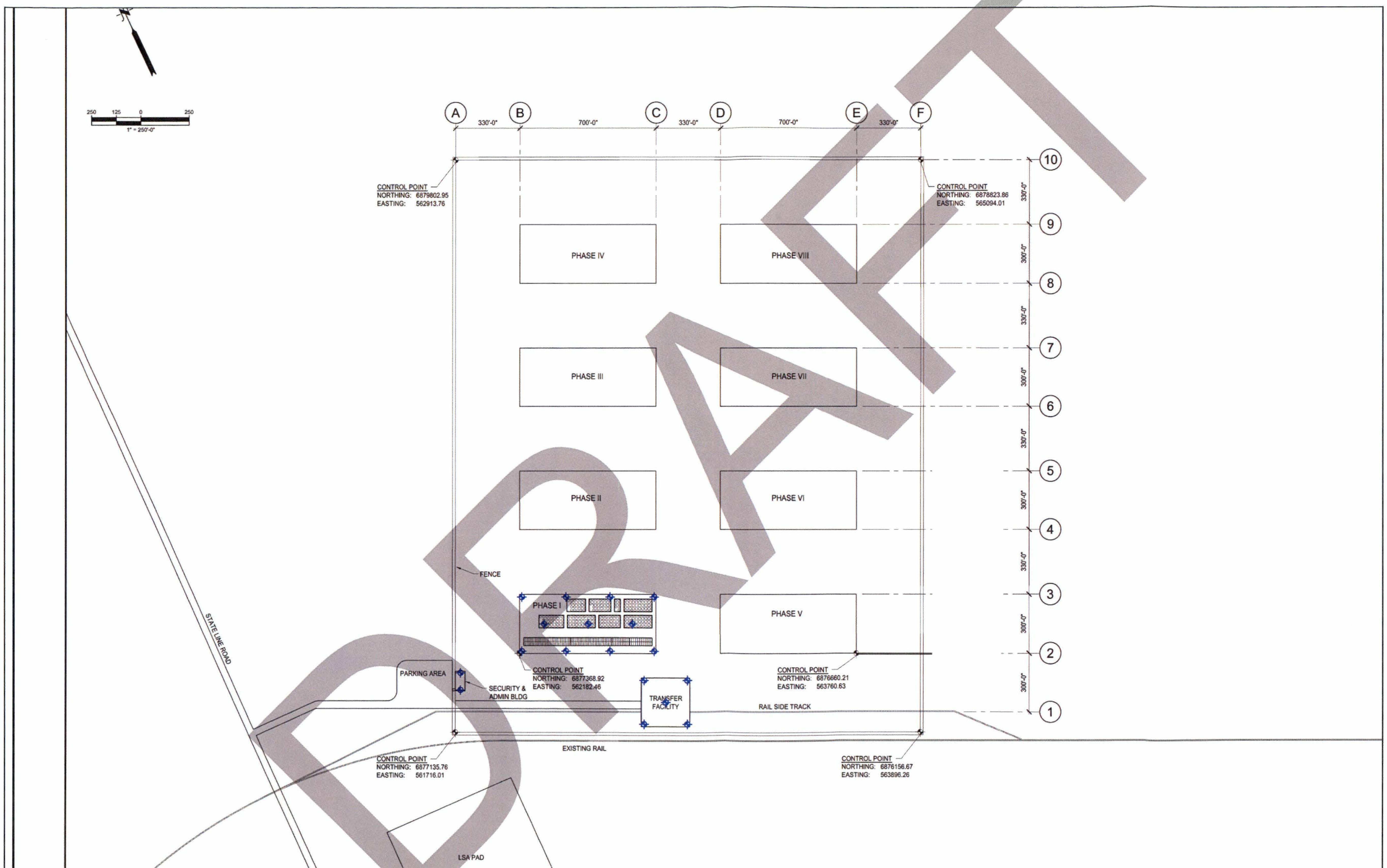
GEOServices, LLC-Geotechnical and Materials Engineers

USGS Site Vicinity Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE:
08/10/2015

GEOS Project No.
31-151247

FIGURE:
2



Notes:

1) Site Source Provided by: WCS, (06/12/2015)
2) Boring Locations are shown in general arrangement only
3) Do Not use Boring Locations for determinations of Distance or Quantities

Boring Location & Identifier

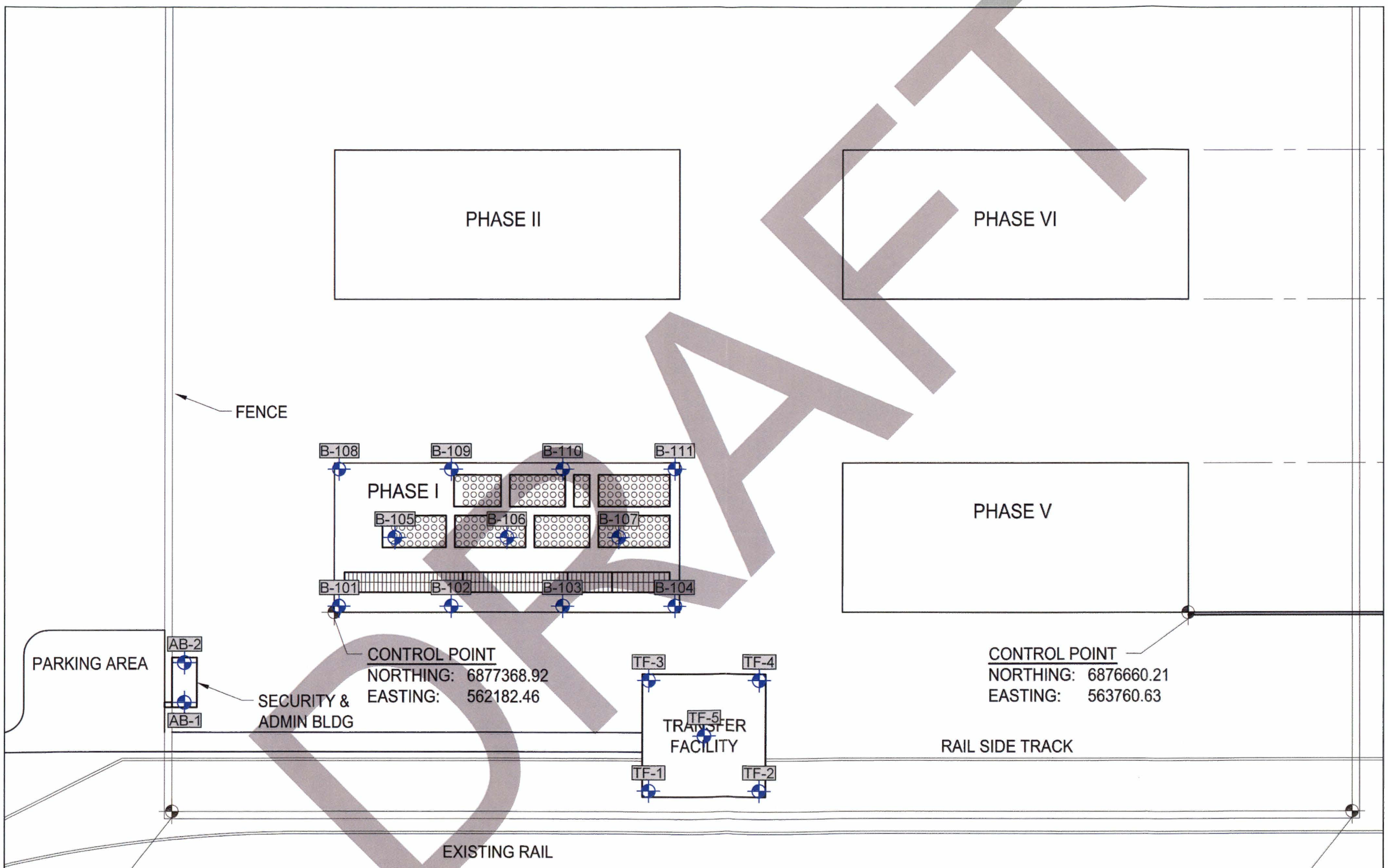
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Location Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 3



Notes:

1) Site Source Provided by: WCS, (06/12/2015)
2) Boring Locations are shown in general arrangement only
3) Do Not use Boring Locations for determinations of Distance or Quantities

Boring Location & Identifier

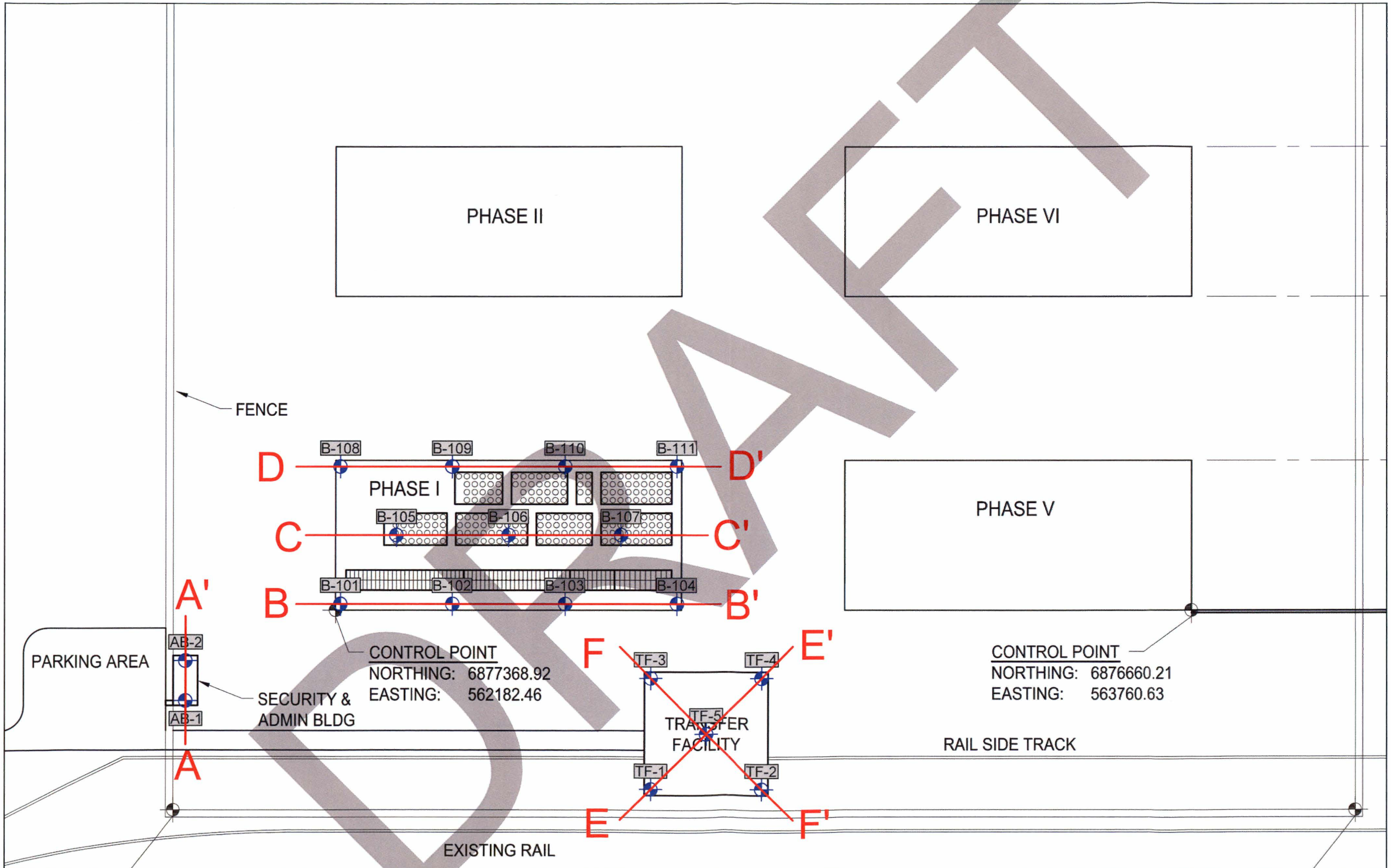
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Location Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 4



Notes:

1) Site Source Provided by: WCS, (06/12/2015)
2) Boring Locations are shown in general arrangement only
3) Do Not use Boring Locations for determinations of Distance or Quantities

Boring Location & Identifier

Cross Section line (Start - End')

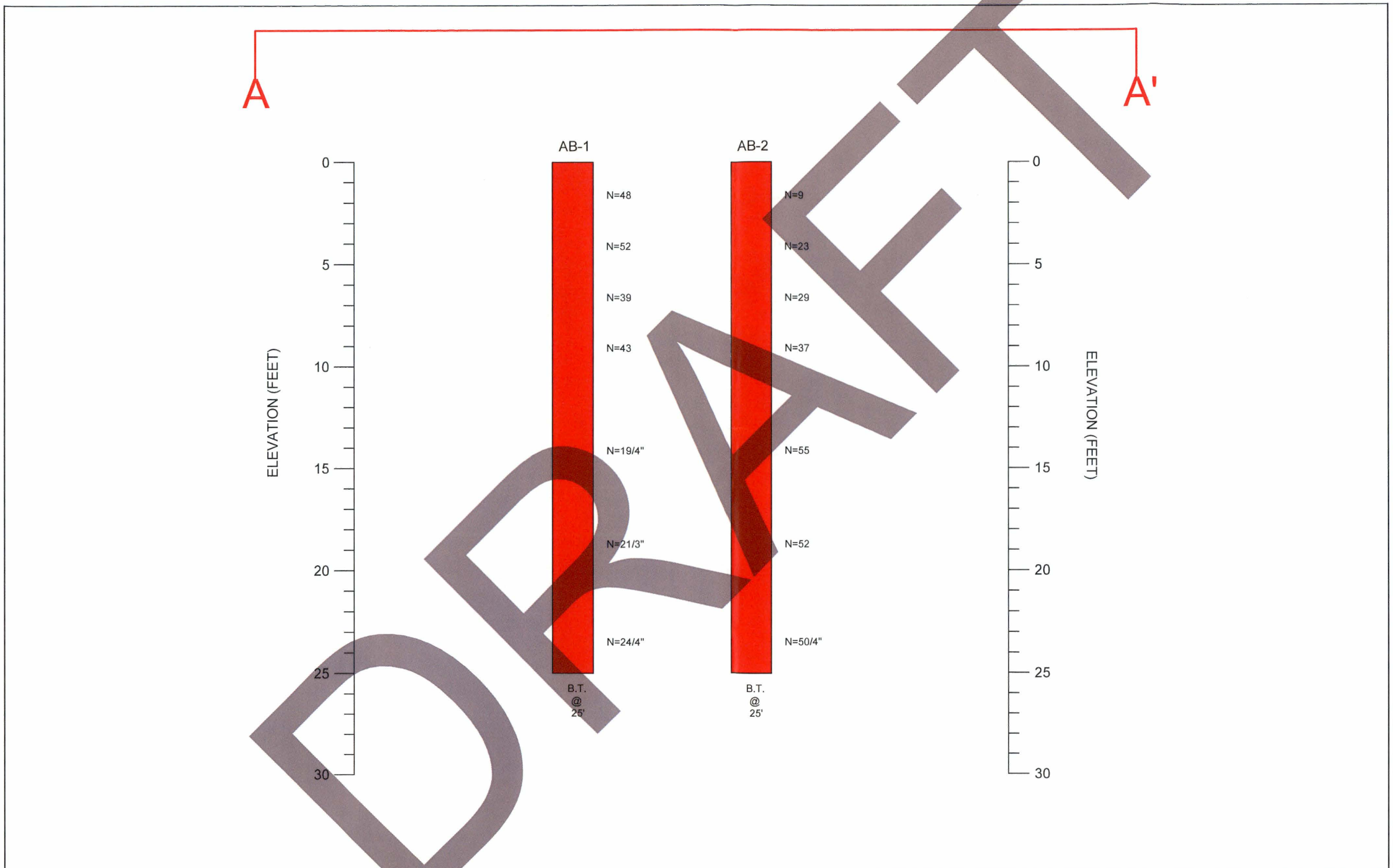
GEOServices, LLC-Geotechnical and Materials Engineers

Cross Section-Location Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 5

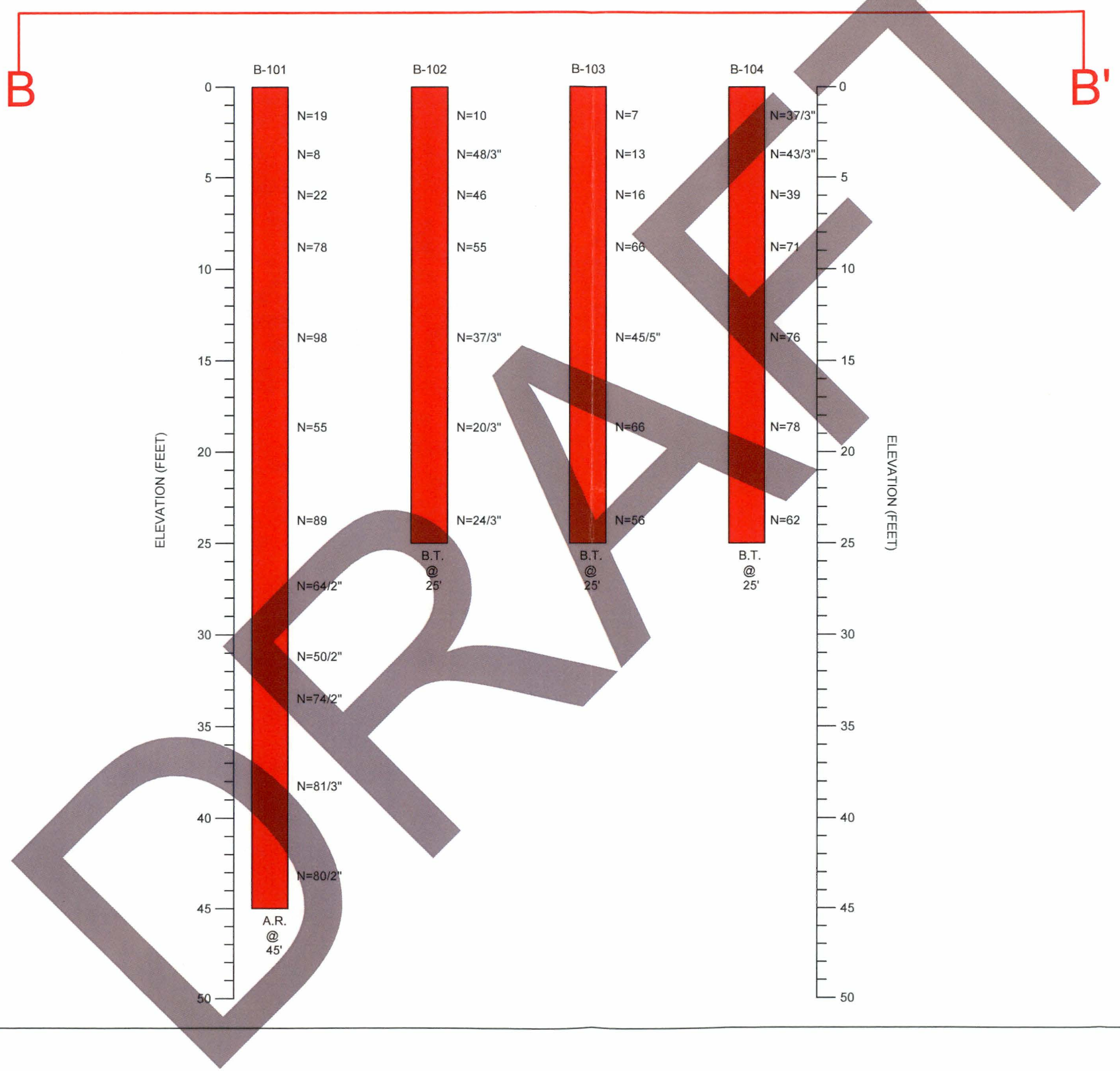


Notes:
1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015
GEOS Project No. 31-151247
FIGURE: 6



Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

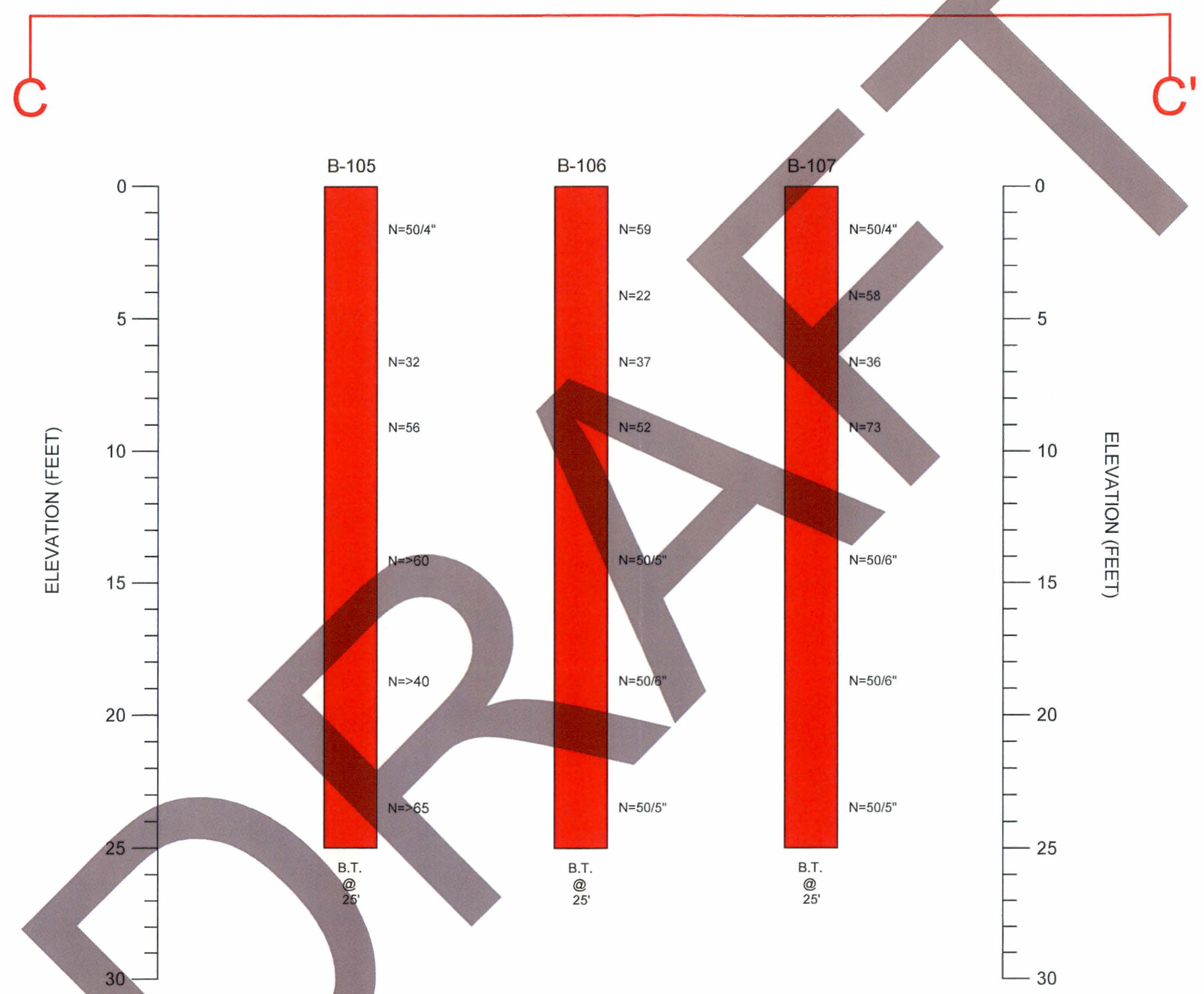
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 7



Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

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Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 8

Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

GEOS GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 9

Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 10

Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 11

GENERAL NOTES

FINE AND COARSE GRAINED SOIL PROPERTIES

PARTICLE SIZE

BOULDERS:	GREATER THAN 300 mm
COBBLES:	75 mm to 300 mm
GRAVEL:	4.74 mm to 75 mm
COARSE SAND:	2 mm to 4.74 mm
MEDIUM SAND:	0.425 mm to 2 mm
FINE SAND:	0.075 mm to 0.425 mm
SILTS & CLAYS:	LESS THAN 0.075 mm

COARSE GRAINED SOILS (SANDS & GRAVELS)

N-VALUE	RELATIVE DENSITY
0 - 4	VERY LOOSE
5 - 10	LOOSE
11 - 30	MEDIUM DENSE
31 - 50	DENSE
OVER 50	VERY DENSE

FINE GRAINED SOILS (SILTS & CLAYS)

N-VALUE	CONSISTENCY	Qu, PSF
0 - 2	VERY SOFT	0 - 500
3 - 4	SOFT	500 - 1000
5 - 8	FIRM	1000 - 2000
9 - 15	STIFF	2000 - 4000
16 - 30	VERY STIFF	4000 - 8000
OVER 31	HARD	8000 +

STANDARD PENETRATION TEST (ASTM D1586)

THE STANDARD PENETRATION TEST AS DEFINED BY ASTM D1586 IS A METHOD TO OBTAIN A DISTURBED SOIL SAMPLE FOR EXAMINATION AND TESTING AND TO OBTAIN RELATIVE DENSITY AND CONSISTENCY INFORMATON. THE 1.4 INCH I.D./2.0 INCH O.D. SAMPLER IS DRIVEN 3-SIX INCH INCREMENTS WITH A 140 LB. HAMMER FALLING 30 INCHES. THE BLOW COUNTS REQUIRED TO DRIVE THE SAMPLER THE FINAL 2 INCREMENTS ARE ADDED TOGETHER AND DESIGNATED THE N-VALUE. AT TIMES, THE SAMPLER CAN NOT BE DRIVEN THE FULL 18 INCHES. THE FOLLOWING REPRESENTS OUR INTERPRETATION OF THE STANDARD PENETRATION TEST WITH VARIATIONS.

BLOWS/FOOT (N-VALUE)	DESCRIPTION
25	25 BLOWS DROVE SAMPLER 12" AFTER INITIAL 6" SEATING
75/10"	75 BLOWS DROVE SAMPLER 10" AFTER INITIAL 6" SEATING
50/PR	PENETRATION REFUSAL OF SAMPLER AFTER INITIAL 6" SEATING

SAMPLING SYMBOLS

ST:	UNDISTURBED SAMPLE
SS:	SPLIT SPOON SAMPLE
CORE:	ROCK CORE SAMPLE
AU:	AUGER OR BAG SAMPLE

SOIL PROPERTY SYMBOLS

N:	STANDARD PENETRATION, BPF
M:	MOISTURE CONTENT %
LL:	LIQUID LIMIT %
PI:	PLASTICITY INDEX %
Qp:	POCKET PENETROMETER VALUE, TSF
Qu:	UNCONFINED COMPRESSIVE STRENGTH, TSF
DUW:	DRY UNIT WEIGHT, PCF

ROCK PROPERTIES

ROCK QUALITY DESIGNATION (RQD)

PERCENT	QUALITY
90 TO 100	EXCELLENT
75 TO 90	GOOD
50 TO 75	FAIR
25 TO 50	POOR
0 TO 25	VERY POOR

ROCK HARDNESS

VERY SOFT:	ROCK DISINTEGRATES OR EASILY COMPRESSES TO TOUCH: CAN BE HARD TO VERY HARD SOIL.
SOFT:	ROCK IS COHERANT BUT BREAKS EASILY TO THUMB PRESSURE AT SHARP EDGES AND CRUMBLES WITH FIRM HAND PRESSURE.
MODERATELY HARD:	SMALL PIECES CAN BE BROKEN OFF ALONG SHARP EDGES BY CONSIDERABLE HARD THUMB PRESSURE: CAN BE BROKEN BY LIGHT HAMMER BLOWS.
HARD:	ROCK CAN NOT BE BROKEN BY THUMB PRESSURE, BUT CAN BE BROKEN BY MODERATE HAMMER BLOWS.
VERY HARD:	ROCK CAN BE BROKEN BY HEAVY HAMMER BLOWS.

GEOS

GEOServices, LLC-Geotechnical and Materials Engineers

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-101

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-101
DATE July 21, 2015 SURFACE ELEV. ____ FT.
REFUSAL: Yes DEPTH 45.0 FT. ELEV. ____ FT.
SAMPLED 45.0 FT. 13.7 M
TOP OF ROCK DEPTH 45.0 FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 45.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS.: DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	3-12-7 N=19				3.6	**Silty SAND (SM)** with caliche - brown; fine grained; loose and medium dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	5-5-3 N=8				3.9	
5.0	-5.0										
		5.0	6.5	3	SS	4-10-12 N=22				6.5	
7.5	-7.5										**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
		8.0	9.5	4	SS	17-30-48 N=78				7.8	
10.0	-10.0										
12.5	-12.5										
		13.0	14.5	5	SS	30-59-39 N=98				6.7	
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	20-24-31 N=55				5.7	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-101
SHEET 2 OF 3

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-101

DATE: July 21, 2015 | SURFACE ELEV. ______ FT.
REFUSAL: Yes | DEPTH 45.0 FT. | ELEV. ______ FT.
SAMPLED: 45.0 FT. | 13.7 M
TOP OF ROCK | DEPTH 45.0 FT. | ELEV. ______ FT.
BEGAN CORING | DEPTH ______ FT. | ELEV. ______ FT.
FOOTAGE CORED (LF) ______ FT.
BOTTOM OF HOLE DEPTH 45.0 FT. | ELEV. ______ FT.

BORING ADVANCED BY: POWER AUGERING X | WASHBORING ______

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. | ELEV. ______ FT.
AFTER 24 HRS: DEPTH N/A FT. | ELEV. ______ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										
		23.0	24.5	7	SS	23-34-55 N=89				5.7	
25.0	-25.0										
27.5	-27.5										
		28.0	28.7	8	SS	42-64/2" N=64/2"					
30.0	-30.0										**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
		31.0	31.2	9	SS	N=50/2"				5.7	
32.5	-32.5										
		33.0	33.2	10	SS	N=74/2"				7.7	
35.0	-35.0										
37.5	-37.5										
		38.0	38.3	11	SS	N=81/3"					
40.0	-40.0										

REMARKS:

GEOS GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices Project No.: 31-151247

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-101

DATE July 21, 2015 SURFACE ELEV. FT.
REFUSAL: Yes DEPTH 45.0 FT. ELEV. FT.
SAMPLED 45.0 FT. 13.7 M
TOP OF ROCK DEPTH 45.0 FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 45.0 FT. ELEV. FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. FT.
AFTER 24 HRS: DEPTH N/A FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
42.5	-42.5	43.0	43.2	12	SS	N=80/2"					**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
45.0	-45.0										**AUGER REFUSAL AT 45 FEET**
47.5	-47.5										
50.0	-50.0										
52.5	-52.5										
55.0	-55.0										
57.5	-57.5										
60.0	-60.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-102

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-102
DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. ____ FT.
AFTER 24 HRS: DEPTH N/A FT.
ELEV. ____ FT.

DRAFT

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	LABORATORY RESULTS Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	7-6-4 N=10				3.4	**Silty SAND (SM)** with caliche - brown; fine grained; loose to very dense; dry
2.5	-2.5	3.0	3.8	2	SS	10-48/3" N=48/3"				2.8	
5.0	-5.0	5.0	6.5	3	SS	16-26-20 N=46				2.9	**Silty SAND (SM)** with caliche - light brown; fine grained; dense to very dense; dry
7.5	-7.5	8.0	9.5	4	SS	25-28-27 N=55				4.0	
10.0	-10.0										
12.5	-12.5	13.0	13.8	5	SS	34-37/3" N=37/3"				6.0	**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5	18.0	18.8	6	SS	43-20/3" N=20/3"				5.8	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-102
SHEET 2 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-102

DATE: July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED: 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
		23.0	23.8	7	SS	26-24/3" N=24/3"				2.9	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-103
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-103
DATE July 17, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. ____ FT.
AFTER 24 HRS: DEPTH N/A FT.
ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	2-3-4 N=7		26	NP	4.3	**Silty SAND (SM)** with caliche - brown; fine grained; loose; dry
2.5	-2.5										
		3.0	4.5	2	SS	3-5-8 N=13				4.8	
5.0	-5.0										
		5.0	6.5	3	SS	7-7-9 N=16				5.8	
7.5	-7.5										
		8.0	9.5	4	SS	16-27-39 N=66				7.1	**Silty SAND (SM)** with caliche - light brown; fine grained; medium dense to very dense; dry
10.0	-10.0										
12.5	-12.5										
		13.0	14.4	5	SS	23-35-45/5" N=45/5"				6.3	
15.0	-15.0										
17.5	-17.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		18.0	19.5	6	SS	17-26-40 N=66				6.1	
20.0	-20.0										

REMARKS:

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices, LLC-Geotechnical and Materials Engineers

GEOServices Project No.: 31-151247

LOG OF BORING B-103

SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.

DRILLER

LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-103

DATE July 17, 2015 SURFACE ELEV. FT.

REFUSAL: No DEPTH FT. ELEV. FT.

SAMPLED 25.0 FT. 7.6 M

TOP OF ROCK DEPTH FT. ELEV. FT.

BEGAN CORING DEPTH FT. ELEV. FT.

FOOTAGE CORED (LF) FT.

BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION: DEPTH Dry FT. ELEV. FT.

AFTER 24 HRS: DEPTH N/A FT. ELEV. FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	17-23-33 N=56				4.5	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices Project No.: 31-151247

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-104
DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.3	1	SS	2-32-37/3" N=37/3"				3.1	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	3.8	2	SS	64-43/3" N=43/3"				2.5	
5.0	-5.0										
		5.0	6.5	3	SS	13-18-21 N=39				4.1	**Silty SAND (SM)** with caliche - light brown; fine grained; very dense; dry
7.5	-7.5										
		8.0	9.5	4	SS	17-26-45 N=71				6.9	
10.0	-10.0										
12.5	-12.5										
		13.0	14.5	5	SS	18-33-43 N=76				6.0	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	23-28-50 N=78				4.6	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-104
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-104
DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS: DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	15-23-39 N=62				3.1	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices Project No.: 31-151247

LOG OF BORING B-105

DRILLING CO Apex Geoscience Inc.

DRILLER

LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-105

DATE July 15, 2015 SURFACE ELEV. ____ FT.

REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.

SAMPLED 25.0 FT. 7.6 M

TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.

BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.

FOOTAGE CORED (LF) ____ FT.

BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION: DEPTH Dry FT. ELEV. ____ FT.

AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	1.8	1	SS	6-50/4" N=50/4"				3.3	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
5.0	-5.0	5.0	6.5	2	SS	10-14-18 N=32				5.3	
7.5	-7.5	8.0	9.5	3	SS	14-27-29 N=56				8.1	
10.0	-10.0										**Silty SAND (SM)** with caliche - orangish brown; fine grained; dense to very dense; dry
12.5	-12.5	13.0	13.5	4	SS	60+ N=>60				4.8	
15.0	-15.0										
17.5	-17.5	18.0	19.0	5	SS	30-40+ N=>40				4.1	
20.0	-20.0										

REMARKS:

GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-105
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-105

DATE July 16, 2015 SURFACE ELEV. ___ FT.
REFUSAL: No DEPTH ___ FT. ELEV. ___ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ___ FT. ELEV. ___ FT.
BEGAN CORING DEPTH ___ FT. ELEV. ___ FT.
FOOTAGE CORED (LF) ___ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ___ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. ___ FT.
AFTER 24 HRS: DEPTH N/A FT.
ELEV. ___ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ___

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	LABORATORY RESULTS Qp	LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with caliche - orangish brown; fine grained; dense to very dense; dry
		23.0	23.5	6	SS	65+ N=>65					
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-106
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-106

DATE	July 16, 2015			SURFACE ELEV.	FT.
REFUSAL:	No	DEPTH	FT.	ELEV.	FT.
SAMPLED	25.0 FT.	7.6 M			
TOP OF ROCK		DEPTH	FT.	ELEV.	FT.
BEGAN CORING		DEPTH	FT.	ELEV.	FT.
FOOTAGE CORED (LF)			FT.		
BOTTOM OF HOLE DEPTH		25.0	FT.	ELEV.	FT.

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION:	DEPTH	Dry	FT.
	ELEV.		FT.
AFTER 24 HRS:	DEPTH	N/A	FT.
	ELEV.		FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	13-33-26 N=59				4.5	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	9-14-8 N=22				3.9	**Silty SAND (SM)** with trace caliche - light brown; fine grained; medium dense to very dense; dry
5.0	-5.0										
		5.0	6.5	3	SS	11-16-21 N=37				3.9	
7.5	-7.5										
		8.0	9.5	4	SS	7-19-33 N=52				5.3	
10.0	-10.0										
12.5	-12.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		13.0	13.9	5	SS	33-50/5" N=50/5"				6.7	
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	25-35-50/6" N=50/6"				6.8	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-106
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-106

DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

Stratum Depth FT.	Stratum Depth ELEV.	Sample Depth FROM FT.	Sample Depth TO FT.	Sample or Run No.	Sample Type	Field Results N-Value	Qp	LL	PI	%M	Stratum Description
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.4	7	SS	12-37-50/5" N=50/5"				6.2	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-107
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-107

DATE	July 16, 2015			SURFACE ELEV.	FT.
REFUSAL:	No	DEPTH	FT.	ELEV.	FT.
SAMPLED	25.0 FT.	7.6 M			
TOP OF ROCK		DEPTH	FT.	ELEV.	FT.
BEGAN CORING		DEPTH	FT.	ELEV.	FT.
FOOTAGE CORED (LF)			FT.		
BOTTOM OF HOLE DEPTH		25.0	FT.	ELEV.	FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. FT.
AFTER 24 HRS: DEPTH N/A FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	1.8	1	SS	7-50/4" N=50/4"				5.3	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	7-25-33 N=58				4.5	
5.0	-5.0										
		5.0	6.5	3	SS	9-16-20 N=36				4.1	**Silty SAND (SM)** with trace caliche - light brown; fine grained; very dense; dry
7.5	-7.5										
		8.0	9.5	4	SS	22-32-41 N=73				5.7	
10.0	-10.0										
12.5	-12.5										
		13.0	14.0	5	SS	31-50/6" N=50/6"				7.2	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	16-32-50/6" N=50/6"				6.3	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-107
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-107
DATE July 16, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.4	7	SS	15-32-50/5" N=50/5"				4.2	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

GEOServices, LLC-Geotechnical and Materials Engineers

LOG OF BORING B-108
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-108
DATE: July 15, 2015 — SURFACE ELEV. ___ FT.
REFUSAL: No — DEPTH ___ FT. — ELEV. ___ FT.
SAMPLED: 25.0 FT. 7.6 M
TOP OF ROCK — DEPTH ___ FT. — ELEV. ___ FT.
BEGAN CORING — DEPTH ___ FT. — ELEV. ___ FT.
FOOTAGE CORED (LF) ___ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. — ELEV. ___ FT.
BORING ADVANCED BY: POWER AUGERING X — WASHBORING ___

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ___ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ___ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	1-5-10 N=15				4.9	**Silty SAND (SM)** with caliche - brown to light brown; fine grained; medium dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	2-9-16 N=25				4.9	
5.0	-5.0										
		5.0	6.5	3	SS	10-17-20 N=37				6.0	**Silty SAND (SM)** with caliche - orangish brown; fine grained; dense to very dense; dry
7.5	-7.5										
		8.0	9.4	4	SS	14-32-23/5" N=23/5"				6.3	
10.0	-10.0										
12.5	-12.5										
		13.0	13.8	5	SS	22-30/4" N=30/4"				8.4	
15.0	-15.0										
17.5	-17.5										
		18.0	18.8	6	SS	38-39/3" N=39/3"				6.6	
20.0	-20.0										

REMARKS:

GEOS

GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices Project No.: 31-151247

LOG OF BORING B-108

SHEET 2 OF 2

DRILLING CO: Apex Geoscience Inc.

DRILLER:

LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-108

DATE: July 15, 2015 | SURFACE ELEV. ____ FT.

REFUSAL: No | DEPTH ____ FT. | ELEV. ____ FT.

SAMPLED: 25.0 FT. | 7.6 M

TOP OF ROCK | DEPTH ____ FT. | ELEV. ____ FT.

BEGAN CORING | DEPTH ____ FT. | ELEV. ____ FT.

FOOTAGE CORED (LF) ____ FT.

BOTTOM OF HOLE DEPTH 25.0 FT. | ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X | WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION: DEPTH Dry FT. | ELEV. ____ FT.

AFTER 24 HRS. DEPTH N/A FT. | ELEV. ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qp	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5	23.0	23.4	7	SS	N=58/5"				6	**Silty SAND (SM)** with caliche - orangish brown; fine grained; dense to very dense; dry
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices Project No.: 31-151247

LOG OF BORING B-109

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-109

DATE July 15, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	3-31-23 N=54				3.4	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	6-15-24 N=39				4.2	
5.0	-5.0										
		5.0	6.5	3	SS	10-18-24 N=42				3.8	**Silty SAND (SM)** with caliche - light brown; fine grained; dense to very dense; dry
7.5	-7.5										
		8.0	9.0	4	SS	29-40+ N=>40				3.2	
10.0	-10.0										
12.5	-12.5										
		13.0	13.3	5	SS	47/3" N=47/3"				8.5	**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	18.7	6	SS	42-40+ N=>40				6.3	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-109
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-109
DATE July 15, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5	23.0	24.5	7	SS	32-33-30 N=63				4.7	Silty SAND (SM) with caliche - orangish brown; fine grained; very dense; dry
25.0	-25.0										BORING TERMINATED AT 25 FEET
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-110
SHEET 1 OF 2
DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-110
DATE July 16, 2015 SURFACE ELEV. ___ FT.
REFUSAL: No DEPTH ___ FT. ELEV. ___ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ___ FT. ELEV. ___ FT.
BEGAN CORING DEPTH ___ FT. ELEV. ___ FT.
FOOTAGE CORED (LF) ___ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ___ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ___

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ___ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ___ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	4-27-16 N=43		29	NP	5.0	**Silty SAND (SM)** with caliche - brown; fine grained; dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	14-18-18 N=36				4.7	
5.0	-5.0	5.0	6.5	3	SS	8-7-6 N=15				6.0	
7.5	-7.5	8.0	9.5	4	SS	9-14-28 N=42				8.9	**Silty SAND (SM)** with caliche - light brown; fine grained; medium dense to very dense; dry
10.0	-10.0										
12.5	-12.5	13.0	14.0	5	SS	23-50+ N=>50				7.1	
15.0	-15.0										
17.5	-17.5	18.0	19.0	6	SS	36-40+ N=>40				5.1	**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
20.0	-20.0										

REMARKS:

Consolidated Interim Storage Facility
Andrews, Texas

GEOServices, LLC-Geotechnical and Materials Engineers

GEOServices Project No.: 31-151247

LOG OF BORING B-110

SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION B-110

DATE July 16, 2015 SURFACE ELEV. FT.

REFUSAL: No DEPTH FT. ELEV. FT.

SAMPLED 25.0 FT. 7.6 M

TOP OF ROCK DEPTH FT. ELEV. FT.

BEGAN CORING DEPTH FT. ELEV. FT.

FOOTAGE CORED (LF) FT.

BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION: DEPTH Dry FT. ELEV. FT.

AFTER 24 HRS. DEPTH N/A FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qp	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	20-32-52 N=84				4.4	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-111
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-111

DATE: July 20, 2015 SURFACE ELEV. ____ FT.
REFUSAL: Yes DEPTH 37.0 FT. ELEV. ____ FT.
SAMPLED: 37.0 FT. 11.3 M
TOP OF ROCK DEPTH 37.0 FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 37.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

DRAFT

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	5-24-65 N=89				6.7	**Silty SAND (SM)** with caliche - brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	15-16-12 N=28				4.5	**Silty SAND (SM)** with trace caliche - light brown; fine grained; medium dense to very dense; dry
5.0	-5.0										
		5.0	6.5	3	SS	12-19-33 N=52				5.2	
7.5	-7.5										
		8.0	9.5	4	SS	14-19-32 N=51				4.3	
10.0	-10.0										
12.5	-12.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		13.0	14.0	5	SS	35-60+ N=>60				6.6	
15.0	-15.0										
17.5	-17.5										
		18.0	19.0	6	SS	32-90+ N=>90				5.8	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING B-111

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: B-111

DATE: July 20, 2015 — SURFACE ELEV. ____ FT.
REFUSAL: Yes — DEPTH 37.0 FT. — ELEV. ____ FT.
SAMPLED: 37.0 FT. — 11.3 M
TOP OF ROCK: DEPTH 37.0 FT. — ELEV. ____ FT.
BEGAN CORING: DEPTH ____ FT. — ELEV. ____ FT.
FOOTAGE CORED (LF): ____ FT.
BOTTOM OF HOLE DEPTH: 37.0 FT. — ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X — WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. — ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. — ELEV. ____ FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	FIELD RESULTS Qp	LABORATORY RESULTS LL	LABORATORY RESULTS PI	LABORATORY RESULTS %M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										
		23.0	24.0	7	SS	37-65+ N=>65				6	
25.0	-25.0										
27.5	-27.5										
		28.0	29.5	8	SS	34-33-33 N=66				4.3	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
30.0	-30.0										
32.5	-32.5										
		33.0	34.5	9	SS	24-31-48 N=79				2.5	
35.0	-35.0										
											AUGER REFUSAL AT 37 FEET
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING AB-1
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION AB-1

DATE July 15, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	20-32-16 N=48				3.3	**Silty SAND (SM)** with caliche - brown; fine grained; dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	22-30-22 N=52				3.9	**Silty SAND (SM)** with caliche - light brown; fine grained; very dense; dry
5.0	-5.0										
		5.0	6.5	3	SS	13-20-19 N=39				6.7	
7.5	-7.5										
		8.0	9.5	4	SS	13-19-24 N=43				6.2	
10.0	-10.0										**Silty SAND (SM)** with caliche - orangish brown; fine grained; dense to very dense; dry
12.5	-12.5										
		13.0	14.3	5	SS	16-19-19/4" N=19/4"				4.3	
15.0	-15.0										
17.5	-17.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		18.0	18.8	6	SS	29-21/3" N=21/3"				4.9	
20.0	-20.0										

REMARKS: ____

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION AB-1
DATE July 15, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	23.8	7	SS	26-24/4" N=24/4"				3.2	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING AB-2
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION AB-2

DATE July 13, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT.
ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	4-4-5 N=9				4.8	**Silty SAND (SM)** with caliche - brown; fine grained; loose; dry
2.5	-2.5										
		3.0	4.5	2	SS	10-10-13 N=23				5.7	**Silty SAND (SM)** with caliche - light brown; fine grained; medium dense; dry
5.0	-5.0										
		5.0	6.5	3	SS	11-12-17 N=29				5.5	
7.5	-7.5										
		8.0	9.5	4	SS	10-17-20 N=37				3.7	**Silty SAND (SM)** with trace caliche - orangish brown; dense; dry
10.0	-10.0										
12.5	-12.5										
		13.0	14.5	5	SS	7-25-30 N=55				3.4	**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	16-22-30 N=52				3.1	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING AB-2
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION AB-2

DATE July 13, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with caliche - orangish brown; fine grained; very dense; dry
		23.0	23.8	7	SS	38-50/4" N=50/4"					
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-1
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: TF-1

DATE: July 16, 2015 | SURFACE ELEV. ____ FT.
REFUSAL: Yes | DEPTH 40.0 FT. | ELEV. ____ FT.
SAMPLED: 40.0 FT. | 12.2 M
TOP OF ROCK | DEPTH 40.0 FT. | ELEV. ____ FT.
BEGAN CORING | DEPTH ____ FT. | ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. | ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. | ELEV. ____ FT.
AFTER 24 HRS: DEPTH N/A FT. | ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X | WASHBORING ____

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
2.5	-2.5	1.0	2.5	1	SS	4-24-15 N=39				3.0	**Silty SAND (SM)** with caliche - grayish brown and brown; fine grained; dense; dry
5.0	-5.0	3.0	4.5	2	SS	24-38-27 N=65				2.8	**Silty SAND (SM)** with caliche -light brown to brown; fine grained; very dense to dense; dry
7.5	-7.5	5.0	6.5	3	SS	11-12-20 N=32				4.8	
10.0	-10.0	8.0	9.5	4	SS	20-20-28 N=48				2.8	
12.5	-12.5	13.0	14.3	5	SS	20-48-32/3" N=32/3"				5.8	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5	18.0	19.3	6	SS	26-48-26/3" N=26/3"				5.4	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-1

DRILLING CO: Apex Geoscience Inc.
DRILLER:

BORING NO. / LOCATION: TF-1
LOGGED BY: Sammy Joe Allison

DATE: July 21, 2015
SURFACE ELEV. ____ FT.
REFUSAL: Yes DEPTH 40.0 FT. ELEV. ____ FT.
SAMPLED 40.0 FT. 12.2 M
TOP OF ROCK DEPTH 40.0 FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

DRAFT

Stratum Depth FT.	Elev.	Sample Depth From FT.	Sample Depth To FT.	Sample or Run No.	Sample Type	Field Results N-Value	Qp	LL	PI	%M	Stratum Description
											CONTINUED
22.5	-22.5										
		23.0	24.5	7	SS	15-16-20 N=36				2.7	
25.0	-25.0										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
27.5	-27.5										
		28.0	29.3	8	SS	28-50-22/3" N=22/3"				3.0	
30.0	-30.0										
32.5	-32.5										
		33.0	33.4	10	SS	N=76/5"				3.7	
35.0	-35.0										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
37.5	-37.5										
		38.0	38.3	11	SS	N=100/4"				4.6	
40.0	-40.0										**AUGER REFUSAL AT 40 FEET**

REMARKS:

GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-2
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-2

DATE July 16, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT.
ELEV. FT.

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	1-2-2 N=4		28	NP	4.3	**Silty SAND (SM)** with trace caliche - brown; fine grained; very loose; dry
2.5	-2.5										
		3.0	4.5	2	SS	9-11-11 N=22				4.8	
5.0	-5.0										
		5.0	6.5	3	SS	8-10-15 N=25				3.9	**Silty SAND (SM)** with trace caliche - light brown; fine grained; medium dense to very dense; dry
7.5	-7.5										
		8.0	9.5	4	SS	15-40-57 N=97				6.7	
10.0	-10.0										
12.5	-12.5										
		13.0	14.2	5	SS	25-55-25/2" N=25/2"				5.9	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	18.8	6	SS	23-70+ N=>70				5.9	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-2
DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										Silty SAND (SM) with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	8-29-35 N=64				4.2	
25.0	-25.0										BORING TERMINATED AT 25 FEET
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-3
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-3
DATE July 16, 2015 SURFACE ELEV. ____ FT.
REFUSAL: No DEPTH ____ FT. ELEV. ____ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ____ FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ____ FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

STRATUM DEPTH FT.	STRATUM DEPTH ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	LABORATORY RESULTS Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	11-9-8 N=17				4.3	**Silty SAND (SM)** with caliche - grayish brown; fine grained; medium desne; dry
2.5	-2.5										
		3.0	4.5	2	SS	17-11-10 N=21				4.6	
5.0	-5.0										**Silty SAND (SM)** with caliche - brown; fine grained; very dense to dense; dry
		5.0	6.5	3	SS	14-16-20 N=36				3.9	
7.5	-7.5										
		8.0	9.5	4	SS	17-37-48 N=85				5.5	
10.0	-10.0										
12.5	-12.5										
		13.0	14.5	5	SS	18-28-45 N=73				9.0	**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	19.5	6	SS	17-33-50 N=83				4.5	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-3
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-3

DATE July 16, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X WASHBORING

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	34-37-60 N=97				3.8	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS:

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

LOG OF BORING TF-4
SHEET 1 OF 2

DRILLING CO: Apex Geoscience Inc.
DRILLER:
LOGGED BY: Sammy Joe Allison

BORING NO. / LOCATION: TF-4

DATE: July 16, 2015 — SURFACE ELEV. ____ FT.
REFUSAL: Yes — DEPTH 40.0 FT. — ELEV. ____ FT.
SAMPLED: 40.0 FT. 12.2 M
TOP OF ROCK — DEPTH ____ FT. — ELEV. ____ FT.
BEGAN CORING — DEPTH ____ FT. — ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. — ELEV. ____ FT.

BORING ADVANCED BY: POWER AUGERING X — WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. — ELEV. ____ FT.
AFTER 24 HRS: DEPTH N/A FT. — ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	4-40-20 N=60				3.7	**Silty SAND (SM)** with caliche - grayish brown; fine grained; very dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	18-18-16 N=34				3.9	
5.0	-5.0										**Silty SAND (SM)** with trace caliche - light brown; fine grained; dense to very dense; dry
		5.0	6.5	3	SS	20-20-30 N=50				6.6	
7.5	-7.5										
		8.0	9.5	4	SS	14-24-35 N=59				7.4	
10.0	-10.0										
12.5	-12.5										
		13.0	14.2	5	SS	20-35-47/2" N=47/2"				5.2	**Silty SAND (SM)** with trace caliche - brown; fine grained; very dense; dry
15.0	-15.0										
17.5	-17.5										
		18.0	19.2	6	SS	19-34-51/2" N=51/2"				4.3	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-4
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-4
DATE July 21, 2015 SURFACE ELEV. ____ FT.
REFUSAL: Yes DEPTH 40.0 FT. ELEV. ____ FT.
SAMPLED 40.0 FT. 12.2 M
TOP OF ROCK DEPTH 40.0 FT. ELEV. ____ FT.
BEGAN CORING DEPTH ____ FT. ELEV. ____ FT.
FOOTAGE CORED (LF) ____ FT.
BOTTOM OF HOLE DEPTH 40.0 FT. ELEV. ____ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ____

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ____ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ____ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5	23.0	24.5	7	SS	20-34-51 N=85				3.9	
25.0	-25.0										**Silty SAND (SM)** - orangish brown; fine grained; very dense; dry
27.5	-27.5	28.0	28.8	8	SS	62-47/3" N=47/3"				6.5	
30.0	-30.0										
32.5	-32.5	33.0	33.3	9	SS	N=100/4"				3.3	
35.0	-35.0										**Silty SAND (SM)** - orangish brown; fine grained; very dense; dry
37.5	-37.5	38.0	38.1	10	SS	N=100/1"				3.5	
40.0	-40.0										**AUGER REFUSAL AT 40 FEET**

REMARKS:

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

GEOServices, LLC-Geotechnical and Materials Engineers

LOG OF BORING TF-5
SHEET 1 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-5
DATE July 16, 2015 SURFACE ELEV. FT.
REFUSAL: No DEPTH FT. ELEV. FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT.
ELEV. FT.
AFTER 24 HRS. DEPTH N/A FT.
ELEV. FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
		1.0	2.5	1	SS	18-10-12 N=22				4.6	**Silty SAND (SM)** with caliche - grayish brown; fine grained; medium dense; dry
2.5	-2.5										
		3.0	4.5	2	SS	6-11-8 N=19				5.2	
5.0	-5.0										
		5.0	6.5	3	SS	2-2-3 N=5				5.5	**Silty SAND (SM)** with caliche - light brown; fine grained; medium dense to loose to very dense; dry
7.5	-7.5										
		8.0	9.5	4	SS	15-24-39 N=63				8.1	
10.0	-10.0										
12.5	-12.5										
		13.0	14.5	5	SS	26-46-60 N=106				6.4	
15.0	-15.0										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
17.5	-17.5										
		18.0	19.3	6	SS	22-39-58 N=97				5.0	
20.0	-20.0										

REMARKS:

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Consolidated Interim Storage Facility
Andrews, Texas
GEOServices Project No.: 31-151247

LOG OF BORING TF-5
SHEET 2 OF 2

DRILLING CO Apex Geoscience Inc.
DRILLER
LOGGED BY Sammy Joe Allison

BORING NO. / LOCATION TF-5
DATE July 16, 2015 SURFACE ELEV. ___ FT.
REFUSAL: No DEPTH ___ FT. ELEV. ___ FT.
SAMPLED 25.0 FT. 7.6 M
TOP OF ROCK DEPTH ___ FT. ELEV. ___ FT.
BEGAN CORING DEPTH ___ FT. ELEV. ___ FT.
FOOTAGE CORED (LF) ___ FT.
BOTTOM OF HOLE DEPTH 25.0 FT. ELEV. ___ FT.
BORING ADVANCED BY: POWER AUGERING X WASHBORING ___

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH Dry FT. ELEV. ___ FT.
AFTER 24 HRS. DEPTH N/A FT. ELEV. ___ FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qp	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											CONTINUED
22.5	-22.5										**Silty SAND (SM)** with trace caliche - orangish brown; fine grained; very dense; dry
		23.0	24.5	7	SS	24-41-45 N=86				4.2	
25.0	-25.0										**BORING TERMINATED AT 25 FEET**
27.5	-27.5										
30.0	-30.0										
32.5	-32.5										
35.0	-35.0										
37.5	-37.5										
40.0	-40.0										

REMARKS: ___

GEOServices, LLC, Geotechnical and Materials Engineers

APPENDIX B

Soil Laboratory Data

SOIL DATA SUMMARY Consolidated Interim Storage Facility - Andrews, Texas GEOServices Project No. 31-151247 August 5, 2015								
Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits			Finer than 200 Sieve (%)	Soil Type
				LL	PL	PI		
B-101	1	1.0 - 2.5	3.6%				48	
	2	3.0 - 4.5	3.9%					
	3	5.0 - 6.5	6.5%					
	4	8.0 - 9.5	7.8%				37	
	5	13.0 - 14.5	6.7%					
	6	18.0 - 19.5	5.7%					
	7	23.0 - 24.5	5.7%					
	9	31.0 - 31.2	5.7%					
	10	33.0 - 33.2	7.7%					
B-102	1	1.0 - 2.5	3.4%					
	2	3.0 - 3.8	2.8%					
	3	5.0 - 6.5	2.9%					
	4	8.0 - 9.5	4.0%					
	5	13.0 - 13.8	6.0%					
	6	18.0 - 18.8	5.8%					
	7	23.0 - 23.8	2.9%					
B-103	1	1.0 - 2.5	4.3%	26	N.P.			
	2	3.0 - 4.5	4.8%					
	3	5.0 - 6.5	5.8%					
	4	8.0 - 9.5	7.1%					
	5	13.0 - 14.4	6.3%					
	6	18.0 - 19.5	6.1%					
	7	23.0 - 24.5	4.5%					
B-104	1	1.0 - 2.3	3.1%					
	2	3.0 - 3.8	2.5%					
	3	5.0 - 6.5	4.1%					
	4	8.0 - 9.5	6.9%					
	5	13.0 - 14.5	6.0%					
	6	18.0 - 19.5	4.6%					
	7	23.0 - 24.5	3.1%					
B-105	1	1.0 - 1.8	3.3%					
	2	5.0 - 6.5	5.3%					
	3	8.0 - 9.5	8.1%					
	4	13.0 - 13.5	4.8%					
	6	18.0 - 19.0	4.1%					

SOIL DATA SUMMARY
Consolidated Interim Storage Facility - Andrews, Texas
GEOServices Project No. 31-151247
August 5, 2015

Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Finer than 200 Sieve (%)	Soil Type
B-106	1	1.0 - 2.5	4.5%					
	2	3.0 - 4.5	3.9%					
	3	5.0 - 6.5	3.9%					
	4	8.0 - 9.5	5.3%				41	
	5	13.0 - 13.9	6.7%					
	6	18.0 - 19.5	6.8%					
	7	23.0 - 24.4	6.2%					
B-107	1	1.0 - 1.8	5.3%					
	2	3.0 - 4.5	4.5%					
	3	5.0 - 6.5	4.1%					
	4	8.0 - 9.5	5.7%					
	5	13.0 - 14.0	7.2%					
	6	18.0 - 19.5	6.3%					
	7	23.0 - 24.4	4.2%					
B-108	1	1.0 - 2.5	4.9%					
	2	3.0 - 4.5	4.9%					
	3	5.0 - 6.5	6.0%					
	4	8.0 - 9.4	6.3%					
	5	13.0 - 13.8	8.4%					
	6	18.0 - 18.8	6.6%					
	7	23.0 - 23.4	6.0%					
B-109	1	1.0 - 2.5	3.4%					
	2	3.0 - 4.5	4.2%					
	3	5.0 - 6.5	3.8%					
	4	8.0 - 9.0	3.2%					
	5	13.0 - 13.3	8.5%					
	6	18.0 - 18.7	6.3%					
	7	23.0 - 24.5	4.7%					
B-110	1	1.0 - 2.5	5.0%	29	N.P.			
	2	3.0 - 4.5	4.7%					
	3	5.0 - 6.5	6.0%					
	4	8.0 - 9.5	8.9%					
	5	13.0 - 14.0	7.1%					
	6	18.0 - 19.0	5.1%					
	7	23.0 - 24.5	4.4%					

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SOIL DATA SUMMARY Consolidated Interim Storage Facility - Andrews, Texas GEOServices Project No. 31-151247 August 5, 2015								
Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits			Finer than 200 Sieve (%)	Soil Type
				LL	PL	PI		
B-111	1	1.0 - 2.5	6.7%					
	2	3.0 - 4.5	4.5%				44	
	3	5.0 - 6.5	5.2%					
	4	8.0 - 9.5	4.3%					
	5	13.0 - 14.0	6.6%				30	
	6	18.0 - 19.0	5.8%					
	7	23.0 - 24.0	6.0%					
	8	28.0 - 29.5	4.3%					
	9	33.0 - 34.5	2.5%					
AB-1	1	1.0 - 2.5	3.3%					
	2	3.0 - 4.5	3.9%					
	3	5.0 - 6.5	6.7%					
	4	8.0 - 9.5	6.2%				24	
	5	13.0 - 14.3	4.3%					
	6	18.0 - 18.8	4.9%					
	7	23.0 - 23.8	3.2%					
AB-2	1	1.0 - 2.5	4.8%				35	
	2	3.0 - 4.5	5.7%					
	3	5.0 - 6.5	5.5%					
	4	8.0 - 9.5	3.7%					
	5	13.0 - 14.5	3.4%					
	6	18.0 - 19.5	3.1%					
TP-1	1	1.0 - 2.5	3.0%					
	2	3.0 - 4.5	2.8%				45	
	3	5.0 - 6.5	4.8%					
	4	8.0 - 9.5	2.8%					
	5	13.0 - 14.3	5.8%					
	6	18.0 - 19.3	5.4%					
	7	23.0 - 24.5	2.7%					
	8	28.0 - 29.3	3.0%					
	9	33.0 - 33.4	3.7%					
	10	38.0 - 38.3	4.6%					
TP-2	1	1.0 - 2.5	4.3%	28	N.P.			
	2	3.0 - 4.5	4.8%					
	3	5.0 - 6.5	3.9%					
	4	8.0 - 9.5	6.7%					
	5	13.0 - 14.2	5.9%					
	6	18.0 - 18.8	5.9%					
	7	23.0 - 24.5	4.2%					

SOIL DATA SUMMARY
Consolidated Interim Storage Facility - Andrews, Texas
GEOServices Project No. 31-151247
August 5, 2015

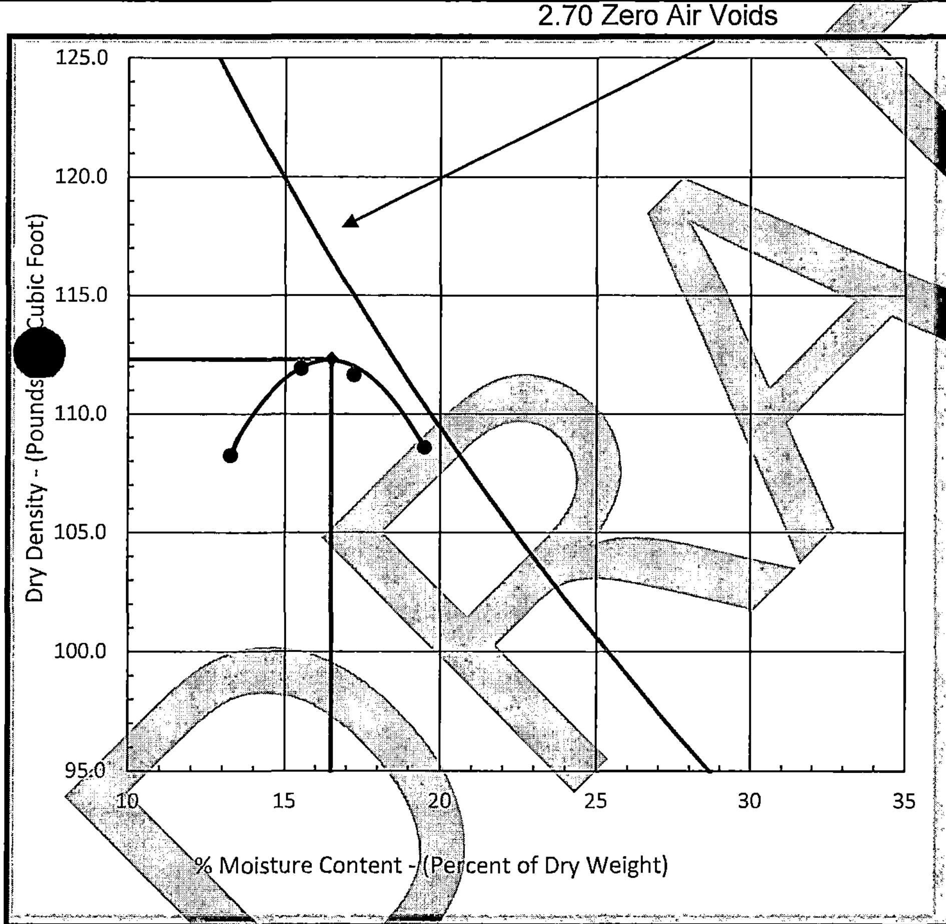
Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Finer than 200 Sieve (%)	Soil Type
TF-3	1	1.0 - 2.5	4.3%					
	2	3.0 - 4.5	4.6%					
	3	5.0 - 6.5	3.9%					
	4	8.0 - 9.5	5.5%					
	5	13.0 - 14.5	9.0%					
	6	18.0 - 19.5	4.5%					
	7	23.0 - 24.5	3.8%					
TF-4	1	1.0 - 2.5	3.7%					
	2	3.0 - 4.5	3.9%					
	3	5.0 - 6.5	6.6%					
	4	8.0 - 9.5	7.4%					
	5	13.0 - 14.2	5.2%				34	
	6	18.0 - 19.2	4.3%					
	7	23.0 - 24.5	3.9%					
	8	28.0 - 28.8	6.5%					
	9	33.0 - 33.3	3.3%					
	10	38.0 - 38.1	3.5%					
TF-5	1	1.0 - 2.5	4.6%					
	2	3.0 - 4.5	5.2%					
	3	5.0 - 6.5	5.5%					
	4	8.0 - 9.5	8.1%					
	5	13.0 - 14.5	6.4%					
	6	18.0 - 19.5	5.0%					
	7	23.0 - 24.5	4.2%					

LABORATORY COMPACTION OF SOILS
ASTM D 698 Method C

GEOServices, LLC-Geotechnical and Materials Engineers

GEOS Project Name:	WCS CISF - Andrews, TX		
GEOS Project Number:	31-151247	Report Date:	December 31, 2018
GEOS Log #:	NA	Date Received:	December 24, 2018

Sample Location:	B-101
Sample Depth:	0 - 10'
Sample Description:	Brown Silty Sandy Clay with rock



Max. Dry Density (pcf)
112.3
Optimum Moisture (%)
16.5

As Received W%	TNP
Rammer Type	Manual
Prep Method	Dry

ASTM D4718
Percent Oversize Material
60.09
OS Bulk Specific Gravity
TNP
Oversize Corrected Data Corrected MDD (pcf)
NA
Corrected Opt Moisture %
7.2

LL / PL / PI - Data By:		ASTM D4318	
Liquid Limit	22	Plasticity Index	9
Plastic Limit	13	Soil Type	CL

SOIL SPECIFIC GRAVITY BY ASTM D854
ASSUMED 2.70

NOTES

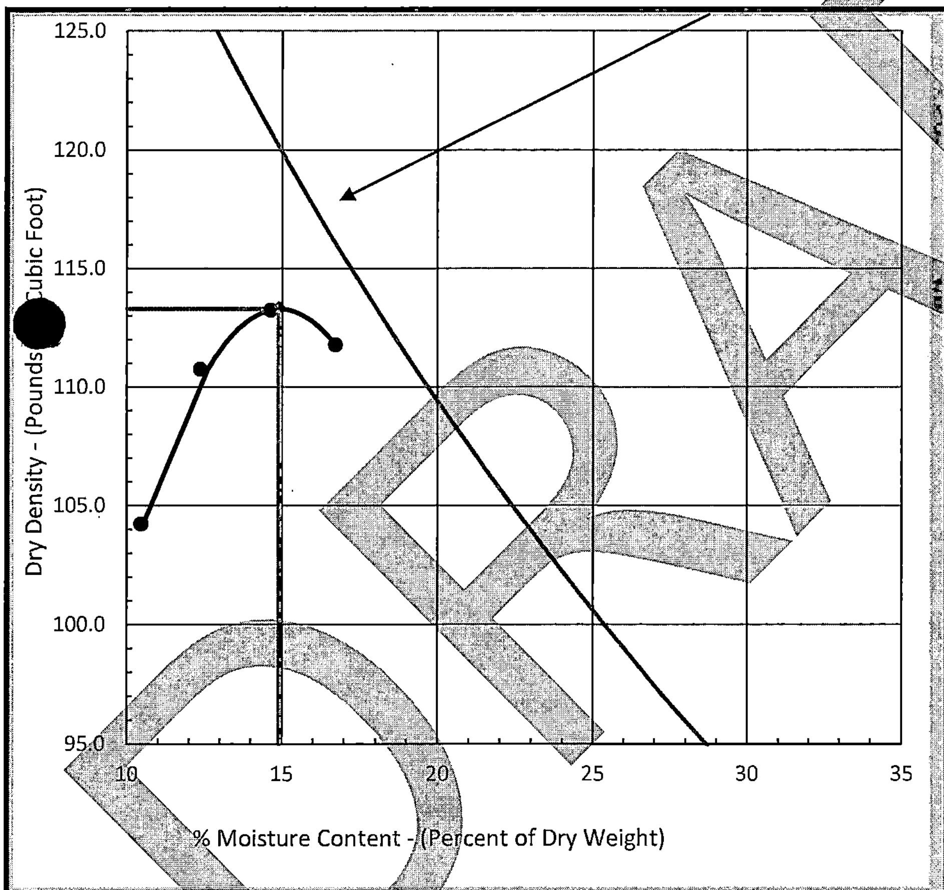
TIP = Test In Progress

LABORATORY COMPACTION OF SOILS
ASTM D 698 Method B

GEOServices, LLC-Geotechnical and Materials Engineers

GEOS Project Name:	WCS CISF - Andrews, TX
GEOS Project Number:	31-151247
GEOS Log #:	
Report Date:	December 31, 2018
Date Received:	December 24, 2018

Sample Location:	B-111
Sample Depth:	0 - 10'
Sample Description:	Brown Silty Clay with rock



Max. Dry Density (pcf)	113.3
Optimum Moisture (%)	14.9

As Received W%	TNP
Rammer Type	Manual
Prep Method	Dry

ASTM D4718
Percent Oversize Material
47.72
OS Bulk Specific Gravity
TNP
Oversize Corrected Data Corrected MDD (pcf)
TNP
Corrected Opt Moisture %
8.3

LL / PL / PI Data By:	ASTM D4318		
Liquid Limit	TIP	Plasticity Index	TIP
Plastic Limit	TIP	Soil Type	TIP

SOIL SPECIFIC GRAVITY BY ASTM D854
ASSUMED 2.70

NOTES

TIP = Test In Progress

DRAFT

GEOServices, LLC-Geotechnical and Materials Engineers

CALIFORNIA BEARING RATIO
ASTM D1883

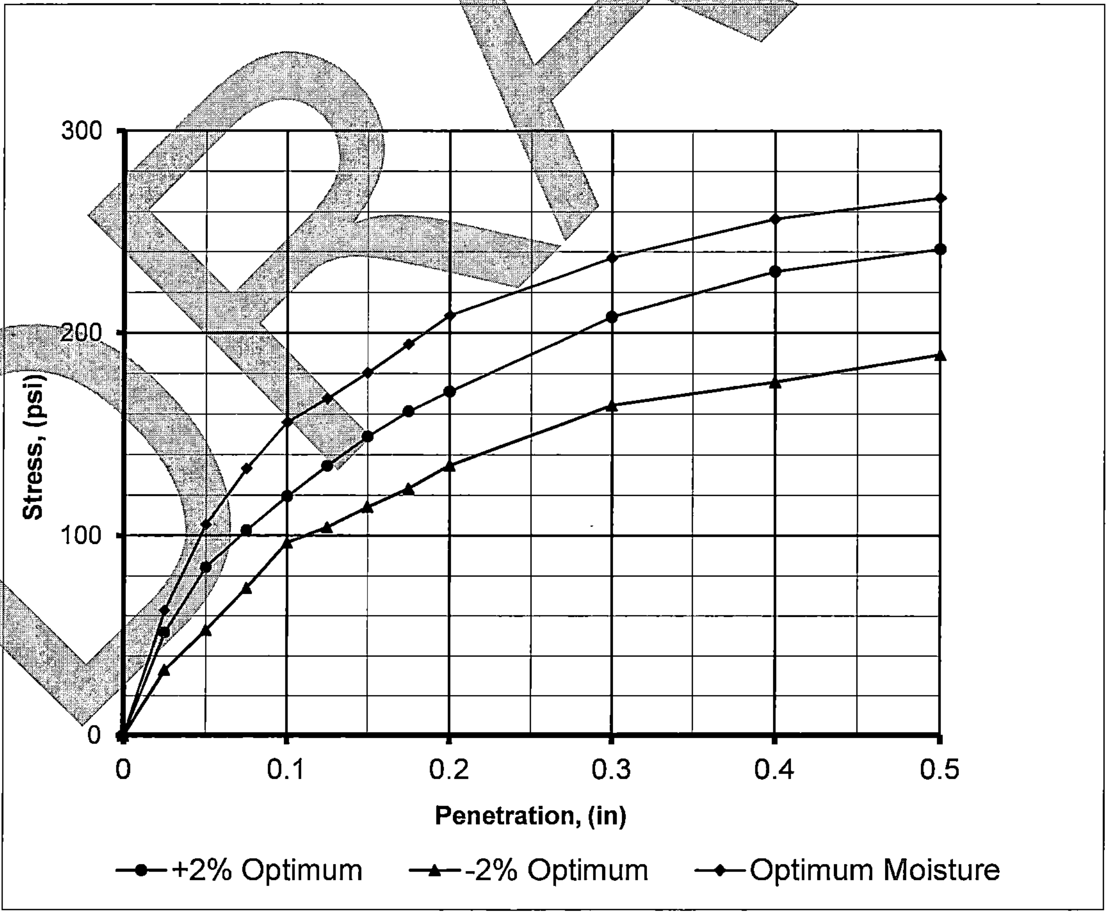
Project Name: **WCS - CISF**

Project #: **31-151247**

Sample ID: **B-101**

Sample Description: **Brown Silty Sand with Rock (CBR on -3/4 material)**

@ + 2% Optimum Moisture			@ Optimum Moisture Content			@ -2% Optimum Moisture		
Wet Density	128.0	pcf	Wet Density	124.8	pcf	Wet Density	120.4	pcf
Dry Density	108.5	pcf	Dry Density	107.1	pcf	Dry Density	105.1	pcf
Water Content	17.9%		Water Content	16.6%		Water Content	14.5%	
Compaction	96.6%		Compaction	95.4%		Compaction	93.6%	
Swell	0.09%		Swell	0.11%		Swell	-0.07%	
CBR @ .1"	**12.0**		**CBR @ .1"**	**15.6**		**CBR @ .1"**	**9.6**	
CBR @ .2"	**11.4**		**CBR @ .2"**	**13.9**		**CBR @ .2"**	**9.0**	



GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

CALIFORNIA BEARING RATIO
ASTM D1883

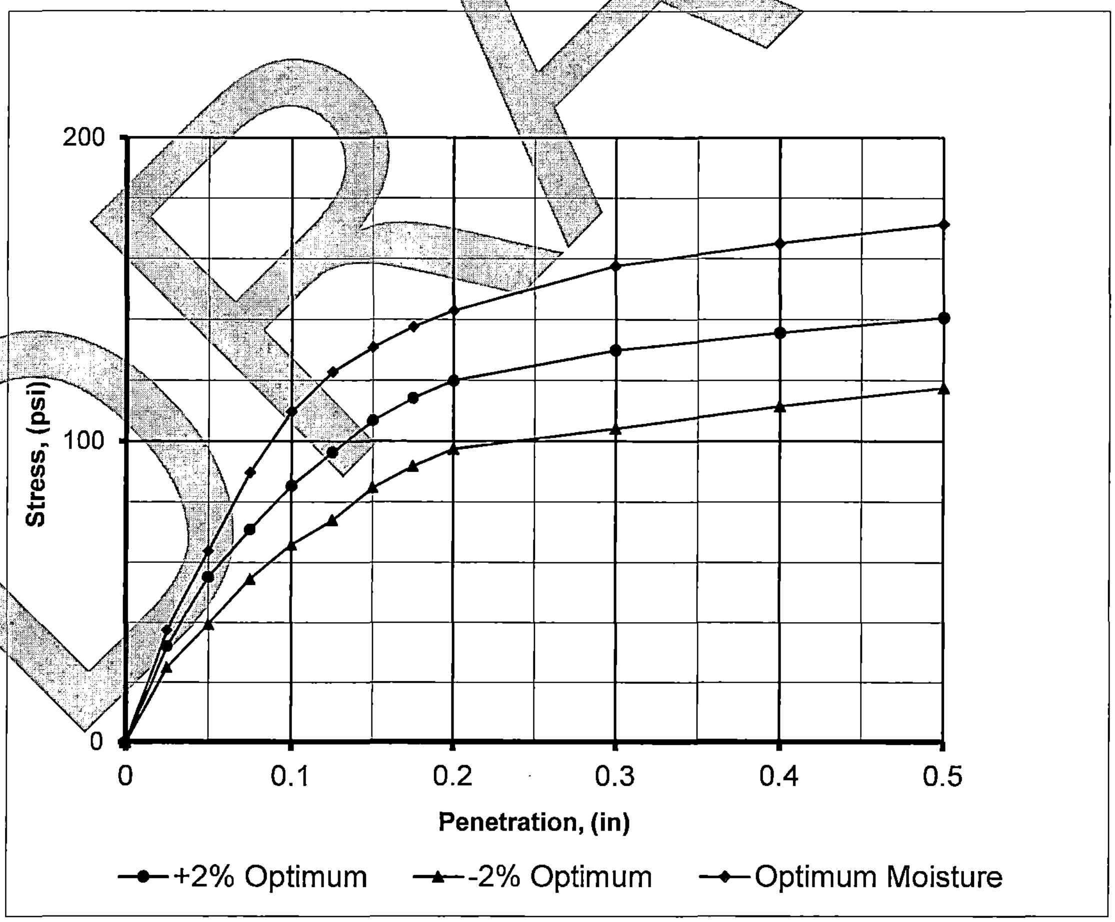
Project Name: **WCS CISF**

Project #: **31-151247**

Sample ID: **B-111**

Sample Description: **Brown Silty Sand with Rock (CBR on -3/8 material)**

	@ + 2% Optimum Moisture		@ Optimum Moisture Content		@ -2% Optimum Moisture	
Wet Density	126.8	pcf	125.1	pcf	122.5	pcf
Dry Density	109.1	pcf	109.1	pcf	109.3	pcf
Water Content	16.2%		14.6%		12.1%	
Compaction	96.3%		96.3%		96.5%	
Swell	0.31%		0.13%		0.48%	
CBR @ .1"	**8.5**		**11.0**		**6.6**	
CBR @ .2"	**8.0**		**9.5**		**6.5**	



GEOServices, LLC-Geotechnical and Materials Engineers

SOIL RESISTIVITY & pH

ASTM G57 & D4972

GEOS Project Name: WCS CISF

Report Date: January 7, 2019

GEOS Project Number: 31-151247

Date Received:

Sample ID	Depth	Resistivity (ohm/cm)	pH
B-101 / Bulk	1.0 - 2.5'	2,550	7.9
B-111 / Bulk	3.5 - 5.0'	2,900	8.1

Notes:

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APPENDIX C

Generalized Soil Column

GEOS

Appendix C – Generalized Soil Column CISF Site

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

As can be seen above the soil column for the site was extended to 600 feet. Only 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) that existed at the time of the exploration. 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 was utilized to extend the soil profile and engineering parameters to a depth of 600 feet. 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.

Provided Additional Documents:

1. Cook-Joyce, Inc. (2007). Geology Report. Dated May 1, 2007.
2. AECOM. (2016). Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, WCS Centralized Interim Storage Facility Project. Dated March 18, 2016.
3. WCS. (2007). (Waste Control Specialists LLC). Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste. Dated March 2007.

Methodology:

The information from the eighteen borings and shear wave data included in this Report of Geotechnical Exploration was supplemented with data obtained from the additionally provided documents. This data was used to produce a soil stratigraphic column to 600 feet. Figure 1 shown below displays the locations of the historical borings provided.

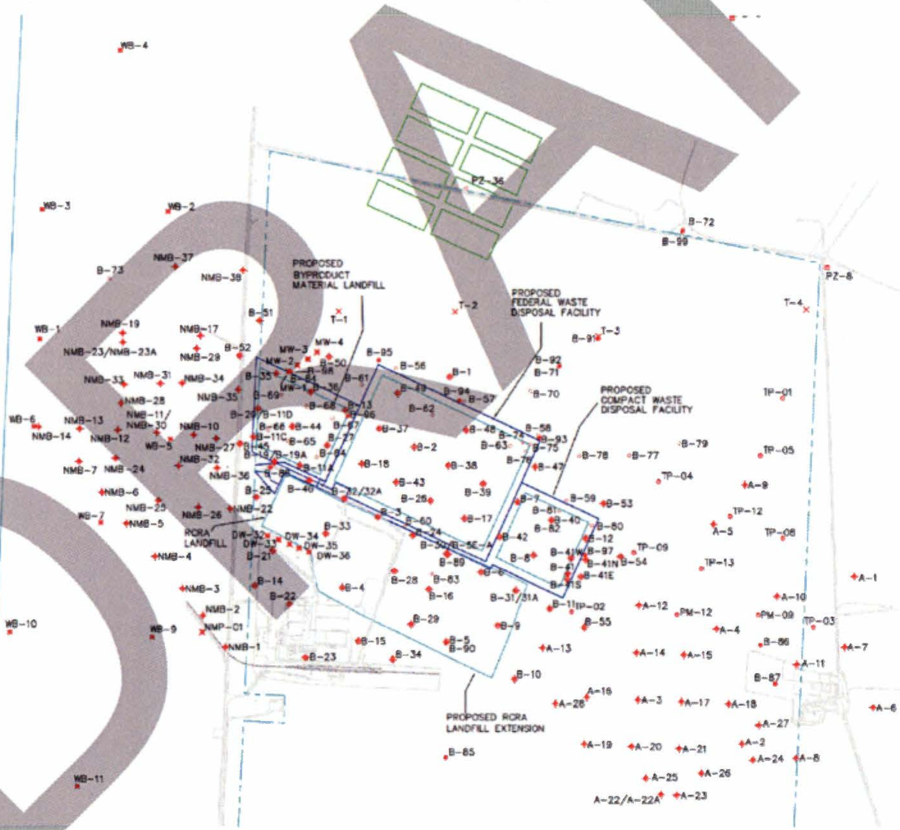

Figure 1: Historical Borings at WCS Site

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.

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APPENDIX D

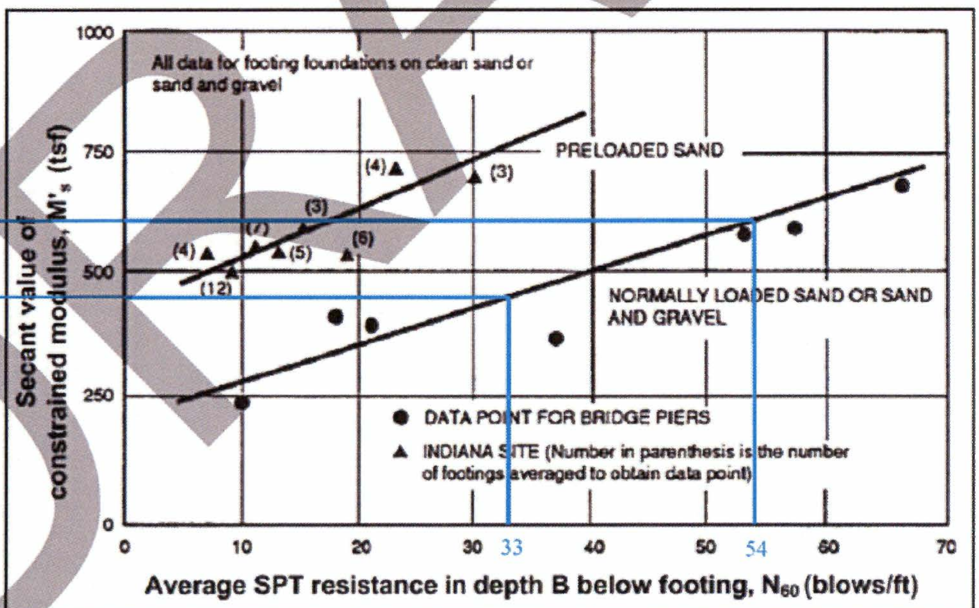
Static Elastic Modulus Calculation

Appendix D – Static Elastic Modulus Calculation

As mentioned previously, it was determined that the settlement analysis would be extended to a depth of 600 feet (the top of the Trujillo Sandstone Formation). Therefore, constrained modulus values needed to be calculated for each of the stratigraphic layers. This was accomplished utilizing two distinct methodologies. The methodologies were selected due to the information available from the borings and shear wave profiles performed as part of this study and the available historical data.

Methodology 1:

To a depth of 20 feet bgs the constrained modulus was correlated to the SPT N-values obtained in the borings. The SPT N-Values were correlated to constrained modulus using the method outlined in Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M. (1991). This methodology allows correlation of constrained modulus to N-value for N-values up to 70 blows per foot. The graphical representation is shown below.



Methodology 2:

The borings performed for the CISF site were only advanced to maximum depths of 45 feet. Additionally, the methodology outlined in Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M. (1991) 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, GEOServices was provided with a Site-Specific Seismic Hazard Evaluation and Development of Seismic Ground Motions prepared by AECOM (2016). This document provided shear wave velocity profiles at the site to depths of approximately 1200 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, Vs = 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 this study 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.

Results

The table below provides the constrained modulus values for each of the stratigraphic layers. These values were utilized to calculate the anticipated settlements for the CISF pads and Cask Handling Building. The results of the settlement analysis are provided in Appendix H of this document.

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	1200
20	25		1530	Caliche - Very Hard	35815
25	35		1900	Caliche - Very Hard	55232
35	50		2290	Ogallala - Sand with Gravel	80233
50	80		1840	Ogallala - Sand with Gravel	53870
80	100		2790	Ogallala - Sand with Gravel	123857
100	130		2300	Dockum - Claystone and Siltstone	84172
130	230		2755	Claystone and Siltstone	120769
230	275		2755	Dockum - Claystone	120769
275	300		2755	Dockum - Silty Sands	120679
300	360		2755	Dockum - Claystone	120679
360	600		3115	Dockum - Claystone	154394

References:

Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M. (1991). 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, 171 pp.

WCS. (2007). (Waste Control Specialists LLC). Application for License to Authorize Near Surface Land Disposal of Low-Level Radioactive Waste. Dated March 2007.

AECOM. (2016). Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, WCS Centralized Interim Storage Facility Project. Dated March 18, 2016.

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APPENDIX E

On-Site Shear Wave Velocity Study

Notes:

1) S = Start of Seismic Shear Wave Velocity Profile
2) E = End of Seismic Shear Wave Velocity Profile

Seismic Shear Wave Velocity Profile Location & Identifier

Aerial Source Provided by: Google Earth Pro, (02/12/2014)

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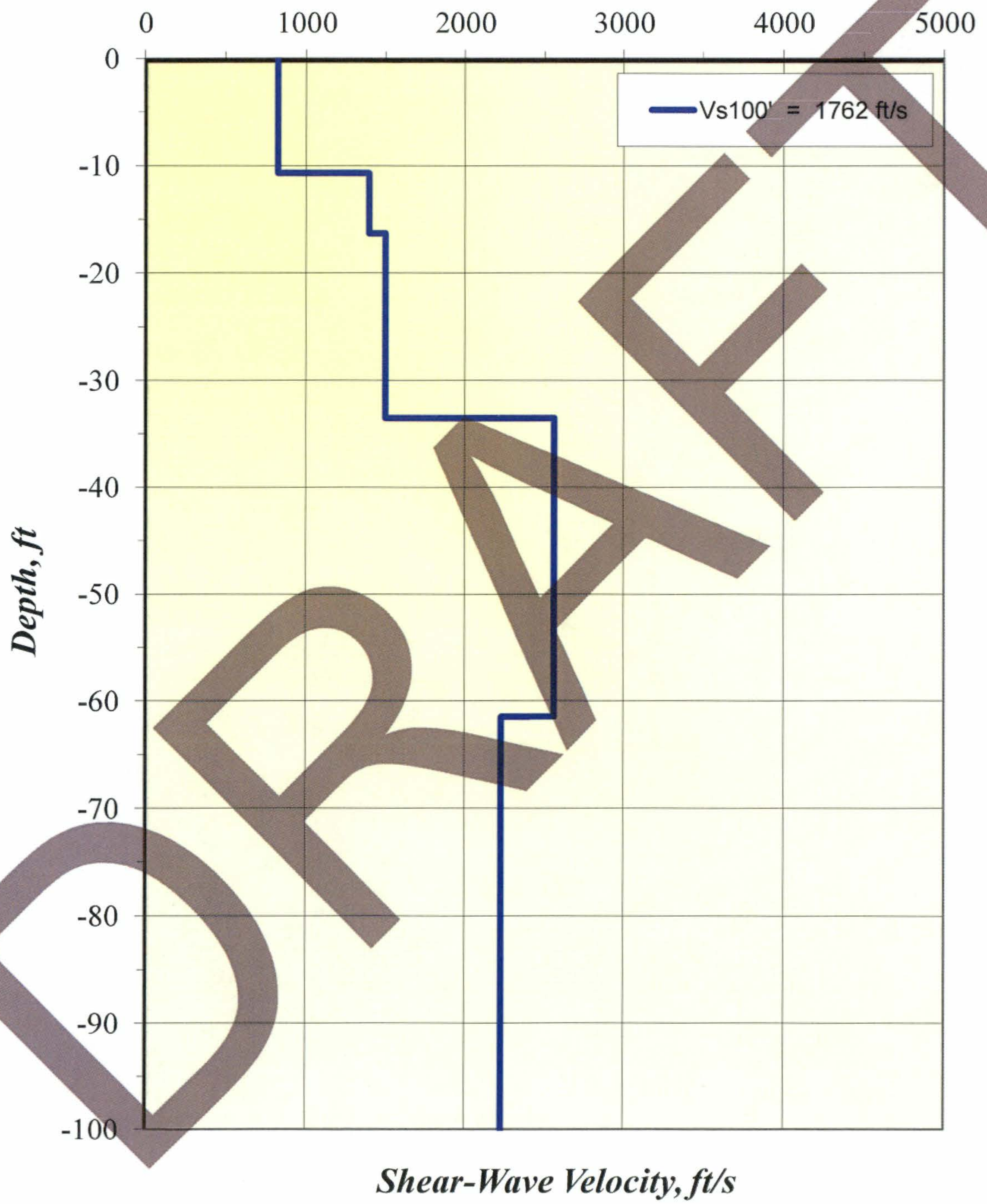
Seismic Shear Wave Velocity Location Plan
Consolidated Interim Storage Facility
Andrews, Texas

DATE: 07/14/2016

GEOS Project No. 31-151247

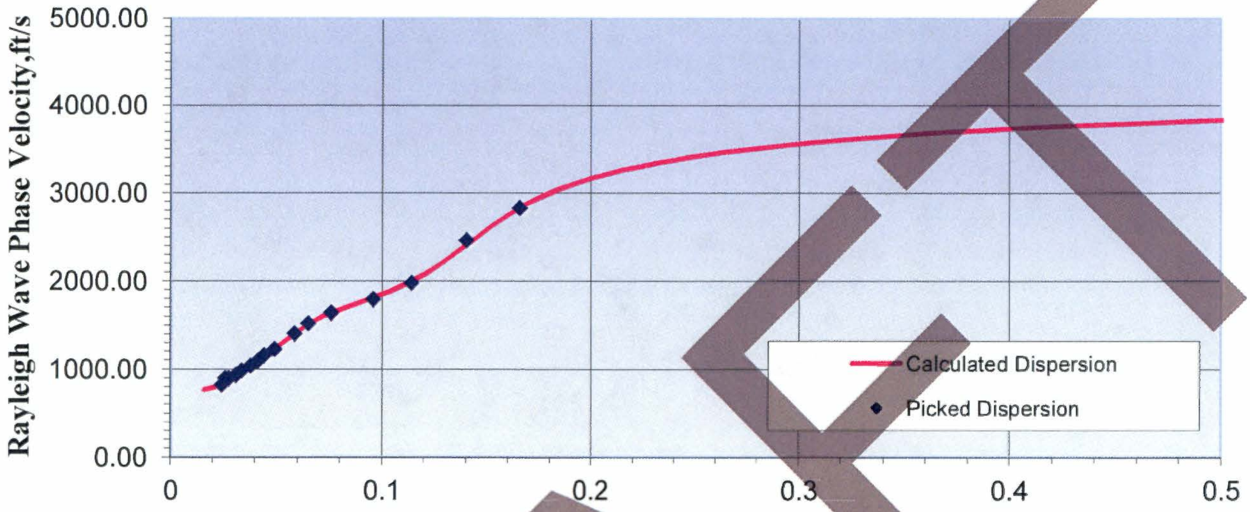
FIGURE: 12

Remi 1: Vs Model



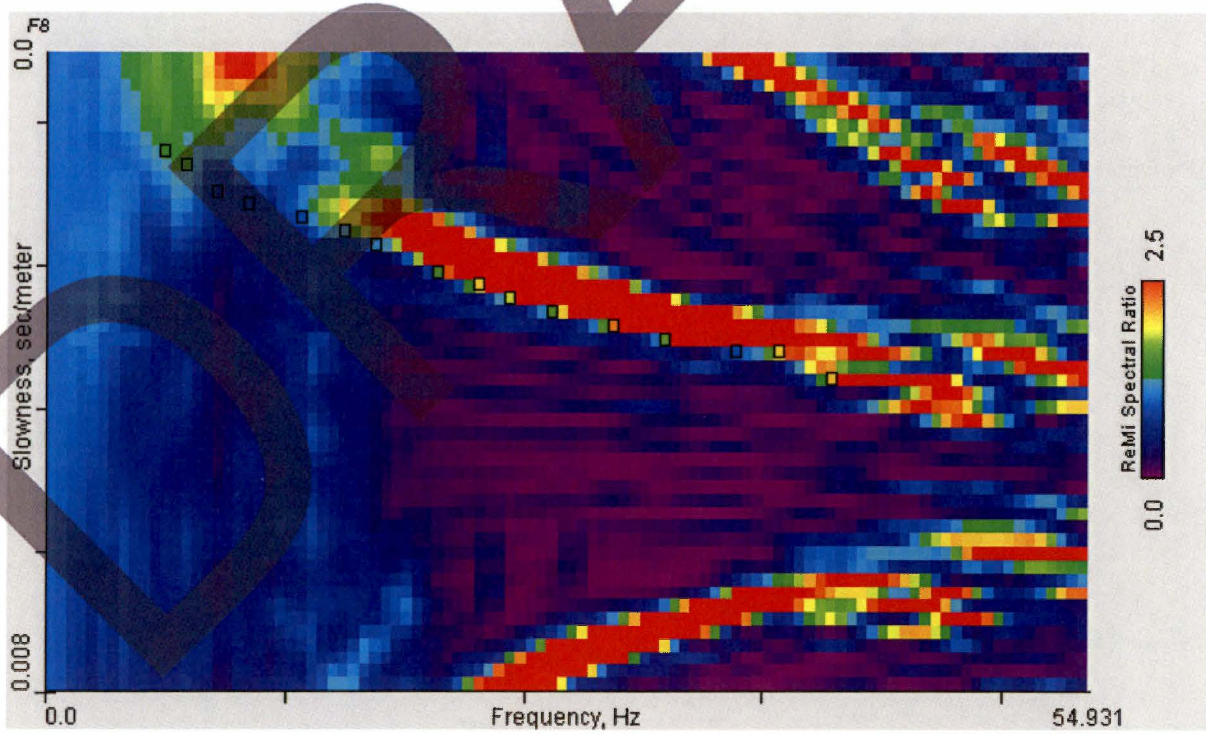
Remi 1: Supportive Illustration

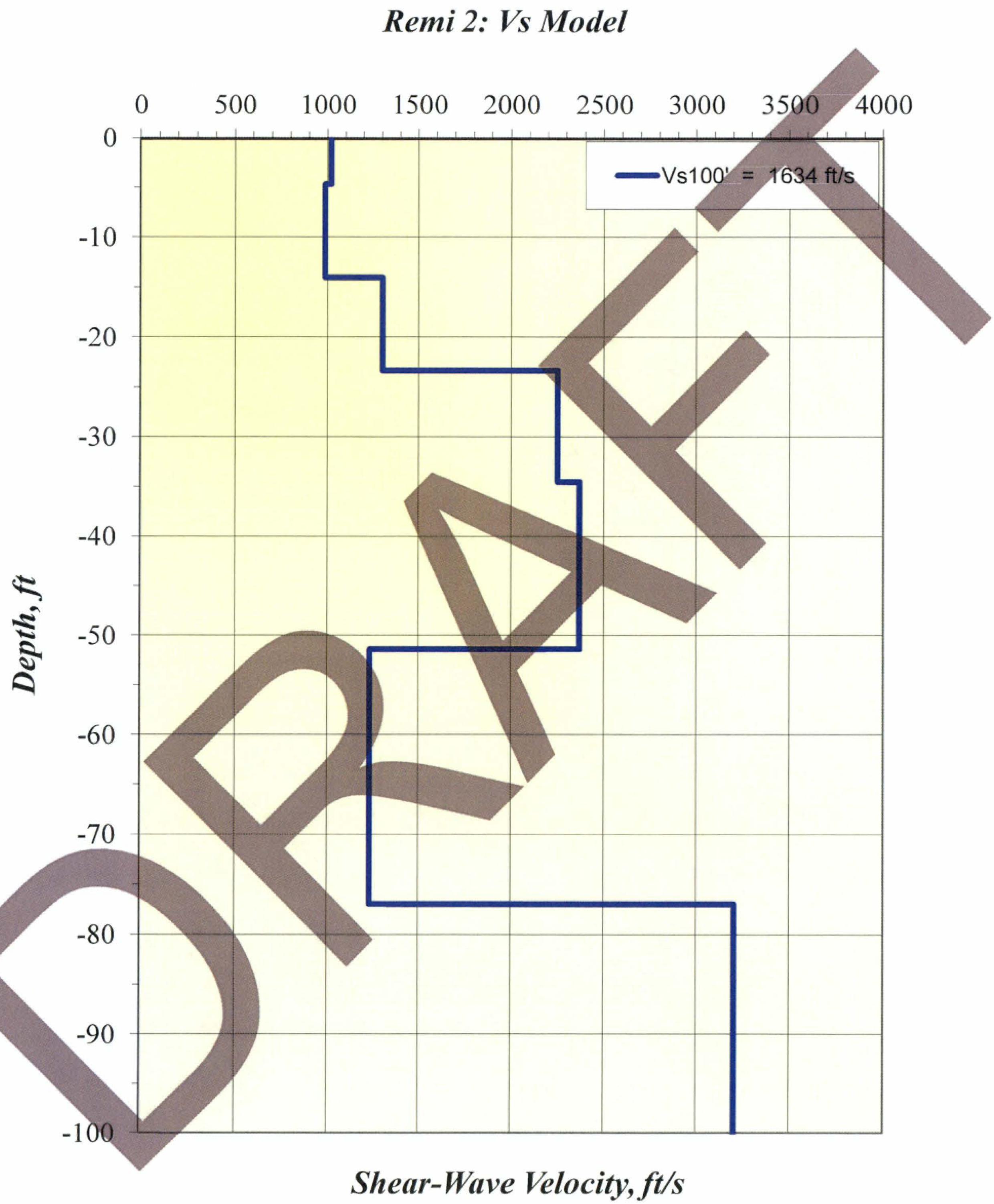
Dispersion Curve Showing Picks and Fit



Period, s

p-f Image with Dispersion Modeling Picks

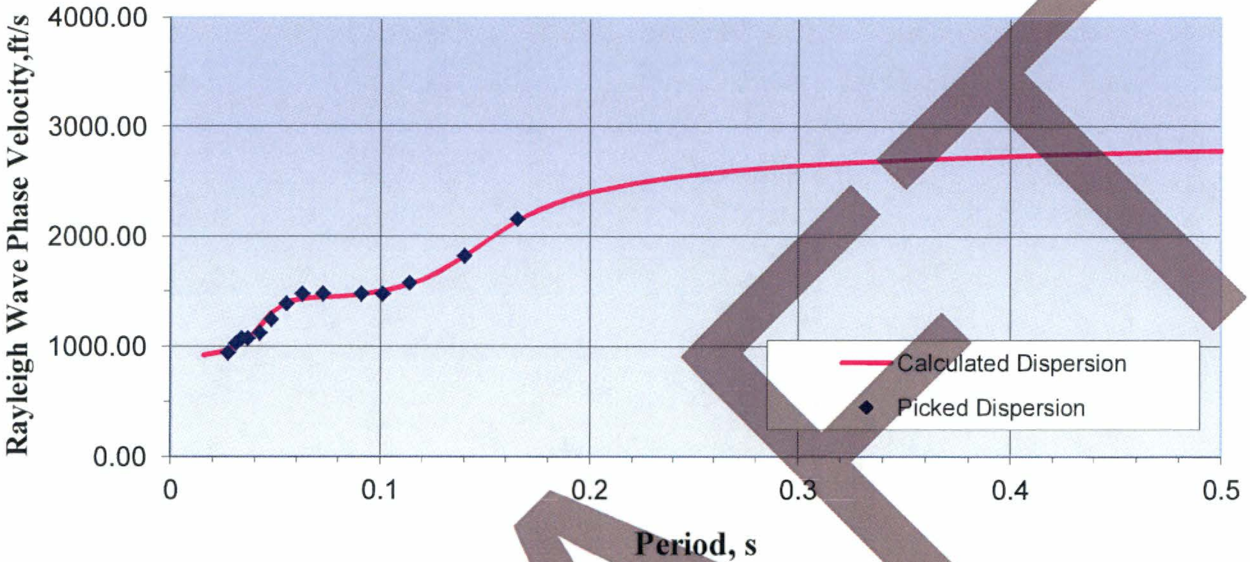




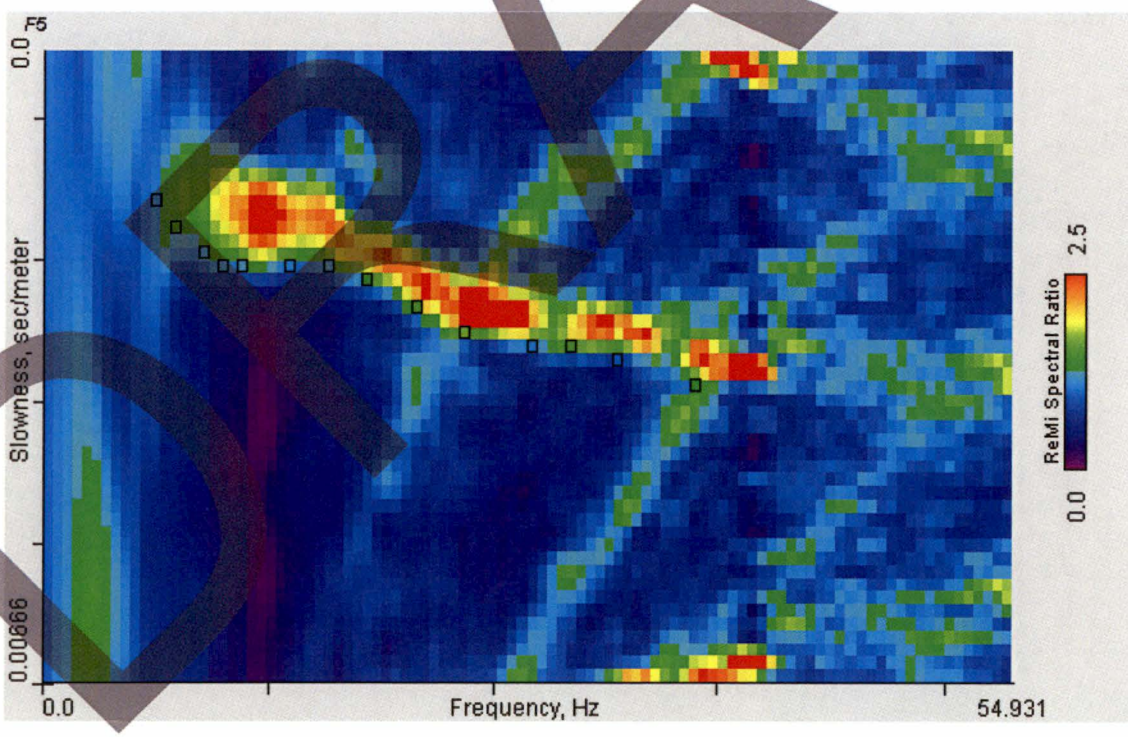
Remi 2: Vs Model
0
500
1000
1500
2000
2500
3000
3500
4000
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Vs100' = 1634 ft/s
Depth, ft
Shear-Wave Velocity, ft/s

Remi 2: Supportive Illustration

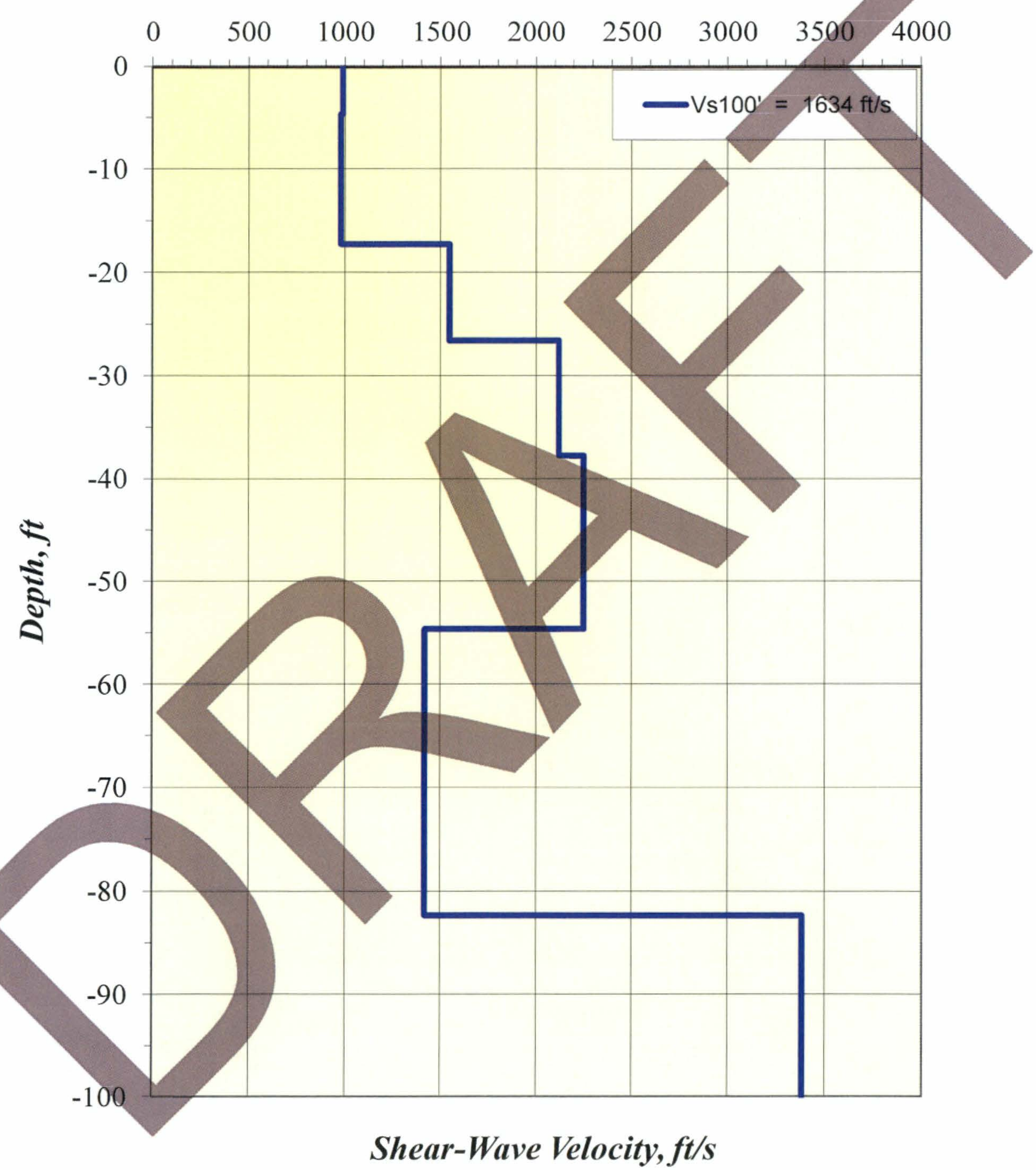
Dispersion Curve Showing Picks and Fit



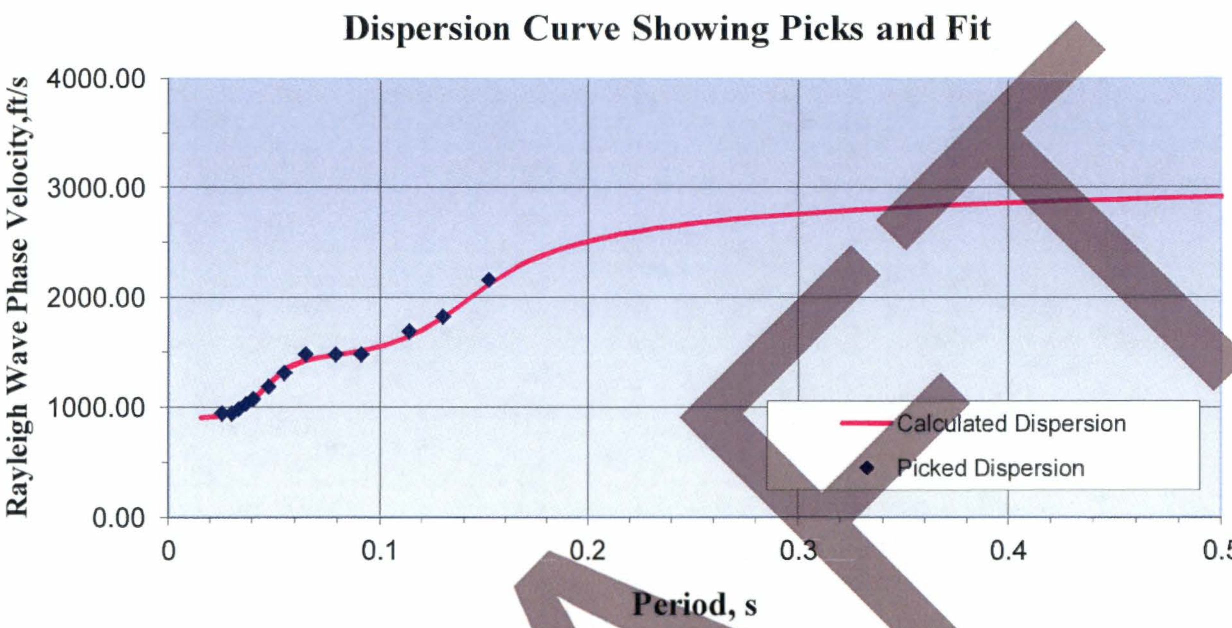
p-f Image with Dispersion Modeling Picks

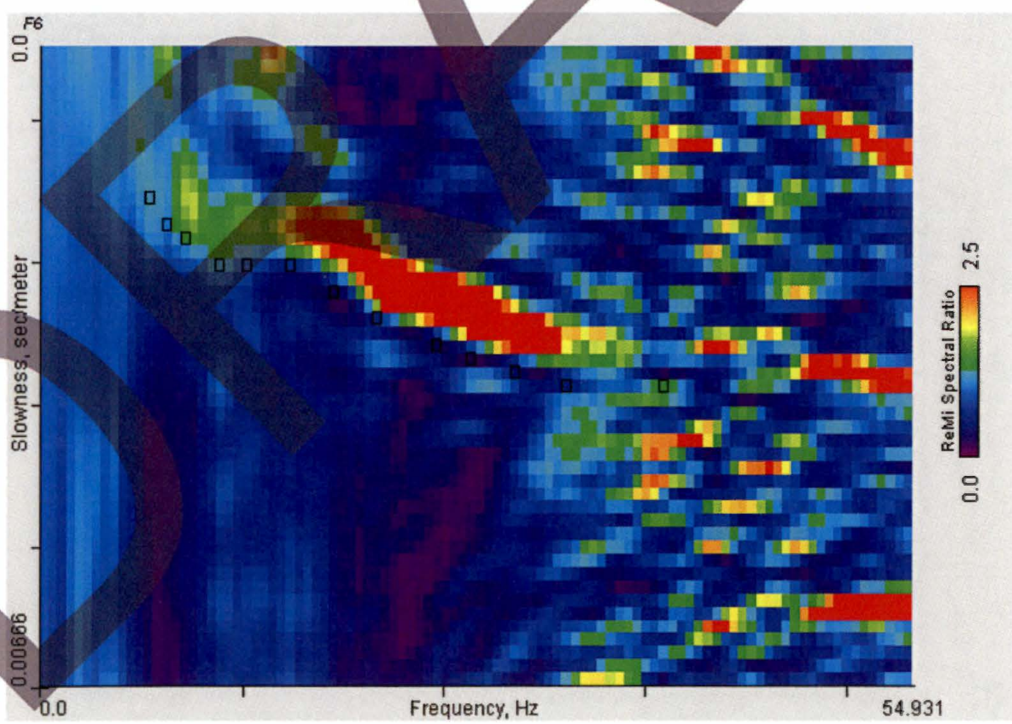


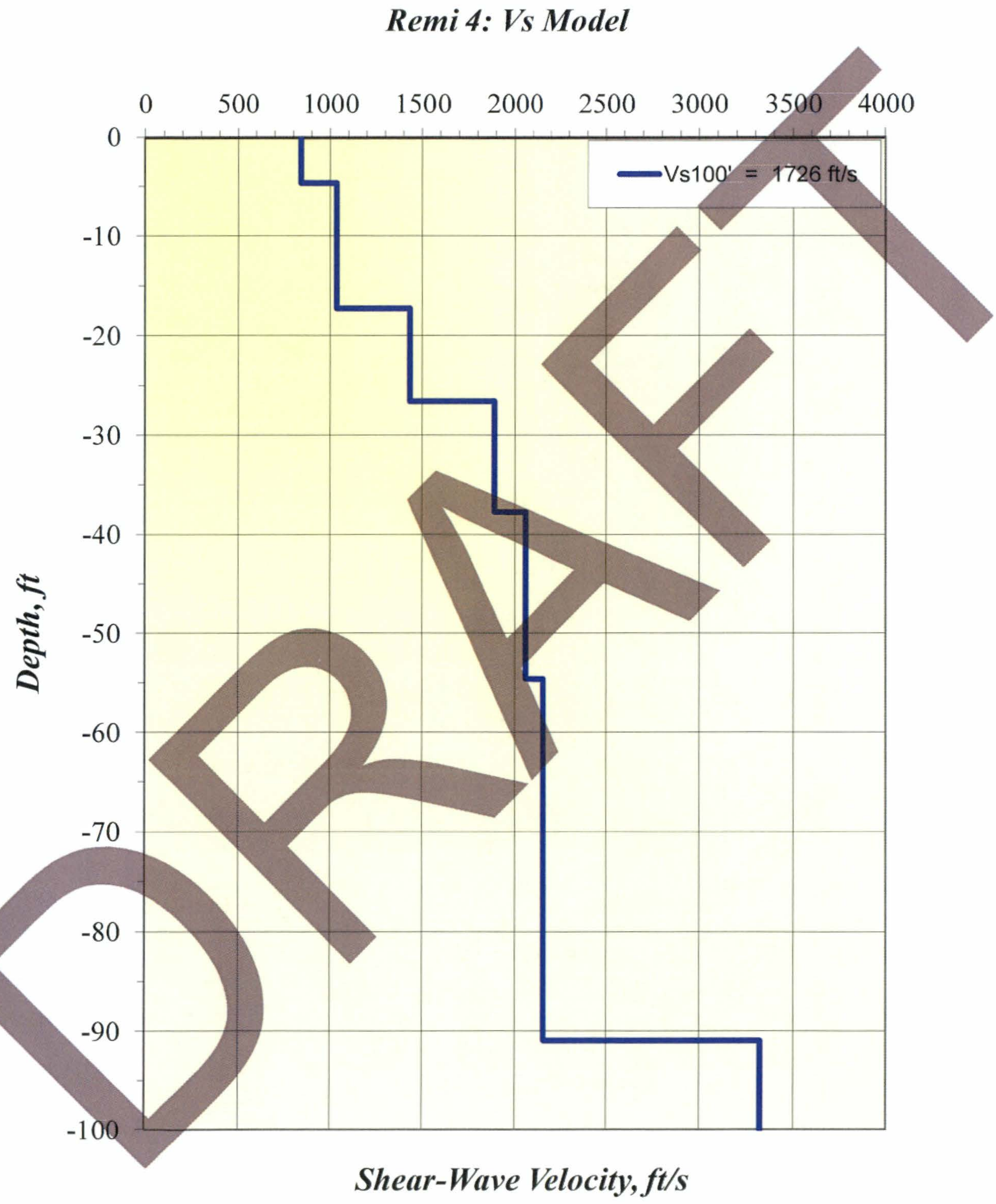
Remi 3: Vs Model



Remi 3: Supportive Illustration



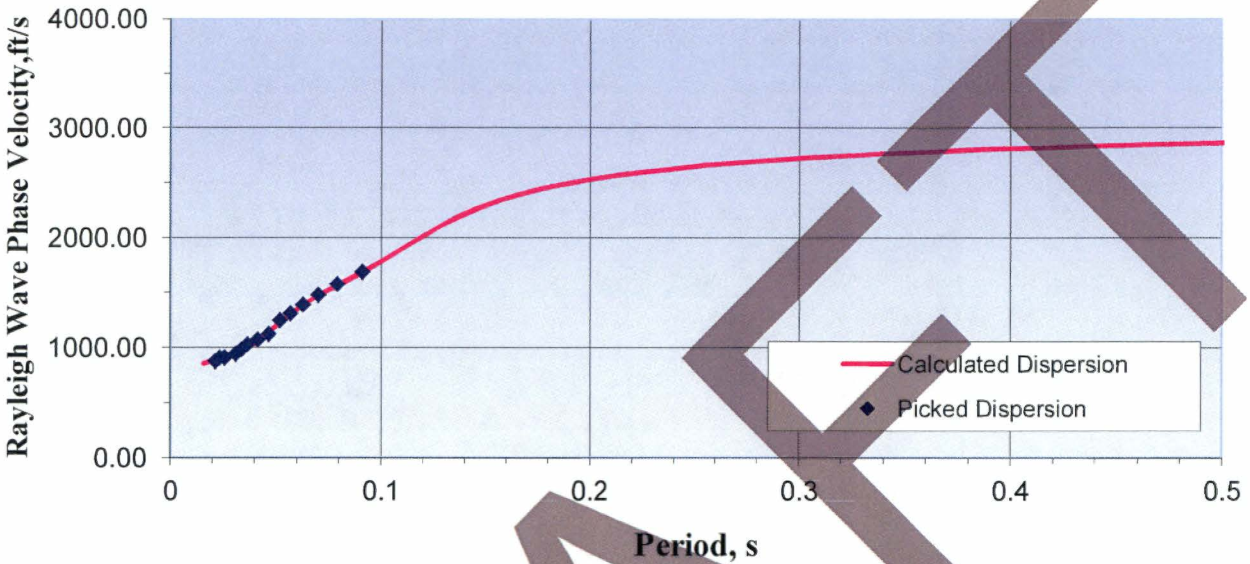




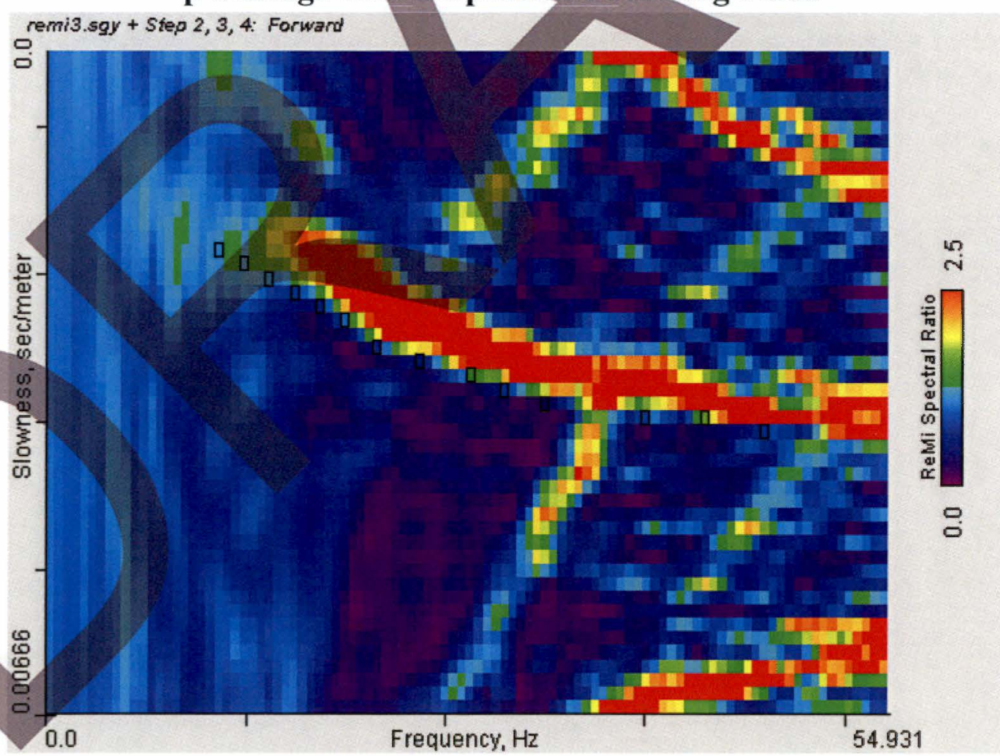
Remi 4: Vs Model
0
500
1000
1500
2000
2500
3000
3500
4000
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Vs100' = 1726 ft/s
Depth, ft
Shear-Wave Velocity, ft/s
DRAFT

Remi 4: Supportive Illustration

Dispersion Curve Showing Picks and Fit



p-f Image with Dispersion Modeling Picks



GEOServices, LLC, Geotechnical and Materials Engineers

APPENDIX F

Seismic Densification Analysis

DAN BROWN AND ASSOCIATES

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TECHNICAL MEMORANDUM

Seismic Densification
CISF Site
Andrews, TX
DBA Project No. 19-017

To: Derek Kilday, P.E./GEOServices

From: Timothy C. Siegel, P.E, G.E., D.GE
Tayler J. Day, P.E.

Date: 18 February 2020

STATE OF TEXAS
TIMOTHY C. SIEGEL
117477
LICENSED
PROFESSIONAL ENGINEER

1. Introduction

Dan Brown and Associates, P.C. (DBA) performed seismic densification calculations as part of our scope of services for the subject project. The calculations show that the seismic densification for the design earthquake will be negligible (on the order of 0.02 inches or less). The basis of our calculations is described in the remaining sections of this TM. The calculations are provided in the Attachments.

2. Design Earthquake

Our calculations use an earthquake magnitude of 5 and a peak ground acceleration of 0.25g. According the AECOM report[1], these values represent the design earthquake determined as part of the site-specific seismic hazard evaluation.

3. Soil Profile

The soil column at the CISF Storage Pad site consists of approximately 2 ft of cover sands overlying a caliche and sand matrix with normalized SPT N-values ranging 10 to 57 over the top 20 feet. This profile was developed based on the boring information (B-101 thru B-110) and laboratory test results presented in the GEOServices report[2]. We expect the cemented caliche materials described below a depth of 20ft to exhibit significantly more resistance to seismic densification than the partially cemented and uncemented sands near the ground surface. Therefore, we intentionally limited our calculations to the upper 20 ft where the sands and caliche/sand matrix appear to exhibit less cementation.

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[1] AECOM (2016) Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, WCS Centralized Interim Storage Facility, Project No. 31787-001, Study No. WCS-12-05-100-001.

[2] GEOServices (2020) Report of Geotechnical Exploration, Consolidated Interim Storage Facility, Andrews, TX, GEOServices Project No. 31-151247.R1.

4. Methodology

DBA used the methodology proposed by Pradel[3] to compute the seismic densification. The Pradel method is applicable to sands and silty sands and we expect that it could tend to overpredict the seismic densification of soils with partial to full cementation. For the design earthquake and soil profile for this project, the computed seismic densification above the cemented layers is very small.

5. Concluding Remarks

DBA appreciates the opportunity be a part of this project. Please contact the following if you would like to discuss this document or this project.

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3 Pradel, D. (1998) Procedure to Evaluate the Earthquake-Induced Settlements of Dry Sandy Soils, Journal of Geotech. and Geo-env. Engineering, 124(4), 364-368.

Attachments

Seismic Densification Calculations

DAN BROWN AND ASSOCIATES

Seismic Densification Calculation for Upper 20 ft
CISF
Andrews, TX
18-Feb-20

M = 5.00
Nc = 1.0

Depth (ft)	(N1)60	σ'v (tsf)	CSR	CSR * σ'v (tsf)	ϕ (deg)	Go (tsf)	K	p (tsf)	R	a	b	γ(%)	evol15 (%)	evol (%)	
2.00	21	0.12	0.13	0.015	30	345.291	0.500	0.080	0.004%	0.127	30131.266	0.01%	0.01%	0.00%	0.001
4.00	10	0.24	0.16	0.038	30	382.268	0.500	0.160	0.010%	0.130	19879.222	0.02%	0.04%	0.01%	0.006
6.00	13	0.36	0.16	0.057	30	510.522	0.500	0.240	0.011%	0.133	15586.357	0.02%	0.03%	0.01%	0.004
8.00	35	0.48	0.16	0.075	30	817.382	0.500	0.320	0.009%	0.136	13115.395	0.01%	0.01%	0.00%	0.001
14.00	57	0.84	0.15	0.124	30	1270.107	0.500	0.560	0.010%	0.145	9374.725	0.01%	0.00%	0.00%	0.001
19.00	36	1.14	0.14	0.161	30	1271.435	0.500	0.760	0.013%	0.152	7805.185	0.02%	0.01%	0.00%	0.003

S (IN) = 0.016

M	Earthquake magnitude	
Nc	Number of equivalent cycles	
(N1)60	Normalized N-value	Computed from SPT boring data
σ'v (tsf)	Effective vertical stress	Computed from depth x soil unit weight
CSR	Cyclic stress ratio	Computed from design earthquake acceleration (No reduction for M lower than 7.5)
ϕ (deg)	Effective friction angle	Typical value of sand
Go (tsf)	Small strain shear modulus	Computed based on based on Seed and Idriss (1970)
K	At-rest horizontal pressure coefficient	
p (tsf)	Mean stress	
R	Ave shear stress/G	
a	coefficient to determine shear strain	
b	coefficient to determine shear strain	
γ(%)	Shear strain	
(N1)60, cs	Normalized N-value, clean sand	
Ic	Soil behavior type index	
evol15 (%)	Volumetric strain after 15 cycles	
evol (%)	olumetric strain adjusted for actual cycles	
S (in)	Ground surface settlement from seismic compression	

References:

Pradel, D. (1998) "Procedure to Evaluate Earthquake-Induced Settlement in Dry Sandy Soils" J. Geotech Engrg, ASCE, 124(4), 364-368
Robertson, P.K. and Shao, L. (2010) "Estimation of Seismic Compression of Dry Soils Using the CPT" Proc. Fifth International Conf on Recent Advances in Earthquake Engineering and Soil Dynamics.
Seed, H.B. and Idriss, I.M. (1970) "Soil moduli and damping factors for dynamic response analyses" Rep No. EERC 70-10, Earthquake Engineering Research Center, University of CA, Berkeley.

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Engineer TCS
Checked by TJD

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil		$\gamma_{total} := 120pcf$	
unit weight of water		$\gamma_{water} := 62.4pcf$	
water table depth below ground surface....very deep			
sample depth	Boring B-103 Sample No. 1	$z := 2ft$	
total vertical stress		$\sigma tot_v := \gamma_{total} \cdot z$	$\sigma tot_v = 240 \cdot psf$
static water pressure		$u_o := 0$	$u_o = 0 \cdot psf$
effective vertical stress		$\sigma eff_v := \sigma tot_v - u_o$	$\sigma eff_v = 240 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value	$N_m := 12$
fines content	$FC := 48$
energy correction	$C_E := 0.75$
borehole diameter correction	$C_B := 1.05$
rod length correction	$C_R := 0.75$
sampler correction	$C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
<3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60%

$$N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$$

$$N_{60} = 7.1$$

overburden correction factor
(based on overburden stress only)

$$C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$$

$$C_{N\_a} = 2.969$$

normalized SPT N-value
(based on overburden stress only)

$$N_{1\_60} := C_{N\_a} \cdot N_{60} \qquad N_{1\_60} = 21$$

overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot(N_{1\_60})^{0.5}}$$

$$C_{N\_b} = 2.529$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 2.64$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 18.7$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot(N_{1\_60})^{0.5}}$$

$$C_{N\_check} = 2.643$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 24.324$$

Earthquake Conditions -

peak ground accelerationl $a_{max} := 0.25\cdot g$

earthquake moment magnitude $M_w := 5$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot \sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118 \sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.998$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.162$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor
(maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 1.919$

coefficient for overburden correction factor
(maximum $C\sigma = 0.3$) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.127$

overburden correction factor
(maximum $K\sigma = 1.1$) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.277$

static shear stress on horizontal plane $\tau_{ho} := 0psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 43.824 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.308$

empirical constant for dilatancy $Q := 10$

Grain type.		Q
quartz and feldspar	10	
limestone	8	
anthracite	7	
chalk		5.5

relative state parameter (-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1+2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.516$$

alpha factor (maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 4.151$$

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Checked by TJD

The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 120pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface....very deep

sample depth — Boring B-101 Sample No. 2 — $z := 4ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ — $\sigma tot_v = 480 \cdot psf$

static water pressure $u_o := 0$ — $u_o = 0 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ — $\sigma eff_v = 480 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value $N_m := 8$

fines content $FC := 48$

energy correction $C_E := 0.75$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.75$

sampler correction $C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3

(Skempton, 1986)

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15

(Skempton, 1986)

SPT N-value for an energy ratio of 60% $N_{60} := C_E C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 4.7$

Rod length	CR
<3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$

$C_{N\_a} = 2.1$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ — $N_{1\_60} = 9.9$

overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 2.222$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 2.22$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 10.5$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 2.2$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 16.103$$

Earthquake Conditions:

peak ground accelerationl $a_{max} := 0.25\cdot g$

earthquake moment magnitude $M_w := 5$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\,\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.986$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.16$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor
(maximum MSF = 1.8)

$MSF := 6.9 \cdot \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 1.919$

coefficient for overburden correction factor
(maximum Cσ=0.3)

$C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.094$

overburden correction factor
(maximum Kσ=1.1)

$K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.139$

static shear stress on horizontal plane $\tau_{ho} := 0psf$

effective friction angle $\phi_{eff} := \text{atan}\left[\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}\right]$ $\phi_{eff} = 36.898 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.4$

empirical constant for dilatancy $Q := 10$

Grain type.		Q
quartz and feldspar	10	
limestone	8	
anthracite	7	
chalk		5.5

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relative state parameter (-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1+2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N_{1\_60}}{46}\right)^{0.5} \qquad \zeta_R = -0.342$$

alpha factor (maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 2.261$$

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 120pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface....very deep

sample depth Boring B-110 Sample No. 3 $z := 6ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 720 \cdot psf$

static water pressure $u_o := 0$ $u_o = 0 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 720 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value $N_m := 13$

fines content $FC := 37$

energy correction $C_E := 0.75$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.75$

sampler correction $C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 7.7$

Rod length	CR
<3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

overburden correction factor (based on overburden stress only) $C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$

$C_{N\_a} = 1.714$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 13.2$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.716$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.7$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 13.1$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.717$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.5$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 18.67$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.25\cdot g$$

earthquake moment magnitude

$$M_w := 5$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\,\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.973$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.158$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor
(maximum MSF = 1.8)

$$MSF := 6.9\exp\left(\frac{-M_w}{4}\right) - 0.058 \qquad MSF = 1.919$$

coefficient for overburden correction factor
(maximum Cσ=0.3)

$$C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}} \qquad C_\sigma = 0.104$$

overburden correction factor
(maximum Kσ=1.1)

$$K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right) \qquad K_\sigma = 1.112$$

static shear stress on horizontal plane $\tau_{ho} := 0psf$

effective friction angle

$$\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34} \qquad \phi_{eff} = 41.31 \cdot deg$$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.34$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1+2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N1\_60}{46}\right)^{0.5} \qquad \zeta_R = -0.392$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 2.573$$

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 120pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface....very deep

sample depth (Boring B-110, Sample No. 4) $z := 8ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\sigma tot_v = 960 \cdot psf$

static water pressure $u_o := 0$ $u_o = 0 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\sigma eff_v = 960 \cdot psf$

Standard Penetration N-value Correction -

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Standard Penetration Test N-value $N_m := 40$

fines content $FC := 41$

energy correction $C_E := 0.75$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.75$

sampler correction $C_S := 1$

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 23.6$

Rod length	CR
<3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

overburden correction factor (based on overburden stress only)

$$C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$$

$C_{N\_a} = 1.485$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $N_{1\_60} = 35.1$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.29$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.31$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 30.9$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.319$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 36.533$$

Earthquake Conditions -

peak ground accelerationl

$$a_{max} := 0.25\cdot g$$

earthquake moment magnitude

$$M_w := 5$$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.959$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.156$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor (maximum MSF = 1.8) $MSF := 6.9 \cdot \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 1.919$

coefficient for overburden correction factor (maximum $C\sigma$=0.3) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.212$

overburden correction factor (maximum $K\sigma$=1.1) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.168$

static shear stress on horizontal plane $\tau_{ho} := 0psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 53.879 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.192$

empirical constant for dilatancy $Q := 10$

Grain type.		Q
quartz and feldspar	10	
limestone	8	
anthracite	7	
chalk		5.5

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relative state parameter
($-0.61<\xi R<0.11$)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1+2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N1\_60}{46}\right)^{0.5} \qquad \zeta_R = -0.677$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 22.457$$

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil $\gamma_{total} := 120pcf$

unit weight of water $\gamma_{water} := 62.4pcf$

water table depth below ground surface....very deep

sample depth — Boring B-104, Sample No. 5 — $z := 14ft$

total vertical stress $\sigma tot_v := \gamma_{total} \cdot z$ $\quad \sigma tot_v = 1680 \cdot psf$

static water pressure $u_o := 0$ $\quad u_o = 0 \cdot psf$

effective vertical stress $\sigma eff_v := \sigma tot_v - u_o$ $\quad \sigma eff_v = 1680 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value $N_m := 76$

fines content $FC := 41$

energy correction $C_E := 0.75$

borehole diameter correction $C_B := 1.05$

rod length correction $C_R := 0.85$

sampler correction $C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3
(Skempton, 1986)	

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15
(Skempton, 1986)	

Rod length	CR
<3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

SPT N-value for an energy ratio of 60% $N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$

$N_{60} = 50.9$

overburden correction factor (based on overburden stress only)

$$C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$$

$C_{N\_a} = 1.122$

normalized SPT N-value (based on overburden stress only) $N_{1\_60} := C_{N\_a} \cdot N_{60}$ $\quad N_{1\_60} = 57.1$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 1.046$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 1.05$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 53.4$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 1.051$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 59$$

Earthquake Conditions -

peak ground accelerationl $a_{max} := 0.25\cdot g$

earthquake moment magnitude $M_w := 5$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.911$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.148$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor
(maximum MSF = 1.8) $MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 1.919$

coefficient for overburden correction factor
(maximum $C\sigma = 0.3$) $C_\sigma := \frac{1}{18.9 - 2.55 \cdot \left(N_{1\_60}\right)^{0.5}}$ $C_\sigma = 3.803$

overburden correction factor
(maximum $K\sigma = 1.1$) $K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 1.877$

static shear stress on horizontal plane $\tau_{ho} := 0 psf$

effective friction angle $\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 58.496 \cdot deg$

Ko for NC soil $K_o := \left(1 - \sin\left(\phi_{eff}\right)\right)$ $K_o = 0.147$

empirical constant for dilatancy $Q := 10$

Grain type.	Q
quartz and feldspar	10
limestone	8
anthracite	7
chalk	5.5

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relative state parameter
(-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1 + 2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N1\_60}{46}\right)^{0.5} \qquad \zeta_R = -0.923$$

alpha factor
(maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1.002$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 8.725\times 10^7$$

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The following calculations for liquefaction of soils with sand-like behavior are based on Soil Liquefaction During Earthquakes - Monograph MNO-12 by I.M. Idriss and R.W. Boulanger (2008) published by the Earthquake Engineering Research Institute.

Soil Stress Conditions -

average total unit weight of soil		$\gamma_{total} := 120pcf$	
unit weight of water		$\gamma_{water} := 62.4pcf$	
water table depth below ground surface....very deep			
sample depth	Boring B-101 Sample No. 6	$z := 19ft$	
total vertical stress		$\sigma tot_v := \gamma_{total} \cdot z$	$\sigma tot_v = 2280 \cdot psf$
static water pressure		$u_o := 0$	$u_o = 0 \cdot psf$
effective vertical stress		$\sigma eff_v := \sigma tot_v - u_o$	$\sigma eff_v = 2280 \cdot psf$

Standard Penetration N-value Correction -

Standard Penetration Test N-value	$N_m := 55$
fines content	$F.C. := 41$
energy correction	$C_E := 0.75$
borehole diameter correction	$C_B := 1.05$
rod length correction	$C_R := 0.85$
sampler correction	$C_S := 1$

Hammer	CE
Doughnut	0.5-1.0
Safety	0.7-1.2
Automatic	0.8-1.3

(Skempton, 1986)

Borehole dia.	CB
65-115mm	1.0
150mm	1.05
200mm	1.15

(Skempton, 1986)

SPT N-value for an energy ratio of 60%

$$N_{60} := C_E \cdot C_B \cdot C_R \cdot C_S \cdot N_m$$

$$N_{60} = 36.8$$

Rod length	CR
< 3m	0.75
3-4m	0.80
4-6m	0.85
6-10m	0.95
10-30m	1.00

overburden correction factor (based on overburden stress only)

$$C_{N\_a} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.5}$$

$$C_{N\_a} = 0.963$$

normalized SPT N-value (based on overburden stress only)

$$N_{1\_60} := C_{N\_a} \cdot N_{60}$$

$$N_{1\_60} = 35.5$$

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overburden correction factor (based on overburden stress and relative density)

$$C_{N\_b} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_b} = 0.976$$

overburden correction factor used in liquefaction analysis (max CN = 1.7)

$$C_N := 0.976$$

recalculated normalized SPT N-value

$$N_{1\_60} := C_N \cdot N_{60}$$

$$N_{1\_60} = 35.9$$

overburden correction factor used in liquefaction analysis

$$C_{N\_check} := \left(\frac{P_a}{\sigma eff_v}\right)^{0.784-0.078\cdot\left(N_{1\_60}\right)^{0.5}}$$

$$C_{N\_check} = 0.977$$

fines content adjustment to SPT N-value

$$\Delta N_{1\_60} := \exp\left[1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right]$$

$$\Delta N_{1\_60} = 5.6$$

normalized SPT N-value (clean sands)

$$N_{1\_60\_cs} := N_{1\_60} + \Delta N_{1\_60}$$

$$N_{1\_60\_cs} = 41.516$$

Earthquake Conditions -

peak ground accelerationl $a_{max} := 0.25\cdot g$

earthquake moment magnitude $M_w := 5$

variable α for stress reduction factor

$$\alpha := -1.012 - 1.126\cdot\sin\left(\frac{\frac{z}{3.28ft}}{11.73} + 5.133\right)$$

variable β for stress reduction factor

$$\beta := 0.106 + 0.118\sin\left(\frac{\frac{z}{3.28ft}}{11.28} + 5.142\right)$$

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stress reduction factor $r_d := \exp(\alpha + \beta \cdot M_w)$ $r_d = 0.868$

Liquefaction Calculations -

cyclic stress ratio $CSR := 0.65 \cdot \frac{\sigma tot_v}{\sigma eff_v} \cdot \frac{a_{max}}{g} \cdot r_d$ $CSR = 0.141$

cyclic resistance ratio

$$CRR_{M7.5\_\sigma eff1} := \exp\left[\frac{N_{1\_60\_cs}}{14.1} + \left(\frac{N_{1\_60\_cs}}{126}\right)^2 - \left(\frac{N_{1\_60\_cs}}{23.6}\right)^3 + \left(\frac{N_{1\_60\_cs}}{25.4}\right)^4 - 2.8\right]$$

magnitude scaling factor
(maximum MSF = 1.8)

$MSF := 6.9 \exp\left(\frac{-M_w}{4}\right) - 0.058$ $MSF = 1.919$

coefficient for overburden correction factor
(maximum Cσ=0.3)

$C_\sigma := \frac{1}{18.9 - 2.55 \cdot (N_{1\_60})^{0.5}}$ $C_\sigma = 0.277$

overburden correction factor
(maximum Kσ=1.1)

$K_\sigma := 1 - C_\sigma \cdot \ln\left(\frac{\sigma eff_v}{P_a}\right)$ $K_\sigma = 0.979$

static shear stress on horizontal plane $\tau_{ho} := 0psf$

effective friction angle

$\phi_{eff} := \operatorname{atan}\left(\frac{N_{60}}{12.2 + 20.3 \cdot \frac{\sigma eff_v}{P_a}}\right)^{0.34}$ $\phi_{eff} = 53.647 \cdot deg$

Ko for NC soil $K_o := (1 - \sin(\phi_{eff}))$ $K_o = 0.195$

empirical constant for dilatancy $Q := 10$

Grain type.		Q
quartz and feldspar	10	
limestone	8	
anthracite	7	
chalk		5.5

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relative state parameter (-0.61<ξR<0.11)

$$\zeta_R := \frac{1}{Q - \ln\left[\frac{100\cdot(1+2K_o)\sigma eff_v}{3\cdot P_a}\right]} - \left(\frac{N1\_60}{46}\right)^{0.5} \qquad \zeta_R = -0.72$$

alpha factor (maximum α=0.35)

$$\alpha\_factor := \frac{\tau_{ho}}{\sigma eff_v} \qquad \alpha\_factor = 0$$

$$a := 1267 + 636\alpha\_factor^2 - 634\cdot\exp(\alpha\_factor) - 632\cdot\exp(-\alpha\_factor)$$

$$b := \exp\left(-1.11 + 12.3\alpha\_factor^2 + 1.31\cdot\ln(\alpha\_factor + 0.0001)\right)$$

$$c := 0.138 + 0.126\cdot\alpha\_factor + 2.52\cdot\alpha\_factor^3$$

static shear stress correction factor

$$K_\alpha := a + b\cdot\exp\left(\frac{-\zeta_R}{c}\right) \qquad K_\alpha = 1$$

corrected cyclic stress ratio

$$CRR_{M\_\sigma effv} := CRR_{M7.5\_\sigma eff1}\cdot MSF\cdot K_\sigma\cdot K_\alpha$$

factor of safety for liquefaction

$$FS_{liq} := \frac{CRR_{M\_\sigma effv}}{CSR} \qquad FS_{liq} = 93.321$$

GEOServices, LLC, Geotechnical and Materials Engineers

APPENDIX G

Sample Bearing Capacity Calculations

Bearing Capacity Calculation

CISF Storage Pad

Date:	2/17/2020
Project:	CISF Cask Storage Pads
Location:	Andrews, Texas
Project No:	31-151247.R2

BEARING CAPACITY CALCULATIONS

Vesic Bearing Capacity Formulas

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

where,

q_{ult}	=	ultimate bearing capacity
c'	=	effective cohesion for soil beneath foundation
ϕ'	=	effective friction angle for soil beneath foundation
σ'_{ZD}	=	vertical effective stress at depth D below ground surface
γ'	=	effective unit weight of the soil
D	=	depth of foundation below ground surface
B	=	width of foundation
L	=	length of foundation
N_c, N_q, N_γ	=	Vesic bearing capacity factors = $f(\phi')$, factors follow
s_c, s_q, s_γ	=	shape factors
d_c, d_q, d_γ	=	depth factors
i_c, i_q, i_γ	=	load inclination factors, not applicable in this analysis
b_c, b_q, b_γ	=	base inclination factors, not applicable in this analysis
g_c, g_q, g_γ	=	ground inclination factors, not applicable in this analysis

Vesic Bearing Capacity Factors											
ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ
11	8.8	2.7	1.4	21.0	15.8	7.1	6.2	31.0	32.7	20.6	26.0
12	9.3	3.0	1.7	22.0	16.9	7.8	7.1	32.0	35.5	23.2	30.2
13	9.8	3.3	2.0	23.0	18.0	8.7	8.2	33.0	38.6	26.1	35.2
14	10.4	3.6	2.3	24.0	19.3	9.6	9.4	34.0	42.2	29.4	41.1
15	11.0	3.9	2.6	25.0	20.7	10.7	10.9	35.0	46.1	33.3	48.0
16	11.6	4.3	3.1	26.0	22.3	11.9	12.5	36.0	50.6	37.8	56.3
17	12.3	4.8	3.5	27.0	23.9	13.2	14.5	37.0	55.6	42.9	66.2
18	13.1	5.3	4.1	28.0	25.8	14.7	16.7	38.0	61.4	48.9	78.0
19	13.9	5.8	4.7	29.0	27.9	16.4	19.3	39.0	67.9	56.0	92.2
20	14.8	6.4	5.4	30.0	30.1	18.4	22.4	40.0	75.3	64.2	109.4

Project Specific Information:

The CISF Pads will bear at an elevation of 4 feet below ground surface. The pads will consist of a reinforced concrete mat measuring 36 inches in thickness. Plan dimensions of the mat are 135 feet by 55 feet. The CISF Pads are anticipated to bear in the Caliche with Sand Matrix. An effective friction angle of 27 degrees and an effective unit weight of 95 pounds per cubic foot were utilized for the calculation.

c'	=	0	psf
ϕ'	=	27	degrees
σ'_{ZD}	=	380	psf
γ'	=	95	pcf
D	=	4	feet
B	=	55	feet
L	=	135	feet

Step 1: Calculate Shape Factors

$s_c = 1 + (B/L)(N_q/N_c)$ $s_c =$ 1.225

$s_q = 1 + (B/L)\tan\phi'$ $s_q =$ 1.208

$s_\gamma = 1 - 0.4(B/L)$ $s_\gamma =$ 0.837

Step 2: Calculate Depth Factors

$d_c = 1 + 0.4k$ where $k = D/B$ $d_c =$ 1.029

$d_q = 1 + 2k\tan\phi'(1-\sin\phi')^2$ $d_q =$ 1.022

$d_\gamma = 1$ $d_\gamma =$ 1.000

Step 3: Calculate Load Inclination Factors

Since the loads act perpendicular to the base of the footing, the I factors equal 1 and may be neglected.

Step 4: Calculate the Base Inclination Factors

Since the base of the footing is level, all of the b factors equal to 1 and may be neglected.

Step 5: Calculate the Ground Inclination Factors

Since the ground surfaceis level the g factors equal to 1 and may be neglected.

Step 6: Calculate the Ultimate Bearing Capacity

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

removing the values which were equal to 1 results in the following:

$q_{ult} = c'N_c s_c d_c + \sigma'_{ZD} N_q s_q d_q + 0.5\Upsilon' B N_\Upsilon s_\Upsilon d_\Upsilon$

q_{ult} = 37899 psf

Step 7: Calculate the Allowable Bearing Capacity

$q_{all} = q_{ult}/FOS$ Factor of Safety of 3 utilized for bearing capacity

q_{all} = 12633 psf

Based on the loading provided by Enercon the CISF Mat Foundations will impart a maximum pressure of 4,500 psf to the subgrade

12633 > 4,500 Bearing Capacity OK

You will note that the calculated allowable bearing capacities often exceed those provided in the text of the report. Typically, on structures which are supported on shallow foundations bearing capacity (of the soil) does not control the foundation size.

GEOS

Bearing Capacity Calculation

Cask Handling Building – Main Columns

Date:	2/17/2020
Project:	CISF - Cask Handling Building (Main Columns)
Location:	Andrews, Texas
Project No:	31-151247.R2

BEARING CAPACITY CALCULATIONS

Vesic Bearing Capacity Formulas

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

where,

q_{ult}	=	ultimate bearing capacity
c'	=	effective cohesion for soil beneath foundation
ϕ'	=	effective friction angle for soil beneath foundation
σ'_{ZD}	=	vertical effective stress at depth D below ground surface
γ'	=	effective unit weight of the soil
D	=	depth of foundation below ground surface
B	=	width of foundation
L	=	length of foundation
N_c, N_q, N_γ	=	Vesic bearing capacity factors = f(ϕ'), factors follow
s_c, s_q, s_γ	=	shape factors
d_c, d_q, d_γ	=	depth factors
i_c, i_q, i_γ	=	load inclination factors, not applicable in this analysis
b_c, b_q, b_γ	=	base inclination factors, not applicable in this analysis
g_c, g_q, g_γ	=	ground inclination factors, not applicable in this analysis

Vesic Bearing Capacity Factors											
ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ
11	8.8	2.7	1.4	21.0	15.8	7.1	6.2	31.0	32.7	20.6	26.0
12	9.3	3.0	1.7	22.0	16.9	7.8	7.1	32.0	35.5	23.2	30.2
13	9.8	3.3	2.0	23.0	18.0	8.7	8.2	33.0	38.6	26.1	35.2
14	10.4	3.6	2.3	24.0	19.3	9.6	9.4	34.0	42.2	29.4	41.1
15	11.0	3.9	2.6	25.0	20.7	10.7	10.9	35.0	46.1	33.3	48.0
16	11.6	4.3	3.1	26.0	22.3	11.9	12.5	36.0	50.6	37.8	56.3
17	12.3	4.8	3.5	27.0	23.9	13.2	14.5	37.0	55.6	42.9	66.2
18	13.1	5.3	4.1	28.0	25.8	14.7	16.7	38.0	61.4	48.9	78.0
19	13.9	5.8	4.7	29.0	27.9	16.4	19.3	39.0	67.9	56.0	92.2
20	14.8	6.4	5.4	30.0	30.1	18.4	22.4	40.0	75.3	64.2	109.4

Project Specific Information:

The foundations for the Cask Handling Building will bear at an elevation of 10 to 11 feet below ground surface. The foundations for the main columns measure 28 feet by 183 feet while the foundations for the wind columns measure 26 feet 3 inches by 40 feet. The foundations for the cask handling building are anticipated to bear in the Caliche with Sand Matrix. An effective friction angle of 35 degrees and an effective unit weight of 130 pounds per cubic foot were utilized for the calculation.

c'	=	0	psf
ϕ'	=	35	degrees
σ'_{ZD}	=	1300	psf
γ'	=	130	pcf
D	=	10	feet
B	=	28	feet
L	=	183	feet

Step 1: Calculate Shape Factors

$s_c = 1 + (B/L)(N_q/N_c)$ $s_c =$ 1.111

$s_q = 1 + (B/L)\tan\phi'$ $s_q =$ 1.285

$s_\gamma = 1 - 0.4(B/L)$ $s_\gamma =$ 0.837

Step 2: Calculate Depth Factors

$d_c = 1 + 0.4k$ where $k = D/B$ $d_c =$ 1.143

$d_q = 1 + 2k\tan\phi'(1-\sin\phi')^2$ $d_q =$ 1.091

$d_\gamma = 1$ $d_\gamma =$ 1.000

Step 3: Calculate Load Inclination Factors

Since the loads act perpendicular to the base of the footing, the I factors equal 1 and may be neglected.

Step 4: Calculate the Base Inclination Factors

Since the base of the footing is level, all of the b factors equal to 1 and may be neglected.

Step 5: Calculate the Ground Inclination Factors

Since the ground surfaceis level the g factors equal to 1 and may be neglected.

Step 6: Calculate the Ultimate Bearing Capacity

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

removing the values which were equal to 1 results in the following:

$q_{ult} = c'N_c s_c d_c + \sigma'_{ZD} N_q s_q d_q + 0.5\Upsilon' B N_\Upsilon s_\Upsilon d_\Upsilon$

q_{ult} = 133823 psf

Step 7: Calculate the Allowable Bearing Capacity

$q_{all} = q_{ult}/FOS$ Factor of Safety of 3 utilized for bearing capacity

q_{all} = 44608 psf

Based on the loading provided by AECOM the Cask Handling Building Main Columns will impart a maximum service level pressure of 3,340 psf to the subgrade

44608 > 3,340 Bearing Capacity OK

You will note that the calculated allowable bearing capacities often exceed those provided in the text of the report. Typically, on structures which are supported on shallow foundations bearing capacity (of the soil) does not control the foundation size.

Bearing Capacity Calculation

Cask Handling Building – Wind Columns

Date:	2/17/2020
Project:	CISF - Cask Handling Building (Wind Columns)
Location:	Andrews, Texas
Project No:	31-151247.R2

BEARING CAPACITY CALCULATIONS

Vesic Bearing Capacity Formulas

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

where,

q_{ult}	=	ultimate bearing capacity
c'	=	effective cohesion for soil beneath foundation
ϕ'	=	effective friction angle for soil beneath foundation
σ'_{ZD}	=	vertical effective stress at depth D below ground surface
Υ'	=	effective unit weight of the soil
D	=	depth of foundation below ground surface
B	=	width of foundation
L	=	length of foundation
N_c, N_q, N_Υ	=	Vesic bearing capacity factors = f(ϕ'), factors follow
s_c, s_q, s_Υ	=	shape factors
d_c, d_q, d_Υ	=	depth factors
i_c, i_q, i_Υ	=	load inclination factors, not applicable in this analysis
b_c, b_q, b_Υ	=	base inclination factors, not applicable in this analysis
g_c, g_q, g_Υ	=	ground inclination factors, not applicable in this analysis

Vesic Bearing Capacity Factors											
ϕ' (deg)	N_c	N_q	N_Υ	ϕ' (deg)	N_c	N_q	N_Υ	ϕ' (deg)	N_c	N_q	N_Υ
11	8.8	2.7	1.4	21.0	15.8	7.1	6.2	31.0	32.7	20.6	26.0
12	9.3	3.0	1.7	22.0	16.9	7.8	7.1	32.0	35.5	23.2	30.2
13	9.8	3.3	2.0	23.0	18.0	8.7	8.2	33.0	38.6	26.1	35.2
14	10.4	3.6	2.3	24.0	19.3	9.6	9.4	34.0	42.2	29.4	41.1
15	11.0	3.9	2.6	25.0	20.7	10.7	10.9	35.0	46.1	33.3	48.0
16	11.6	4.3	3.1	26.0	22.3	11.9	12.5	36.0	50.6	37.8	56.3
17	12.3	4.8	3.5	27.0	23.9	13.2	14.5	37.0	55.6	42.9	66.2
18	13.1	5.3	4.1	28.0	25.8	14.7	16.7	38.0	61.4	48.9	78.0
19	13.9	5.8	4.7	29.0	27.9	16.4	19.3	39.0	67.9	56.0	92.2
20	14.8	6.4	5.4	30.0	30.1	18.4	22.4	40.0	75.3	64.2	109.4

Project Specific Information:

The foundations for the Cask Handling Building will bear at an elevation of 10 to 11 feet below ground surface. The foundations for the main columns measure 28 feet by 183 feet while the foundations for the wind columns measure 26 feet 3 inches by 40 feet. The foundations for the cask handling building are anticipated to bear in the Caliche with Sand Matrix. An effective friction angle of 35 degrees and an effective unit weight of 130 pounds per cubic foot were utilized for the calculation.

c'	=	0	psf
ϕ'	=	35	degrees
σ'_{ZD}	=	1300	psf
γ'	=	130	pcf
D	=	10	feet
B	=	26.25	feet
L	=	40	feet

Step 1: Calculate Shape Factors

$s_c = 1 + (B/L)(N_q/N_c)$ $s_c =$ 1.474

$s_q = 1 + (B/L)\tan\phi'$ $s_q =$ 1.285

$s_\gamma = 1 - 0.4(B/L)$ $s_\gamma =$ 0.837

Step 2: Calculate Depth Factors

$d_c = 1 + 0.4k$ where $k = D/B$ $d_c =$ 1.152

$d_q = 1 + 2k\tan\phi'(1-\sin\phi')^2$ $d_q =$ 1.097

$d_\gamma = 1$ $d_\gamma =$ 1.000

Step 3: Calculate Load Inclination Factors

Since the loads act perpendicular to the base of the footing, the I factors equal 1 and may be neglected.

Step 4: Calculate the Base Inclination Factors

Since the base of the footing is level, all of the b factors equal to 1 and may be neglected.

Step 5: Calculate the Ground Inclination Factors

Since the ground surfaceis level the g factors equal to 1 and may be neglected.

Step 6: Calculate the Ultimate Bearing Capacity

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

removing the values which were equal to 1 results in the following:

$q_{ult} = c'N_c s_c d_c + \sigma'_{ZD} N_q s_q d_q + 0.5\Upsilon' B N_\Upsilon s_\Upsilon d_\Upsilon$

q_{ult} = 129590 psf

Step 7: Calculate the Allowable Bearing Capacity

$q_{all} = q_{ult}/FOS$ Factor of Safety of 3 utilized for bearing capacity

q_{all} = 43197 psf

Based on the loading provided by AECOM the Cask Handling Building Wind Columns will impart a maximum service level pressure of 1,860 psf to the subgrade

43197 > 1,860 Bearing Capacity OK

You will note that the calculated allowable bearing capacities often exceed those provided in the text of the report. Typically, on structures which are supported on shallow foundations bearing capacity (of the soil) does not control the foundation size.

GEOS

GEOServices, LLC, Geotechnical and Materials Engineers

APPENDIX H

Settlement Calculations

DAN BROWN AND ASSOCIATES

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(423)942-8681
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TECHNICAL MEMORANDUM

Settlement Analyses
CISF Storage Pad Site
Andrews TX
DBA Project No. 19-017

To:	Derek Kilday, P.E./GEOServices
From:	Timothy C. Siegel, P.E, G.E., D.GE Tayler J. Day, P.E.
Date:	17 February 2020

STATE OF TEXAS
TIMOTHY C. SIEGEL
117477
LICENSED
PROFESSIONAL ENGINEER

1. Introduction

This Technical Memorandum (TM) presents the results of settlement analyses for the waste interim storage pad system at the subject site in Andrews, Texas. Subsurface conditions were described as a soil column in correspondence from GEOServices as part of their revised recommendations to be included in the final geotechnical report. Loading and pad dimensions were provided to DBA by Enercon. The remaining sections of this TM briefly describe the soil column, analysis considerations and results.

2. Soil Column Used for Analysis

The settlement analyses were performed based on soil properties and stresses applied to the ground by the proposed structure. Geotechnical explorations were performed by AECOM[1] and GEOServices[2]. Boring information collected by GEOServices extends to auger refusal at approximately 45 feet below the ground surface that existed at time of exploration. Typical settlement analyses consider a depth of influence of twice the least dimension of the loaded area. DBA understands that for this project, the team prefers to consider a greater depth. DBA's analyses consider the soil column to 600 feet below ground surface (top of the Trujillo Sandstone Formation) with the layers presented in Table 1.

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[1] AECOM. (2016). Site-Specific Seismic Hazard Evaluation and Development of Seismic Design Ground Motions, WCS Centralized Interim Storage Facility Project. Dated March 18, 2016.

[2] GEOServices (2018) Report of Geotechnical Exploration: Consolidated Interim Storage Facility (CISF) Andrews, Texas Geoservices,, LLC Project No. 31-151247.R1. Submitted to Waste Control Specialists, LLC. Dated 15 July 2016

Table 1 - Soil column layer information provided in revised GEOServices Report of Geotechnical Exploration

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

3. Settlement Analysis Parameters

Settlement analyses were performed using Settle3 ver 5.001, a 3-dimensional program for the analysis of vertical consolidation and settlement under foundations and surface loads. Settle3 is designed to compute both immediate compression settlement and settlement due to consolidation. Immediate settlement occurs when a load is applied to materials that can be assumed to behave linear elastically. In this case, the composition of the soil column and relative stiffness (reported in boring logs and shear wave velocity profiles) coupled with the absence of a consistent high groundwater table identifies Immediate Settlement as the dominant deflection mechanism. Consolidation occurs in materials where excess pore pressures gradually dissipate. As there is no consistent permanent water table observed in the extensive geotechnical exploration of the areas, excess porewater pressures are unlikely to be generated so DBA concludes that primary consolidation will not significantly contribute to storage pad deflections in the reported conditions. The stress computation method used in this analysis was the Westergaard Solution for consideration of multiple layers with Poisson's ratio inputs.

Immediate settlement can be estimated in Settle3 using constrained modulus to represent the compressibility of the geotechnical material. Constrained modulus was not directly provided in the provided geotechnical explorations. The constrained modulus of soil was determined using a correlation with average standard penetration test (SPT) N-value that was proposed by Tan et al.[3] The average SPT N-values (excluding refusal) in the GEOServices borings were compared to the "Normally Loaded Sand or Sand and Gravel" relationship to estimate the constrained modulus using the highest resolution measurements available. The majority of the SPT N-values

[3] Tan, C.K., Duncan, J.M., Rojiani, K.B., and Barker, R.M.(1991) Engineering Manual for Shallow Foundations, prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) Sponsored by AASHTO and FHWA, Washington, D.C., 171pp

measured in the remaining layers of the GEOServices borings were larger than 70 blows/ft and are therefore considered outside the range of the correlation.

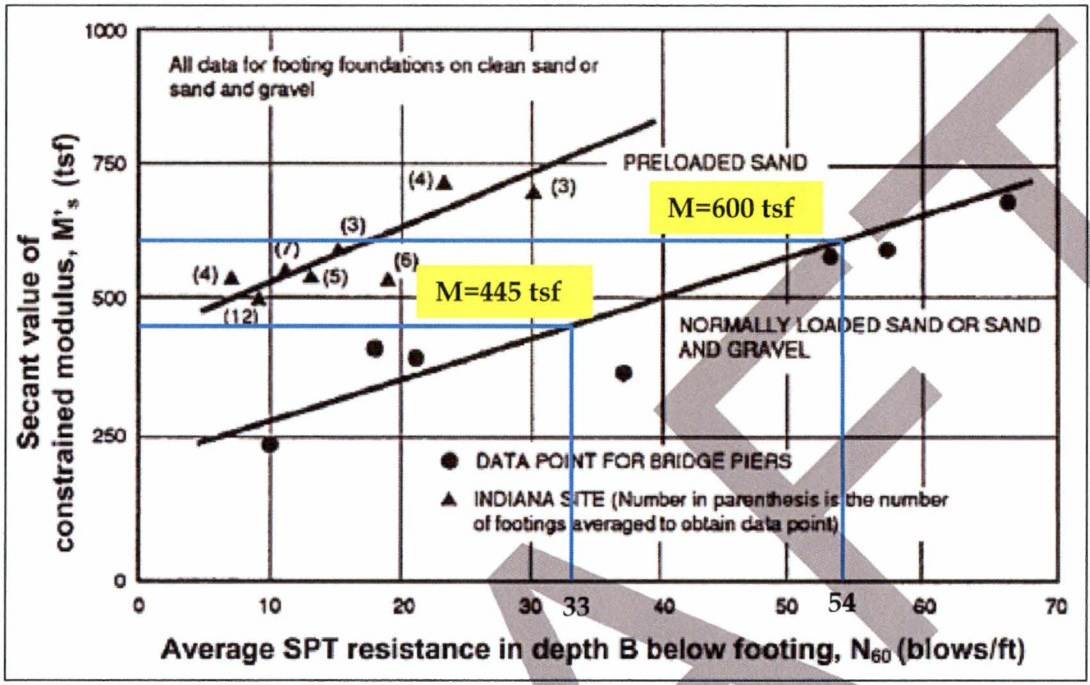

Figure 1 - Relationship between SPT resistance and constrained modulus used for top 20 feet of soil column (after Tan et al., 1991)

A summary of the estimated constrained moduli for the top 20 feet of the soil column is presented in Table 2. Several refusal SPT N-values in the top 20 feet were intentionally excluded from the average in recognition of potential reductions in modulus due to inconsistent or partial cementation in the Caliche Sand matrix as described to DBA by GEOServices.

Table 2 - Correlated constrained modulus values for top 20feet of soil column using Tan et al. (1991).

Top (feet)	Bottom (feet)	Average N-Value (bpf)	Layer Material (from GEOS Soil Column)	Constrained Modulus (tsf) from Tan et al. (1991) correlation
0	2	33	Cover Sands	445
2	10	54	Caliche with Sand Matrix - Moderately Hard	600
10	20	54	Caliche with Sand Matrix - Moderately Hard	600

Below a depth of 20feet, DBA relied upon the shear wave velocity profiles collected by GEOServices and the Site-Specific Seismic Hazard Evaluation and Development of Seismic Ground Motions prepared by AECOM (2016). The GEOServices study targeted the top 100feet of the profile while the AECOM study collected shear wave velocity measurements in the area to depths of 600feet below ground surface. Each layer in the soil column between 20feet and 100feet below ground surface were assigned average shear wave velocities using the GEOServices shear wave velocity measurements. Layers between 100feet and 600feet below ground surface were assigned average shear wave velocities using the AECOM measurements. A summary of the layers, their associated Settle3 model layer names, and the average shear wave velocities is provided in Table 3.

Table 3 - Average shear wave velocities assigned to each soil column layer between 20 feet and 600 feet below ground surface.

Top (feet)	Bottom (feet)	Avg. Layer Shear Wave Velocity (ft/s)	Layer Material (from GEOS Column)
20	25	1530	Caliche - Very Hard
25	35	1900	Caliche - Very Hard
35	50	2290	Ogallala - Sand with Gravel
50	80	1840	Ogallala - Sand with Gravel
80	100	2790	Ogallala - Sand with Gravel
100	130	2300	Dockum - Claystone and Siltstone
130	230	2755	Claystone and Siltstone
230	275	2755	Dockum - Claystone
275	300	2755	Dockum - Silty Sands
300	360	2755	Dockum - Claystone
360	600	3115	Dockum - Claystone

Average shear wave velocities were then 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, Vs = shear wave velocity;
G = shear modulus;
M = constrained modulus;
ν = Poisson's ratio (from AECOM report); and,
ρ = unit weight.

The correlations between shear wave velocity and moduli are based on small-strain wave theory which is reasonable for use in engineering analysis of settlement of these hard/stiff layers which are unlikely to experience large strains as a result of the pressures exerted by the storage pads. A summary of the layers and modulus values used in this Settle 3 soil column is presented in Table 5.

Table 4 - Soil Layers and Parameters used in Settle3

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	1200
20	25		1530	Caliche - Very Hard	35815
25	35		1900	Caliche - Very Hard	55232
35	50		2290	Ogallala - Sand with Gravel	80233
50	80		1840	Ogallala - Sand with Gravel	53870
80	100		2790	Ogallala - Sand with Gravel	123857
100	130		2300	Dockum - Claystone and Siltstone	84172
130	230		2755	Claystone and Siltstone	120769
230	275		2755	Dockum - Claystone	120769
275	300		2755	Dockum - Silty Sands	120679
300	360		2755	Dockum - Claystone	120679
360	600		3115	Dockum - Claystone	154394

4. Single Storage Pad Analysis and Results

The first objective of the settlement analyses was to perform multiple iterations between Settle3 and GTSTRUDL for four construction conditions of a single storage pad (135ft by 55ft in plan dimension) to develop appropriate bearing pressures and structural loads for pad design. Considered single storage pad configurations are summarized in Table 5.

Table 5 - Summary of analysis conditions and loading.

Condition	Loading
Configuration 1	Single Pad – 24 Loaded Casks
Configuration 2	Single Pad – 6 Loaded Casks
Configuration 3	Single Pad – 12 Loaded Casks
Configuration 4	Single Pad – 18 Loaded Casks

The first iteration of the settlement analysis was performed using the dead loads of the casks provided in WCS Consolidated Interim Storage Facility System Safety Analysis Report (Revision 2- June 2018) and the estimated footing weight. The resulting bearing pressures were used to develop an initial Settle3 model for each configuration using the soil column described previously with the end goal of estimating settlement and values of subgrade modulus k (psf/in). The calculated values of subgrade modulus were then submitted to Enercon to be integrated into the GTSTRUDL structural analysis of the storage pad. The resulting load distribution reported by Enercon was then a more accurate estimate of soil response thus refining the slab pressure distribution. The results of the refined slab analysis were provided to DBA and used to update the Settle3 model resulting in revised subgrade modulus values.

4.1. Configurations 1-4 Loading Information and Iteration

After initial modeling of the dead loads applied directly to the soil column, DBA used the loading information provided by Enercon in the form of GTSTRUDL output of loads at individual node locations. DBA understands that Enercon grouped nodes into zones of similar modulus of subgrade reaction as identified in the initial Settle3 model (20+ zones for each configuration shown in the attachments). When revised loading was returned, DBA integrated the nodal loads in each zone to determine the revised zone bearing pressures. This approach considers effects of mat stiffness on the distribution of stresses to the soil. The resulting bearing pressures were then updated in the Settle3 model and revised values of modulus of subgrade reaction were determined for each zone. This iterative process continued for Configurations 1-4 until the change in modulus of subgrade reaction values was less than or equal to 10% when compared to the values resulting from the previous iteration. Generally, the zones identified in the analysis were one of four groups:

- Loaded Casks
- Mat Edges
- Intermediate (between casks) Areas
- Indirectly Loaded (no casks) Areas for Configurations 2, 3 and 4

The final pressure distributions for each configuration can be found in the attachments.

4.2. Configurations 1-4 Settlement Analysis Results

Computed settlements for Configurations 1-4 are summarized in Table 6. Maximum estimated settlements for all configurations occur under the center of the cask nearest the center of the loaded group. Settlements decrease radially from the heavier loaded cask zones. This behavior matches the expected behavior of a loaded mat. Settlement plots for each configuration and the inputs used for Settle3 are attached to this TM. It is important to note that the results of geotechnical settlement models are estimates that are input dependent. Therefore, settlement recommendations should be based on a combination of calculations and experience that acknowledges the calculations are not precise to the multiple decimal places reported by the program.

Table 6 - Summary of single pad settlement estimates.

Condition	Loading	Estimated maximum settlement (in.)
Configuration 1	Single Pad – 24 Loaded Casks	0.7
Configuration 2	Single Pad – 6 Loaded Casks	0.6
Configuration 3	Single Pad – 12 Loaded Casks	0.7
Configuration 4	Single Pad – 18 Loaded Casks	0.7

The Settle3 bearing pressure input files report a lower bearing pressure than the calculated zone bearing pressure in the attachments. To avoid loading edge effects caused by drastic changes in bearing pressure over very small distance, the bearing pressure zones input into Settle3 are cumulative. An example of this concept is shown in Figure 2.

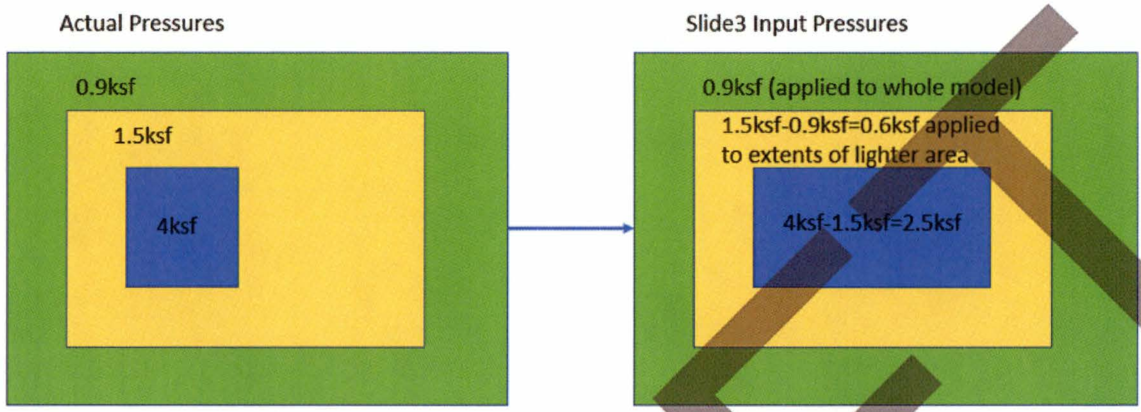

Figure 2 - Example of Slide3 bearing pressure input determination

5. Multiple Pad Analysis and Results

The second objective of the settlement analyses is to estimate the settlement of a configuration of four fully loaded pads in their final constructed condition. The converged Configuration 1 loads were applied in an orientation of four pads spaced per Figure 1-6 in the WCS CISF Safety Analysis Report. The maximum predicted settlement for the four pad scenario is similar to the single pad Configuration 1 case (0.7inches at the center of each pad). The main effect of the adjacent loaded pads is an increase of settlement estimated at the corner of the pads (0.2inches compared to 0.1inches), and the related slight decrease in predicted differential settlement between the center of the footings and the corners of the pads. This behavior results from overlapping stress influence from the loaded footings, which is consistent with predicted behavior. Inputs and results of the model are shown in the attachments.

6. Cask Handling Building Analysis and Results

DBA understands that an auxiliary structure will be utilized for handling the filling of the casks before they are moved to the storage pads. According to Chapter 7 of the WCS CISF Safety Analysis report, the Cask Handling Building (CHB) is a two-bay steel structure measuring 175ft by 193ft with a height of 72ft. Based on a preliminary foundation layout plan for the building, DBA understands the main column footings and wind column footings will be constructed to bear 10ft below ground surface. The soil column developed for storage pad analysis was used to estimate settlements of the CHB.

Based on provided loading information, DBA understands the maximum service level bearing pressure is approximately 3.5ksf or less with maximum limit state bearing pressures approaching 5.5ksf. It is standard practice in geotechnical shallow foundation design to analyze settlements for sustained loading and service loading so two models were developed for the CHB. Two dead load only cases were analyzed: 1.0 DL (1.79ksf for main column footings and 1.67ksf for wind column footings); and a net bearing pressure from Dead loading case (0.66ksf for main column footings and 0.55ksf for wind column footings. The resulting settlements for both dead load cases are 0.25inches or less at the center of the footings. Estimated settlement for the maximum service load (3.5ksf for all footings) case is 0.5inches or less at the center of the footings.

7. Concluding Remarks

Please contact the following if you would like to discuss this document or this project.

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Attachments

Summary of Analysis Basis.
GEOS Blow Count Profiles.
GEOS Shear Wave Velocity Profiles.
AECOM Shear Wave Velocity Profile.
Enercon Model Configurations.
Configuration 1 Final Bearing Zones and Pressures, Settlement Results, and Modulus of Subgrade Reaction Values for Enercon Models.
Configuration 1 Settle3 Inputs .
Configuration 2 Final Bearing Zones and Pressures, Settlement Results, and Modulus of Subgrade Reaction Values for Enercon Models.
Configuration 2 Settle3 Inputs .
Configuration 3 Final Bearing Zones and Pressures, Settlement Results, and Modulus of Subgrade Reaction Values for Enercon Models.
Configuration 3 Settle3 Inputs .
Configuration 4 Final Bearing Zones and Pressures, Settlement Results, and Modulus of Subgrade Reaction Values for Enercon Models
Configuration 4 Settle3 Inputs.
Overall Pad Layout From WCS CISF Safety Analysis Report
Four Pad Settle3 Analysis, Inputs and Results.
Cask Handling Building Settle3 Analysis, Inputs, and Results.

CISF Site
Andrews, TX
Settlement Analysis
Methodology

Problem Statement

- Nuclear waste storage pads
- Settlement concerns so calculation of footing settlement required

DBA Approach

- Utilize extensive shear wave velocity data and soil boring explorations to create a constrained modulus profile
    - In general, the soil profile is caliche & sand with gravel to a depth of ~100ft overlying rock
- Convert SPT N-Values to constrained modulus using Tan, C. K., Duncan, J. M., Rojiani, K. B., and Barker, R. M. (1991) in top 20 feet.
- Convert the shear wave velocity to constrained modulus (M)

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

- Create the constrained modulus profile combining the SPT data from 0 to 20 ft depth with the shear wave velocity data beyond 20 ft depth
- Use Westergaard's stress calculation method and the constrained modulus profile to calculate settlement in *Settle3 by Rocscience*
    - Loading information provided by the structural

CISF Site
Andrews, TX
Settlement Analysis
Methodology

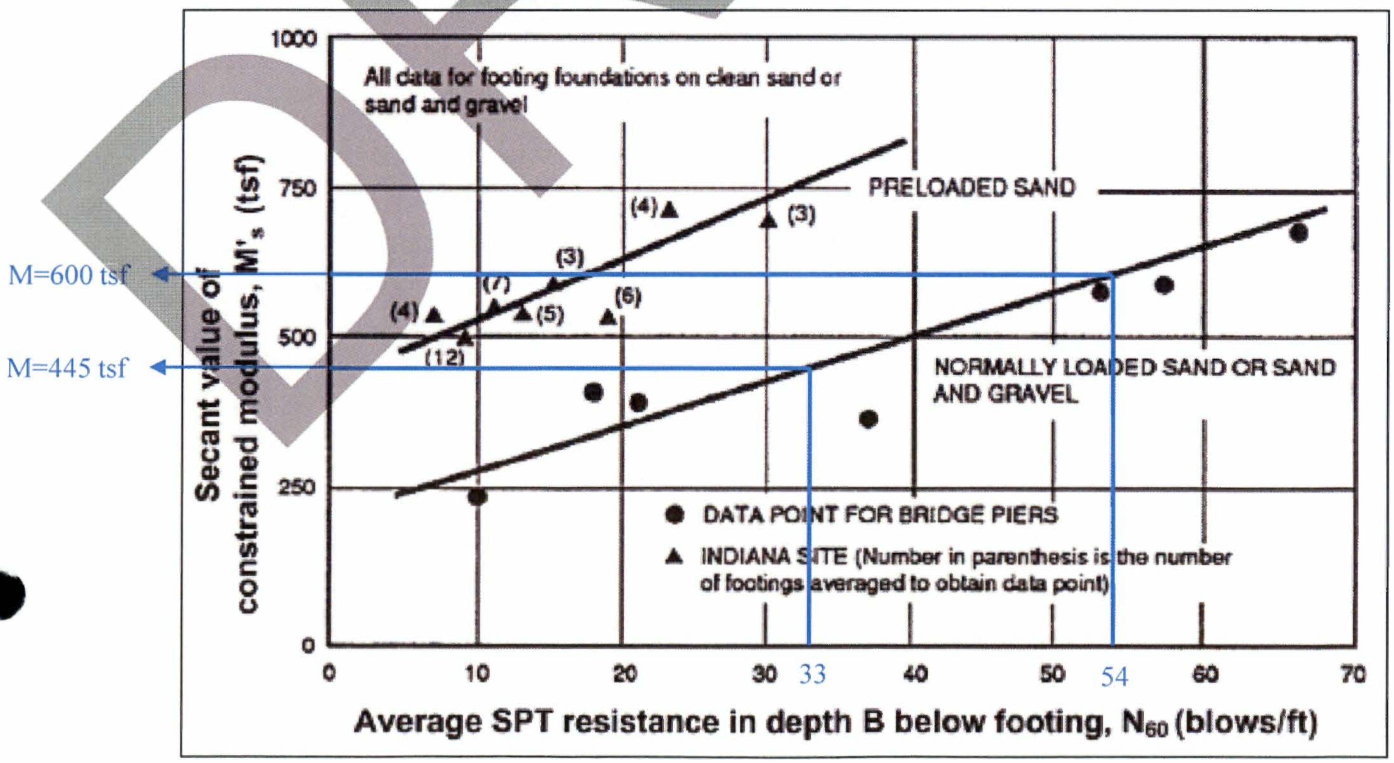
Layers based on Borings Performed by GEOS

- Interpreted from GEOS borings – data only to 45 ft depth
- Layers below 20 ft will be determined using the shear wave velocity data. SPT N-Values from 20 to 45 ft are beyond the range of the correlation shown below

Top of Layer (ft)	Bottom of Layer (ft)	N-Value (bpf)	Layer Material (From GEOS Column)	Constrained Modulus (tsf)
0	2	33	Cover Sands	445
2	10	54	Caliche w/ Sand Matrix – Mod. Hard	600
10	20	54	Caliche w/ Sand Matrix – Mod. Hard	600

SPT N-Value to Constrained Modulus Correlation

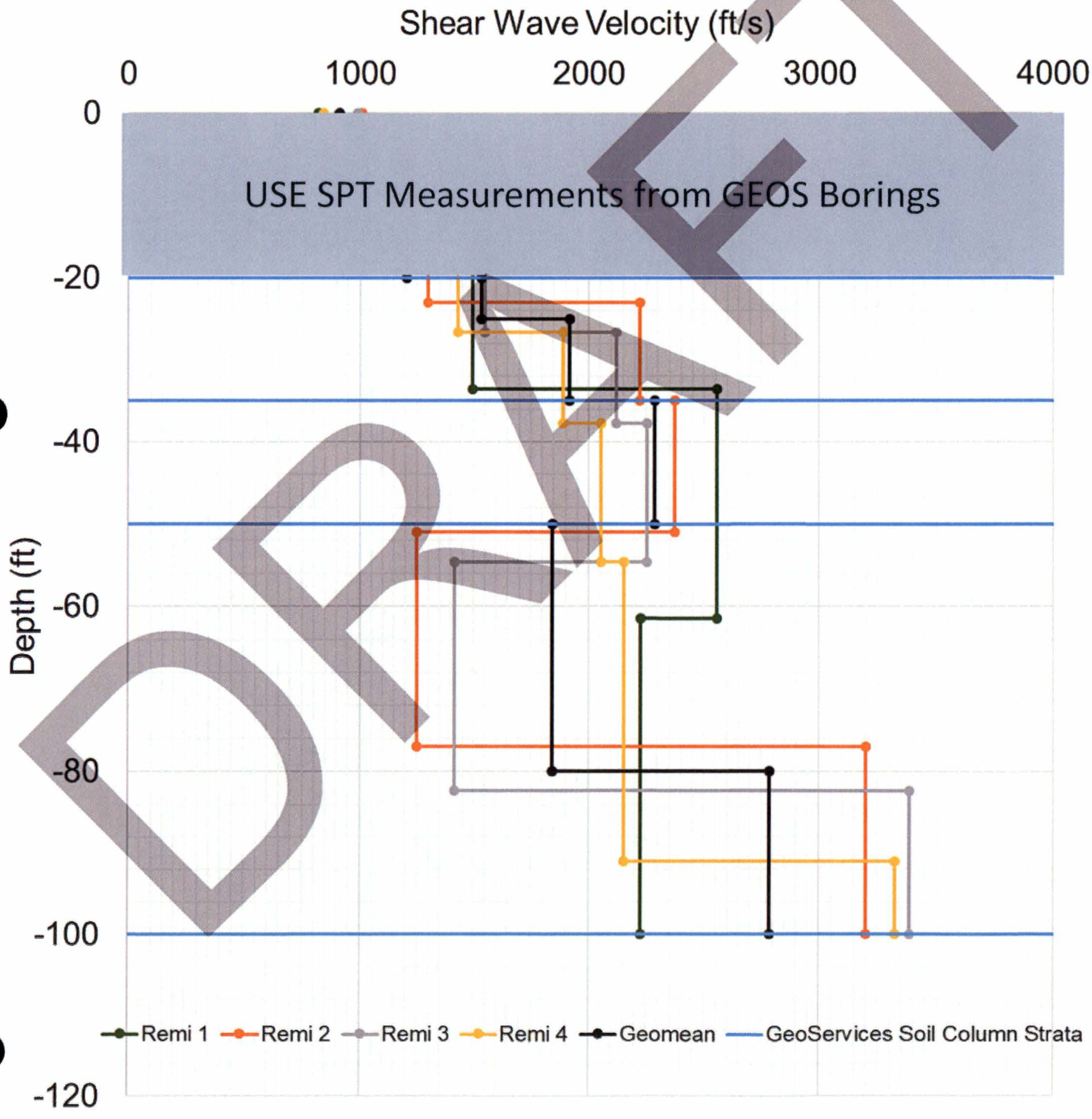
- Tan, C. K., Duncan, J. M., Rojiani, K. B., and Barker, R. M. (1991). Engineering Manual for Shallow Foundations, prepared for the National Cooperative Highway Research Program (NCHRP Project 24-4) in cooperation with Virginia Polytechnic Institute and State University. Sponsored by American Association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



CISF Site
Andrews, TX
Settlement Analysis
Methodology

Layers based on Average Shear Wave Velocity Measurements

- Interpreted from all explorations and collaboration between GEOS and DBA.
- Top 100ft contains 7 stratum in 5 materials identified by GEOS



CISF Site
Andrews, TX
Settlement Analysis
Methodology

DAN BROWN AND ASSOCIATES

Soil Column Based on Average Shear Wave Velocity Measurements

- Interpreted from all explorations and collaboration between GEOS and DBA.
- Top 100ft contains 7 stratum in 5 materials identified by GEOS
- 100ft-600ft (approx. location of incompressible layers with sharp contrast of velocity) contains 3 average velocity values for 6 layers identified in the soil column provided by GEOS

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Layer Material (From GEOS column)	Model Layer Name
20	25	1530	Caliche - Very Hard	Caliche Hard 1
25	35	1900	Caliche - Very Hard	Caliche Hard 2
35	50	2290	Ogallala - Sand with Gravel	Ogallala 1
50	80	1840	Ogallala - Sand with Gravel	Ogallala 2
80	100	2790	Ogallala - Sand with Gravel	Ogallala 3
100	130	2300	Dockum - Claystone and Siltstone	Dockum Claystone/Siltstone
130	230	2755	Claystone and Siltstone	Claystone and Siltstone
230	275	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone1
275	300	2755	Dockum - Silty Sands	Dockum Silty/Sands
300	360	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone 2
360	600	3115	Dockum - Clay/Claystone	Dockum Clay/Claystone 3

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Unit weight (pcf)	Poisson's Ratio	Gmax (psf)
20	25	1530	125	0.33	9,087,345
25	35	1900	125	0.33	14,013,975
35	50	2290	125	0.33	20,357,531
50	80	1840	130	0.33	13,668,571
80	100	2790	130	0.33	31,426,491
100	130	2300	130	0.33	21,357,143
130	230	2755	130	0.33	30,642,958
230	275	2755	130	0.33	30,642,958
275	300	2755	130	0.33	30,642,958
300	360	2755	130	0.33	30,642,958
360	600	3115	130	0.33	39,174,511

CISF Site
Andrews, TX
Settlement Analysis
Methodology

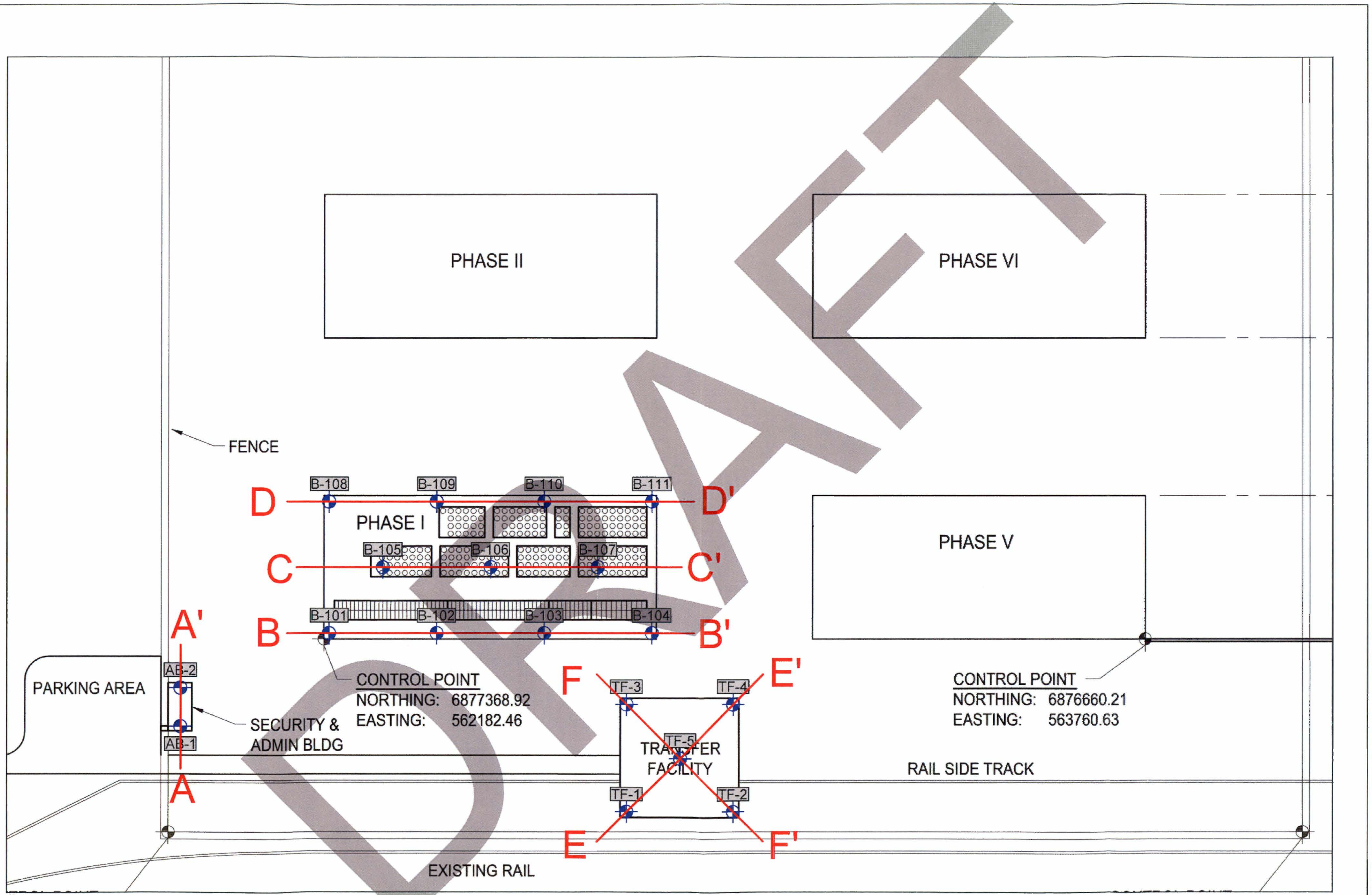
DAN BROWN AND ASSOCIATES

Total Constrained Modulus Profile

- 0 to 20 ft depth
  - Constrained modulus obtained from GEOS SPT data and correlating N-Value with constrained modulus using Tan, C. K., Duncan, J. M., Rojiani, K. B., and Barker, R. M. (1991).
- 20 to 600 ft depth
  - Constrained modulus obtained from converting the GEOS shear wave velocity to constrained modulus using the unit weight and Poisson's ratio

Top (ft)	Bottom (ft)	Layer Material (From GEOS column)	Constrained Modulus (ksf)
0	2	Cover Sands	890
2	10	Caliche with Sand Matrix - Moderately Hard	1,200
10	20	Caliche with Sand Matrix - Moderately Hard	1,200
20	25	Caliche - Very Hard	35,815
25	35	Caliche - Very Hard	55,232
35	50	Ogallala - Sand with Gravel	80,233
50	80	Ogallala - Sand with Gravel	53,870
80	100	Ogallala - Sand with Gravel	123,857
100	130	Dockum - Claystone and Siltstone	84,172
130	230	Claystone and Siltstone	120,769
230	275	Dockum - Clay/Claystone	120,769
275	300	Dockum - Silty Sands	120,769
300	360	Dockum - Clay/Claystone	120,769
360	600	Dockum - Clay/Claystone	154,394

GEOS Blow Count Profiles



Notes:

1) Site Source Provided by: WCS, (06/12/2015)
2) Boring Locations are shown in general arrangement only
3) Do Not use Boring Locations for determinations of Distance or Quantities

Boring Location & Identifier

Cross Section line (Start - End')

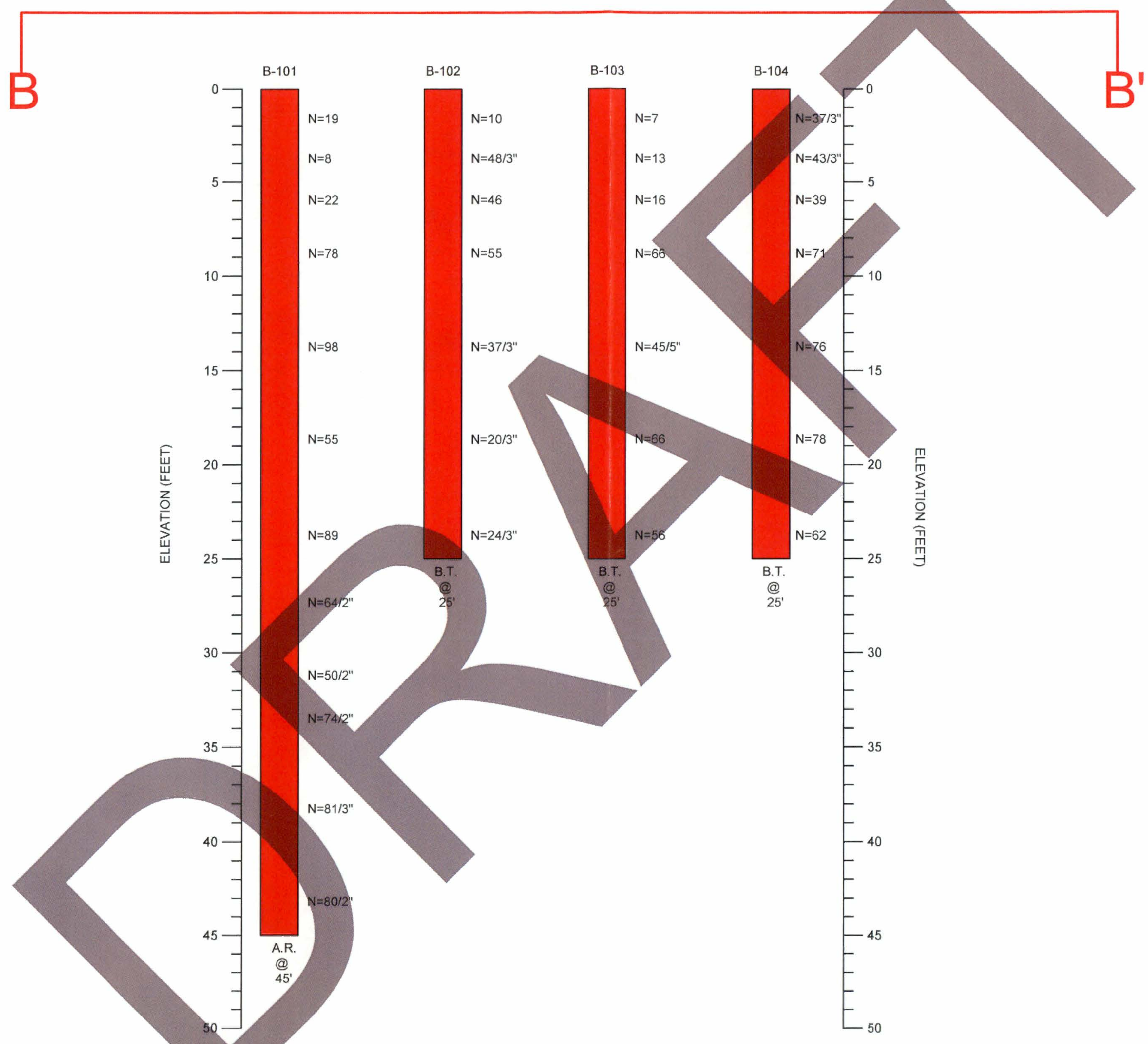
GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Cross Section-Location Plan
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 5



Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

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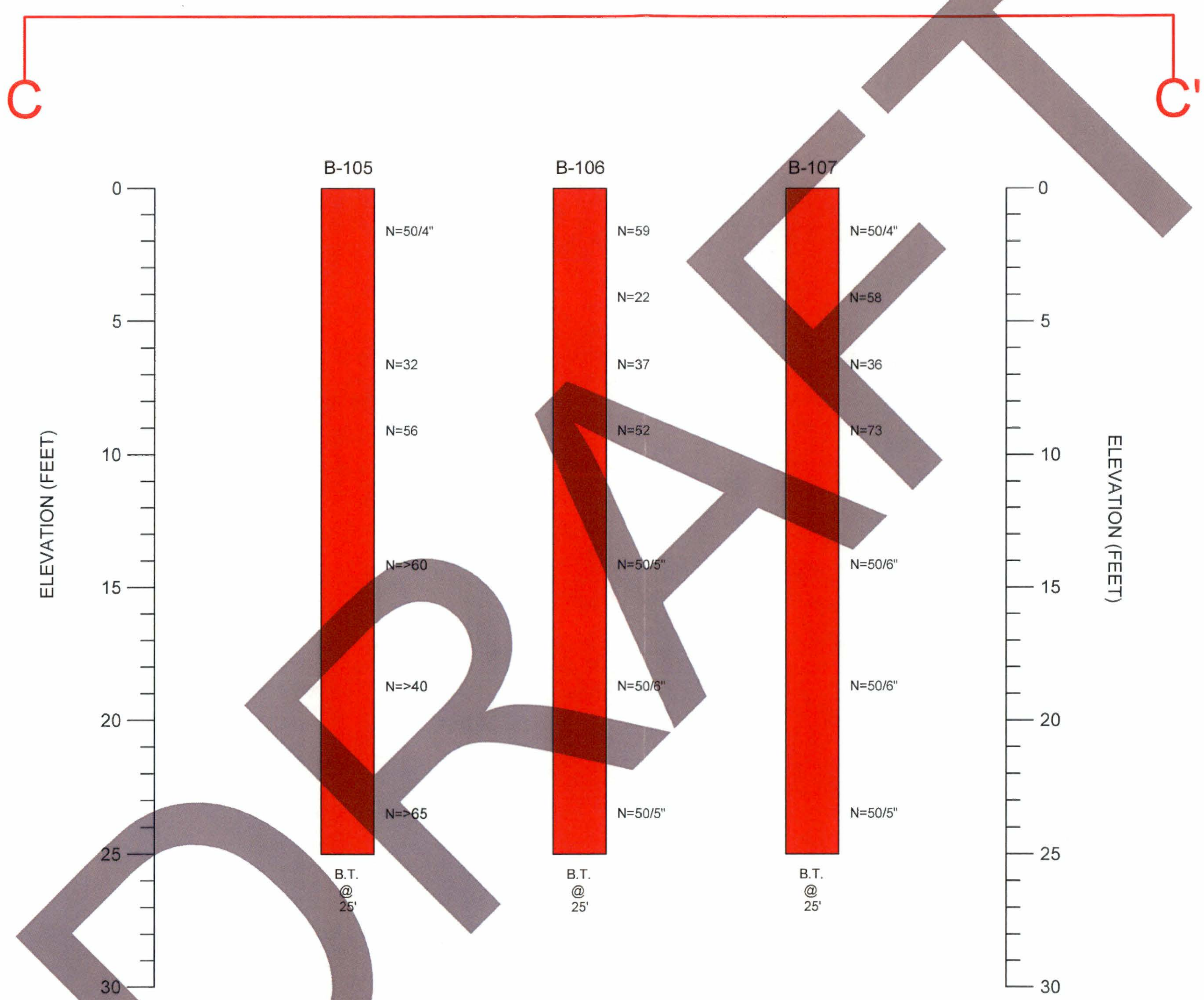
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 7



Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

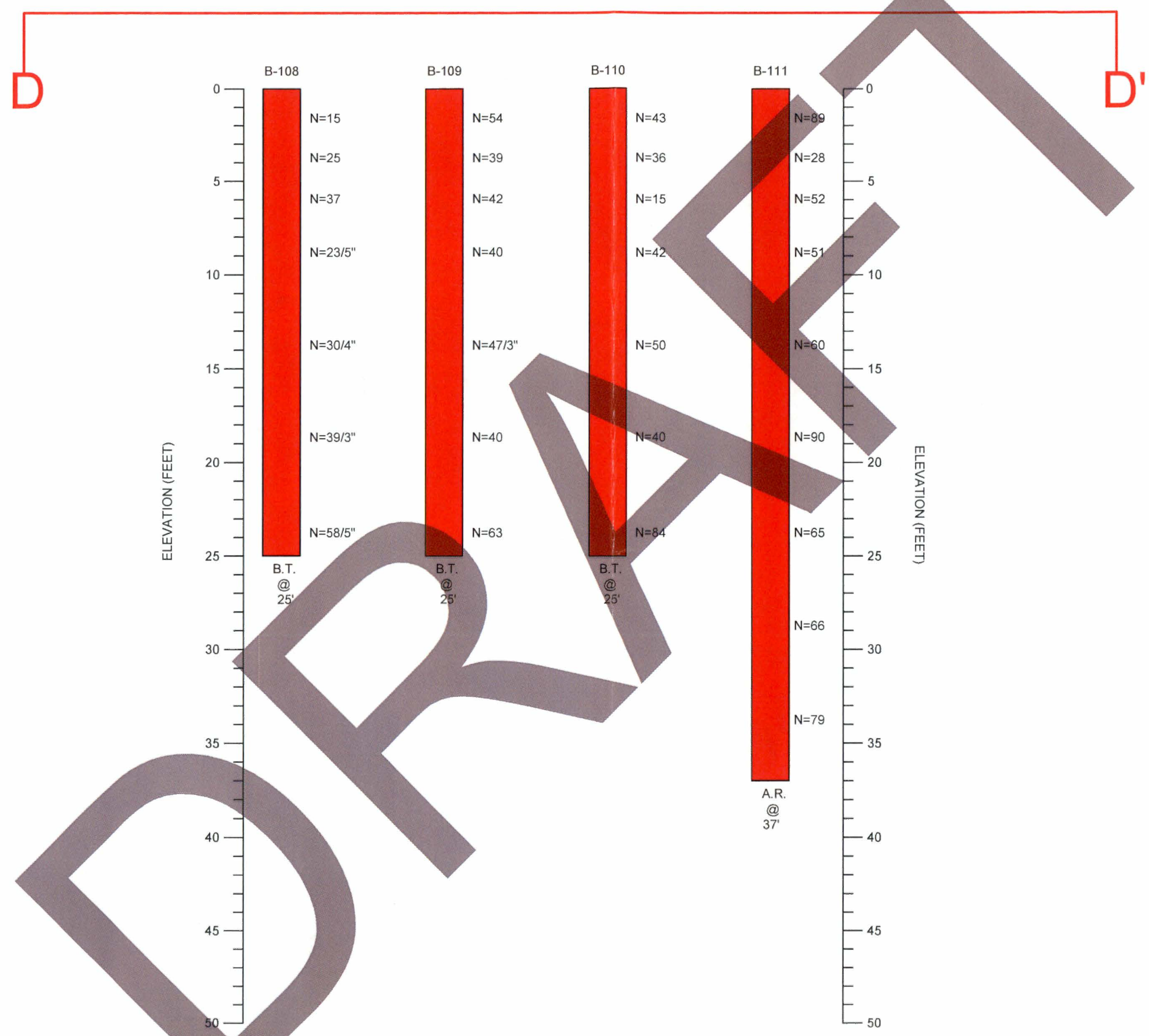
GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE: 08/10/2015

GEOS Project No. 31-151247

FIGURE: 8



Notes:

1) B.T. = Boring Termination
2) A.R. = Auger Refusal
3) Not To Scale

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers

Boring Cross-Section
WCS Consolidated Interim Storage Facility
WCS
Andrews, Texas

DATE:
08/10/2015

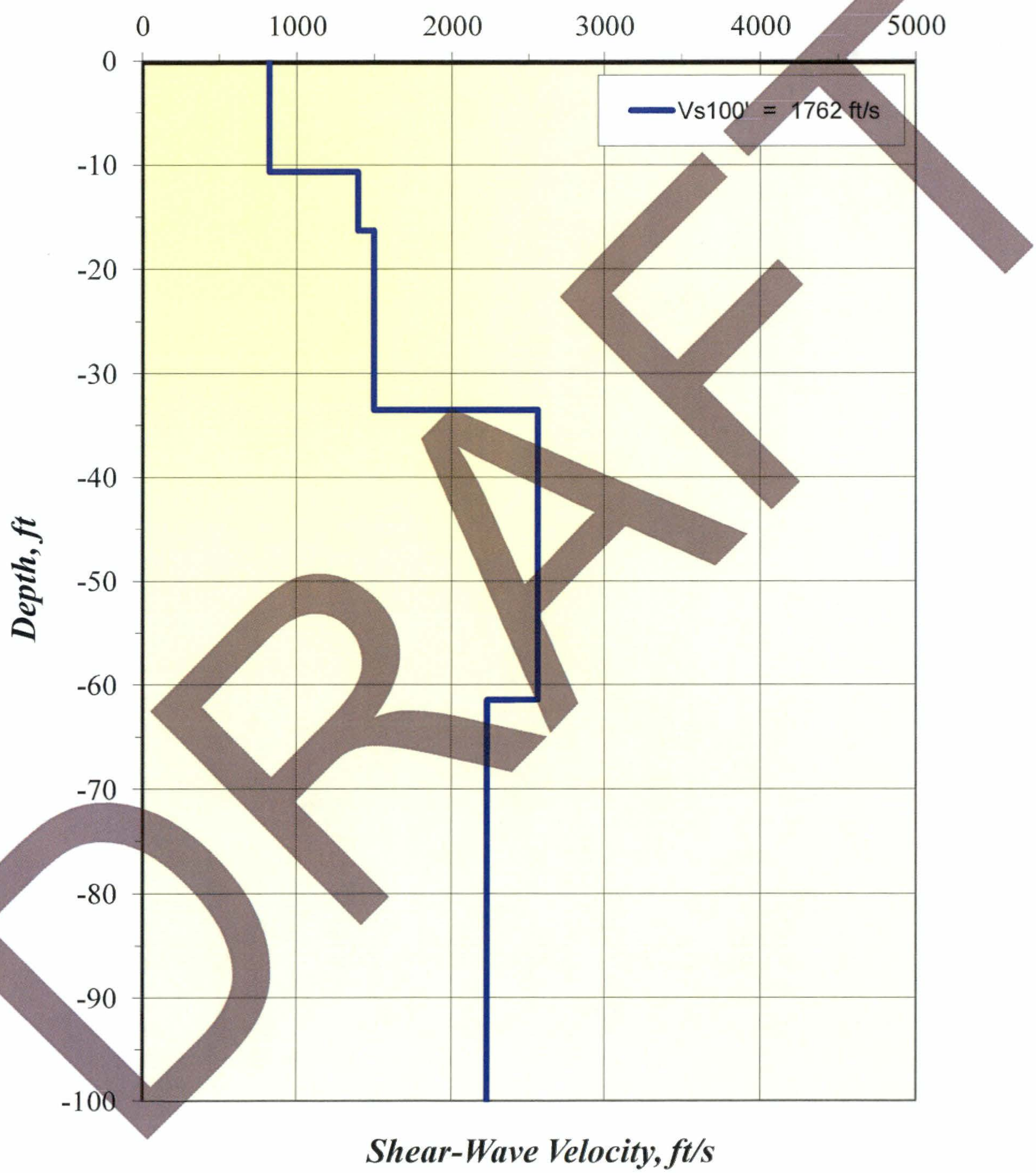
GEOS Project No.
31-151247

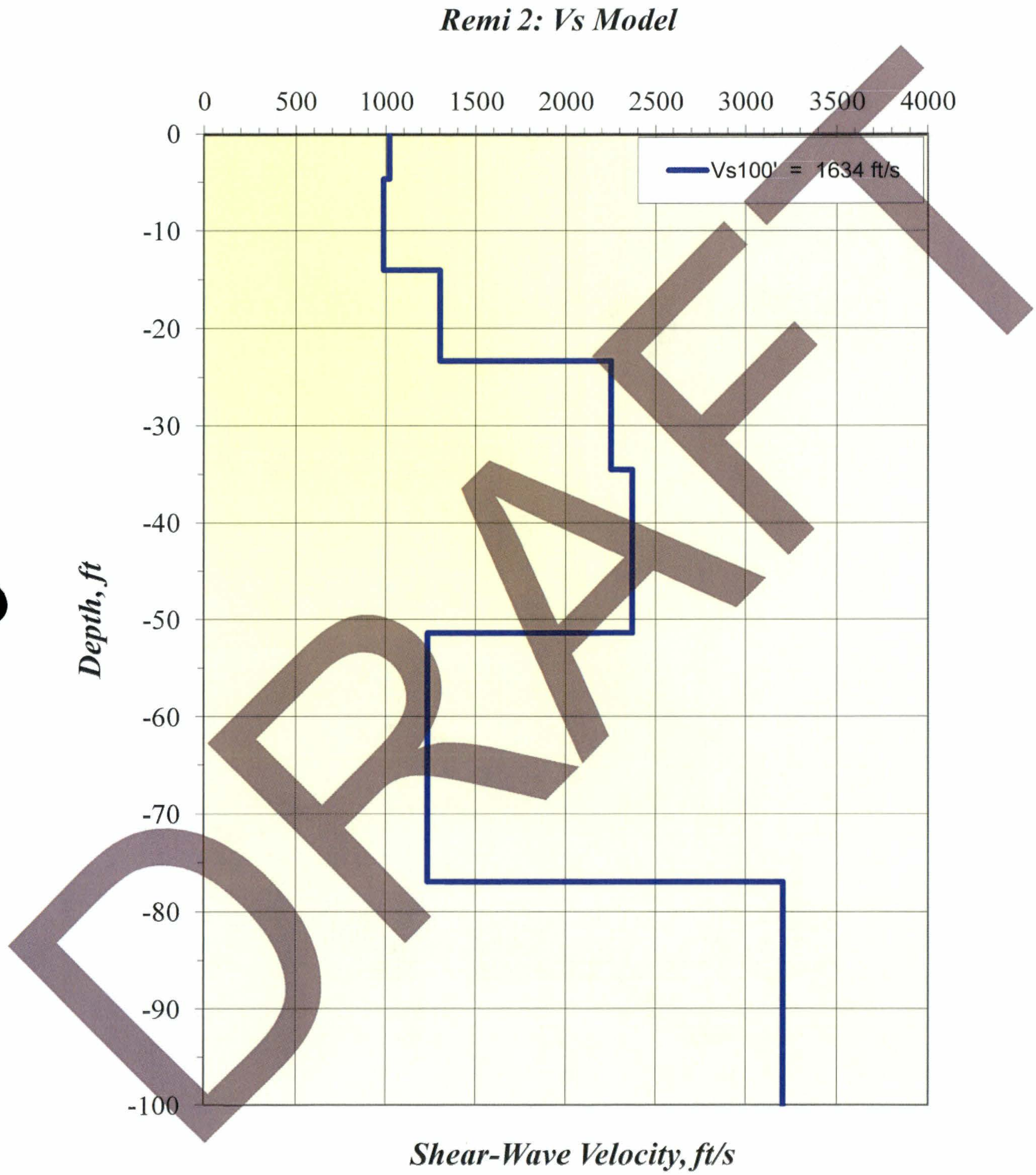
FIGURE:
9

DAN BROWN AND ASSOCIATES

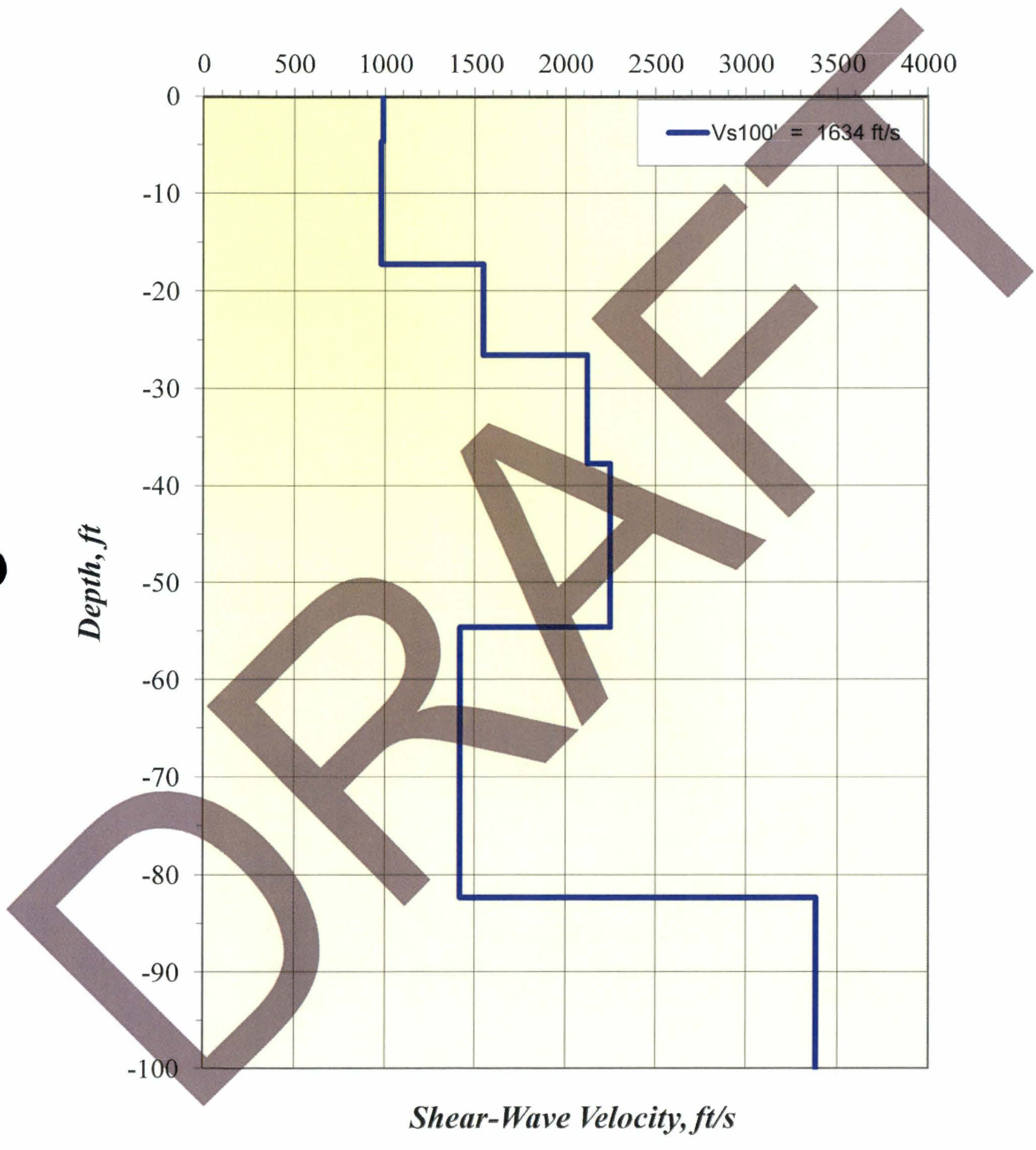
Shear Wave Velocity Profiles

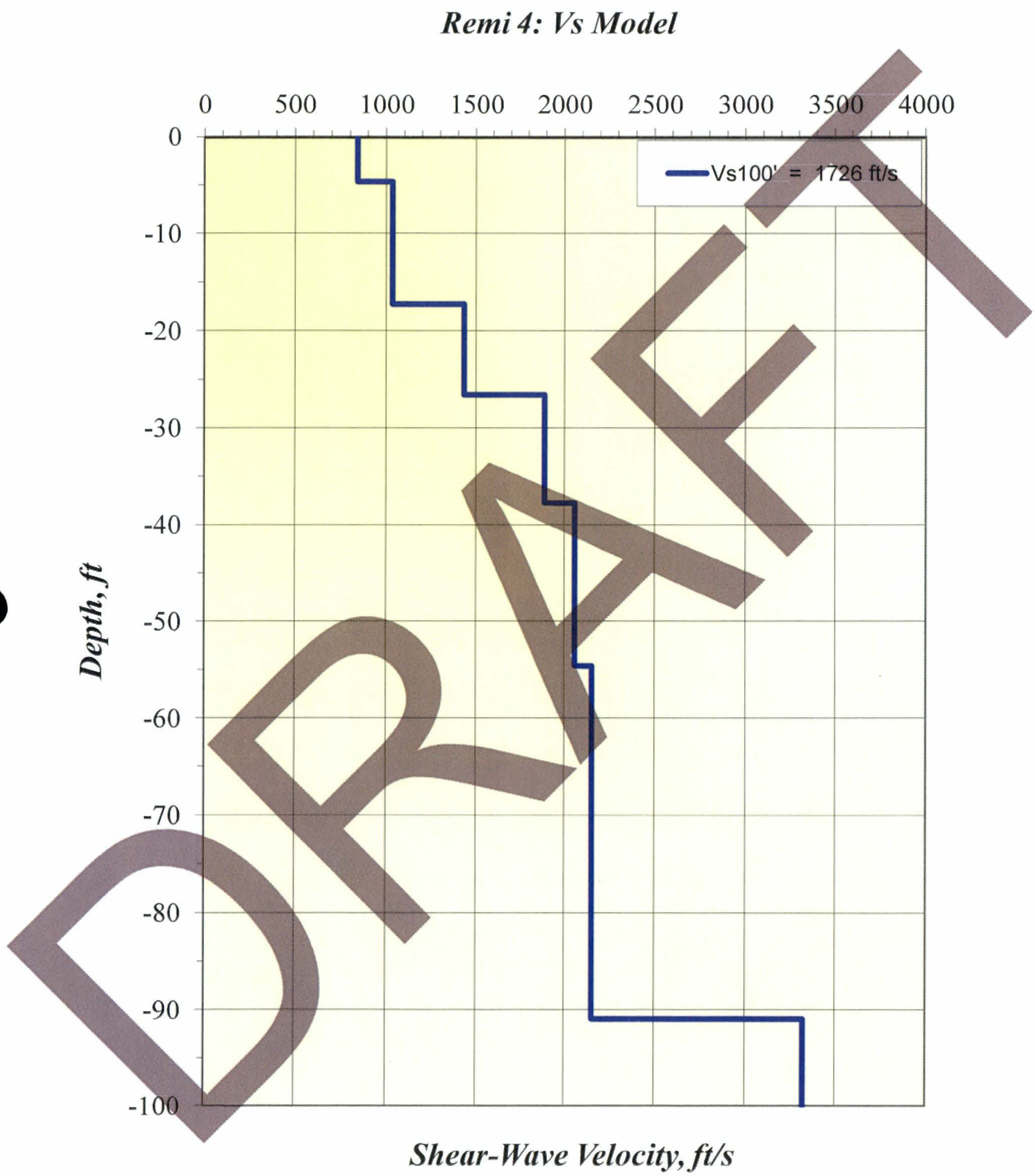
Remi 1: Vs Model




Remi 2: Vs Model
0
500
1000
1500
2000
2500
3000
3500
4000
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Vs100' = 1634 ft/s
Depth, ft
Shear-Wave Velocity, ft/s
DRAFT

Remi 3: Vs Model

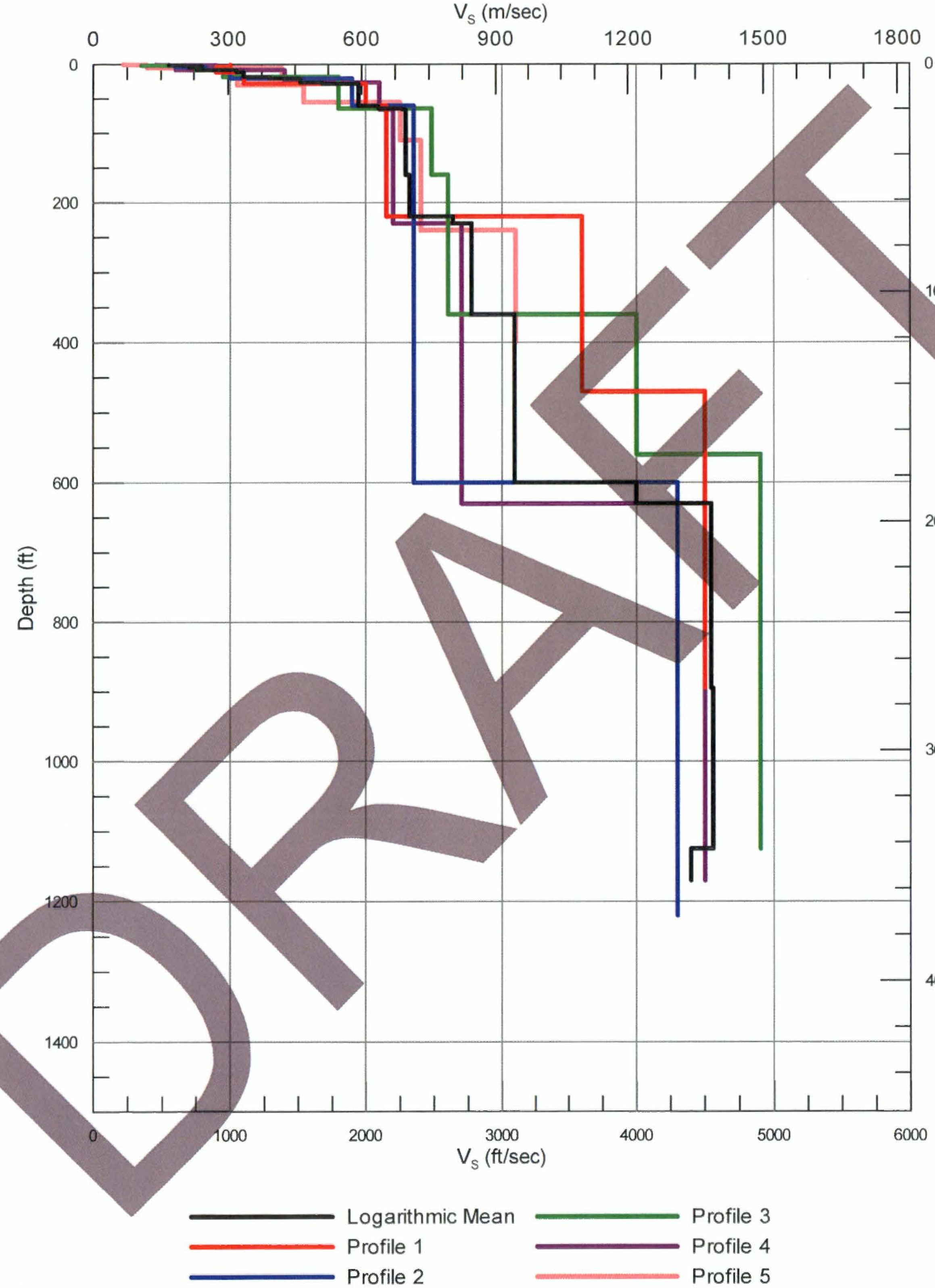




Remi 4: Vs Model
0
500
1000
1500
2000
2500
3000
3500
4000
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Vs100' = 1726 ft/s
Depth, ft
Shear-Wave Velocity, ft/s
DRAFT

DAN BROWN AND ASSOCIATES

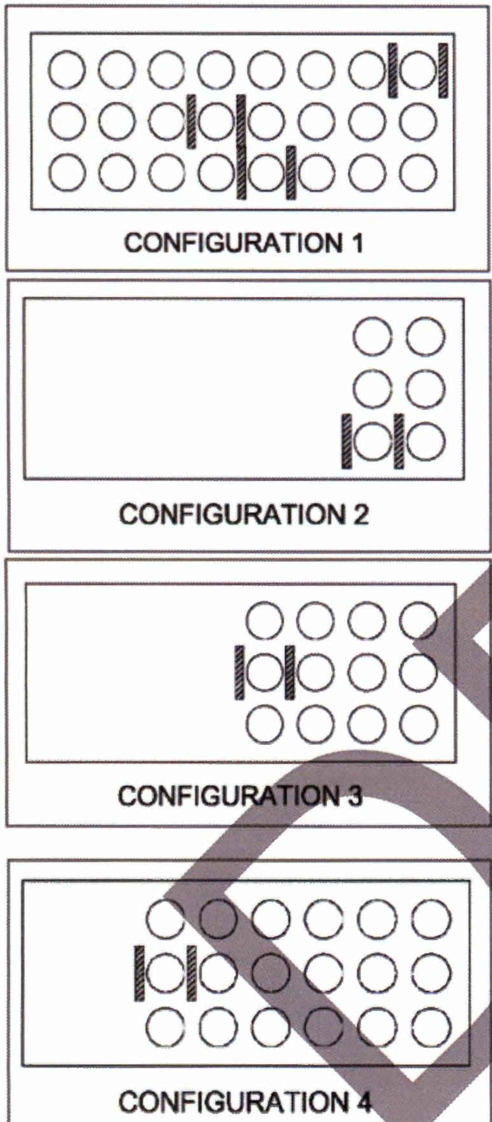
AECOM Shear Wave Velocity Profile

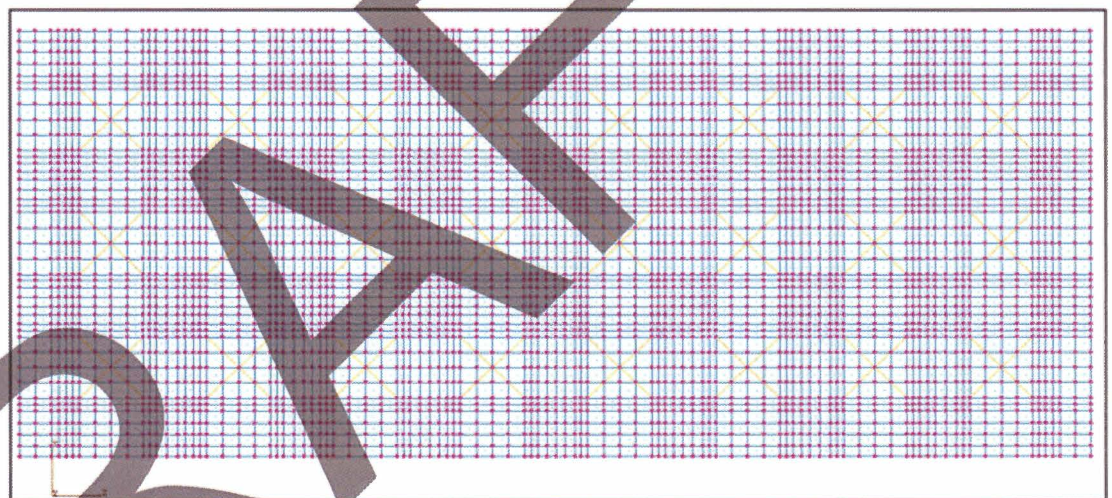


AECOM	Project No. 31787-001 Waste Control Specialists Andrews, Texas	V_S PROFILES FROM 2015 SASW SURVEYS	Figure 20

Enercon Model Configurations

CISF Site
Single Pad Analysis
Andrews, TX





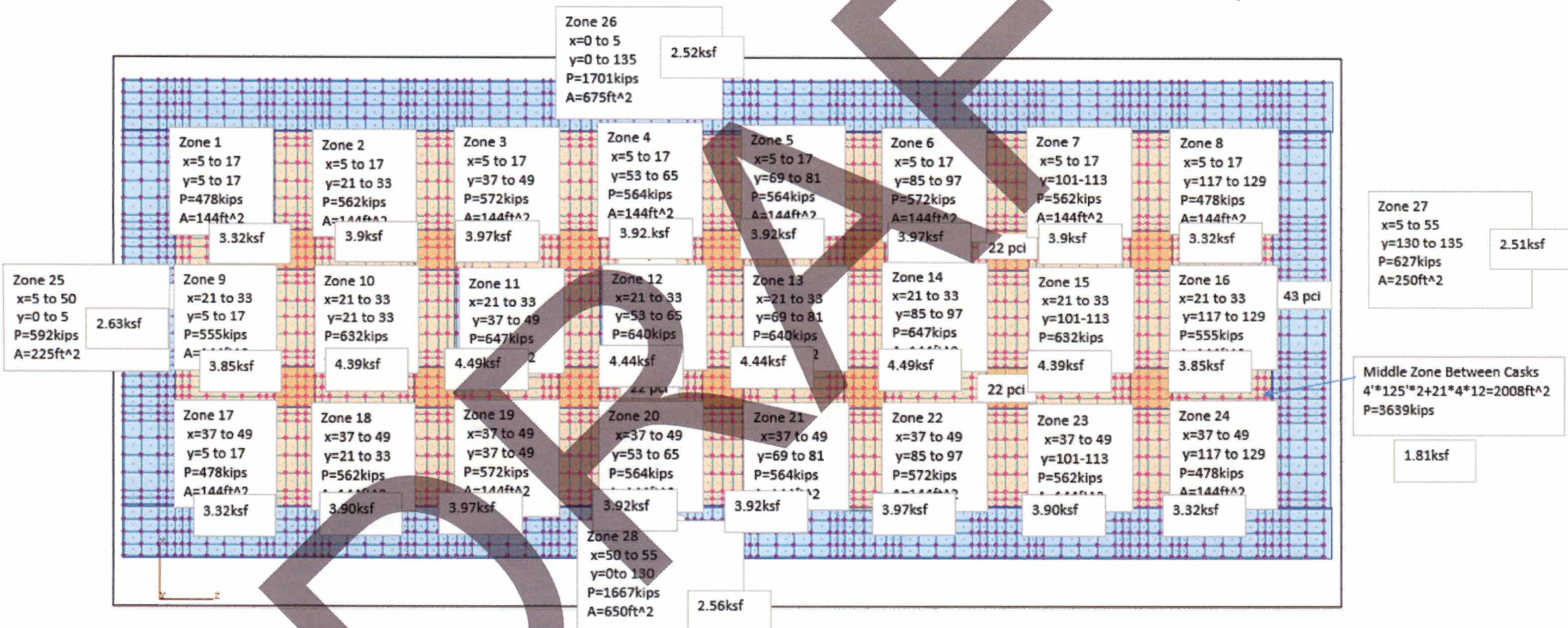
ISFSI Pad General Layout

DAN BROWN AND ASSOCIATES

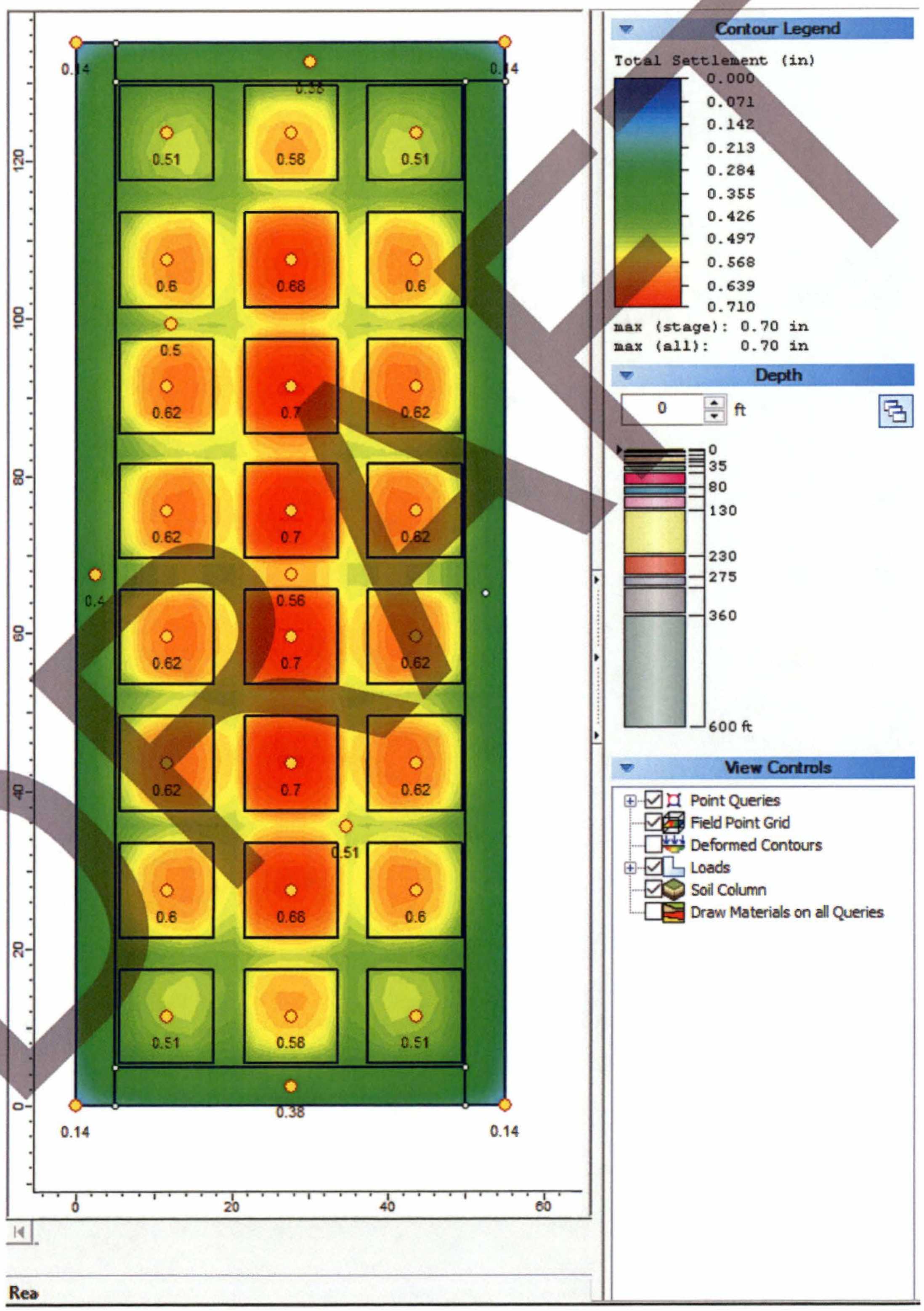
Configuration 1
Final Bearing Pressure Zones,
Settlement Results,
and Modulus of Subgrade Reaction
Values for Enercon Model

CISF Site
Single Pad Analysis
Andrews, TX

Configuration 1-Bearing Pressures (INPUT 5 Spreadsheet Loads)

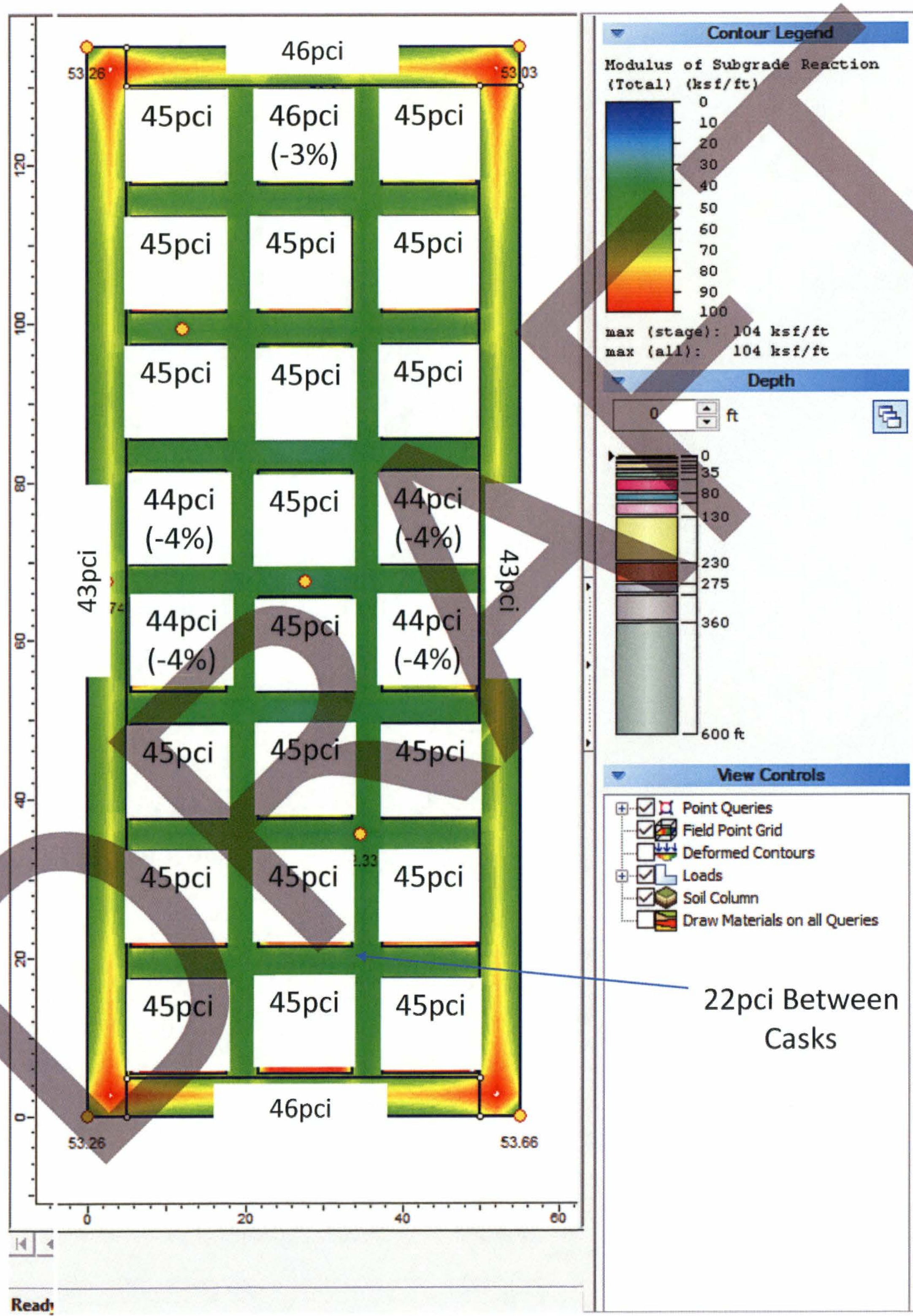


Configuration 1-Settlement Estimate (INPUT 5 Spreadsheet Loads)



CISF Site
Andrews, TX
Single Pad Analysis Results

- Configuration 1-k values (INPUT 5 Spreadsheet)



Configuration 1 Settle3 Inputs

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF Storage Pad Config 1 200214 INPUT 5.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.701508
Loading Stress ZZ [ksf]	0.034724	4.49

Loads

1. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft²
Load	1.81 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.71 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	135
0	135

3. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	130
55	135
5	135
5	130

4. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	130
50	130

5. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

6. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

7. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

8. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	117.5
49.5	117.5
49.5	129.5
37.5	129.5

9. Rectangular Load: "Zone 8"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	117.5
17.5	117.5
17.5	129.5
5.5	129.5

10. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	101.5
49.5	101.5
49.5	113.5
37.5	113.5

11. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	85.5
49.5	85.5
49.5	97.5
37.5	97.5

12. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	69.5
49.5	69.5
49.5	81.5
37.5	81.5

13. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	53.5
49.5	53.5
49.5	65.5
37.5	65.5

14. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	37.5
49.5	37.5
49.5	49.5
37.5	49.5

15. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

16. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	117.5
33.5	117.5
33.5	129.5
21.5	129.5

17. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	101.5
17.5	101.5
17.5	113.5
5.5	113.5

18. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	85.5
17.5	85.5
17.5	97.5
5.5	97.5

19. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	69.5
17.5	69.5
17.5	81.5
5.5	81.5

20. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	53.5
17.5	53.5
17.5	65.5
5.5	65.5

21. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	37.5
17.5	37.5
17.5	49.5
5.5	49.5

22. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

23. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

24. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.58 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

25. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	37.5
33.5	37.5
33.5	49.5
21.5	49.5

26. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	53.5
33.5	53.5
33.5	65.5
21.5	65.5

27. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	69.5
33.5	69.5
33.5	81.5
21.5	81.5

28. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	85.5
33.5	85.5
33.5	97.5
21.5	97.5

29. Rectangular Load: "Zone 15"

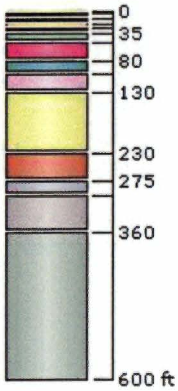
Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.57 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	101.5
33.5	101.5
33.5	113.5
21.5	113.5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Footing Bottom Left	0, 0	Auto: 101
26	Footing Bottom Right	55, 0	Auto: 101
27	Footing Top Left	0, 135	Auto: 101
28	Footing Top Right	55, 135	Auto: 101
29	Footing Center	27.5, 67.5	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Query Point 31	30, 132.5	Auto: 101
32	Query Point 32	52.5, 65	Auto: 101
33	Query Point 33	34.543, 35.669	Auto: 101
34	Query Point 34	27.5, 2.5	Auto: 101

Field Point Grid

Number of points 1672
Expansion Factor 1

Grid Coordinates

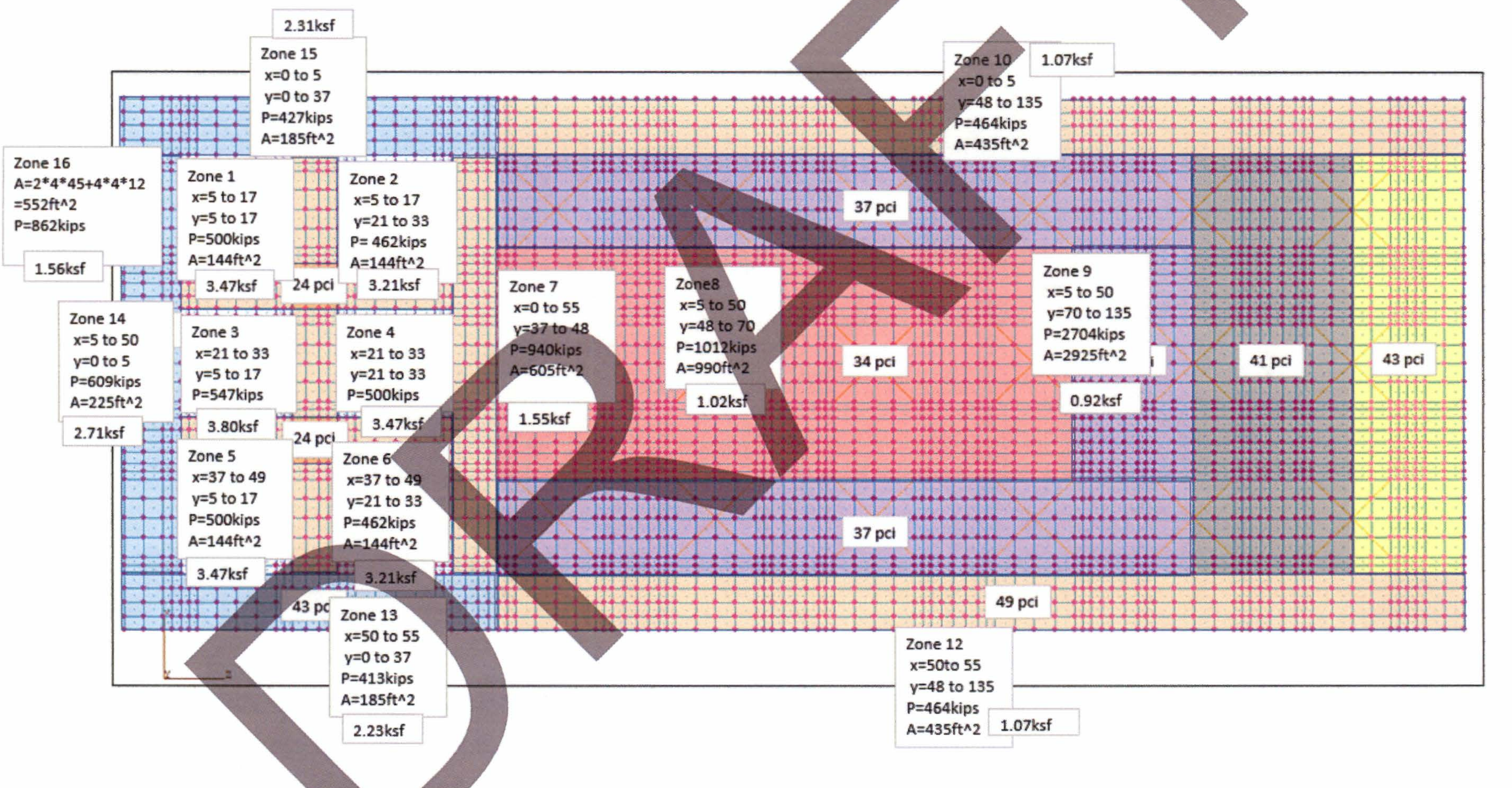
X [ft]	Y [ft]
122.5	202.5
122.5	-67.5
-67.5	-67.5
-67.5	202.5

Configuration 2
Final Bearing Pressure Zones,
Settlement Results,
and Modulus of Subgrade Reaction
Values for Enercon Model

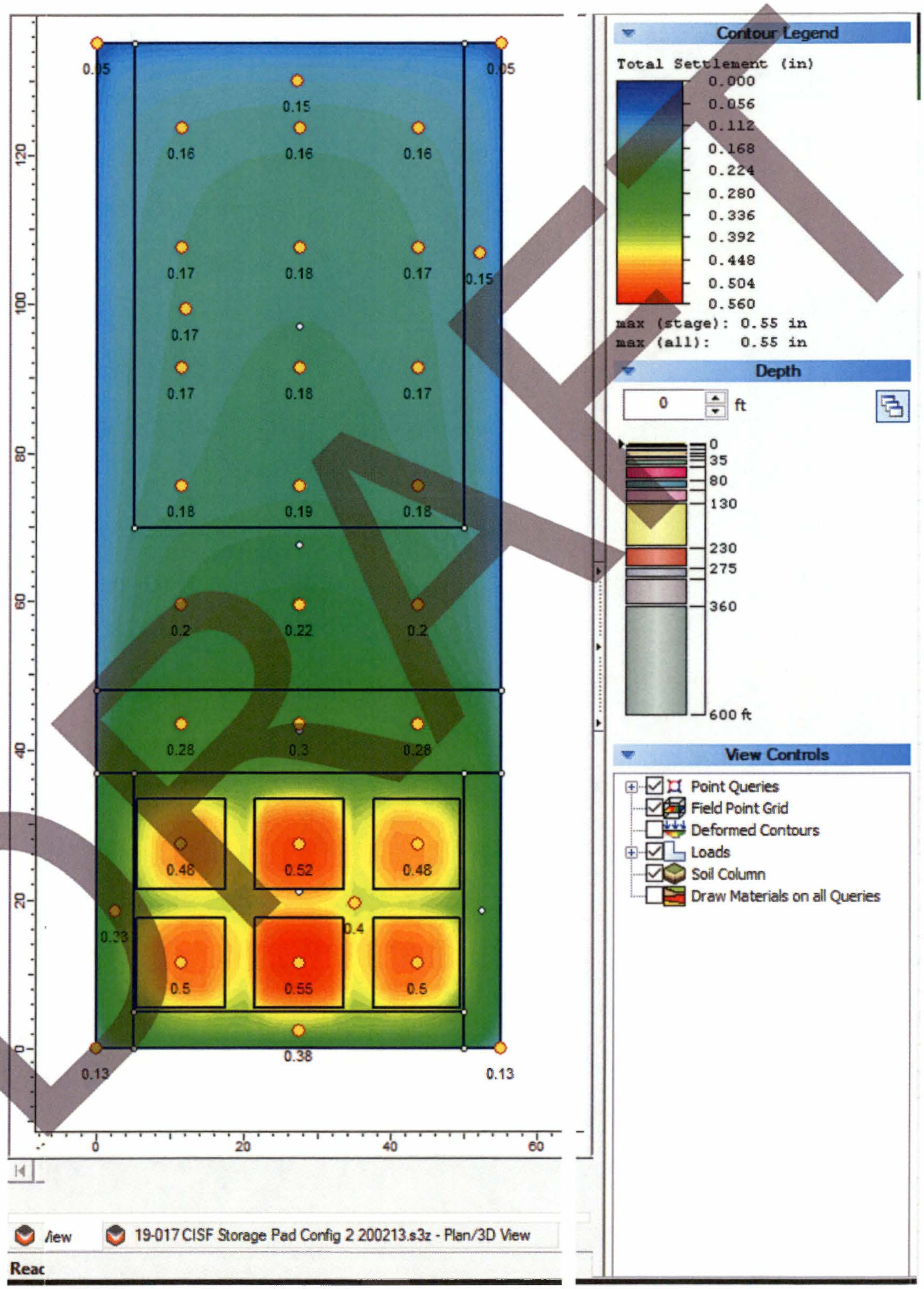
DAN BROWN AND ASSOCIATES

CISF Site
Single Pad Analysis
Andrews, TX

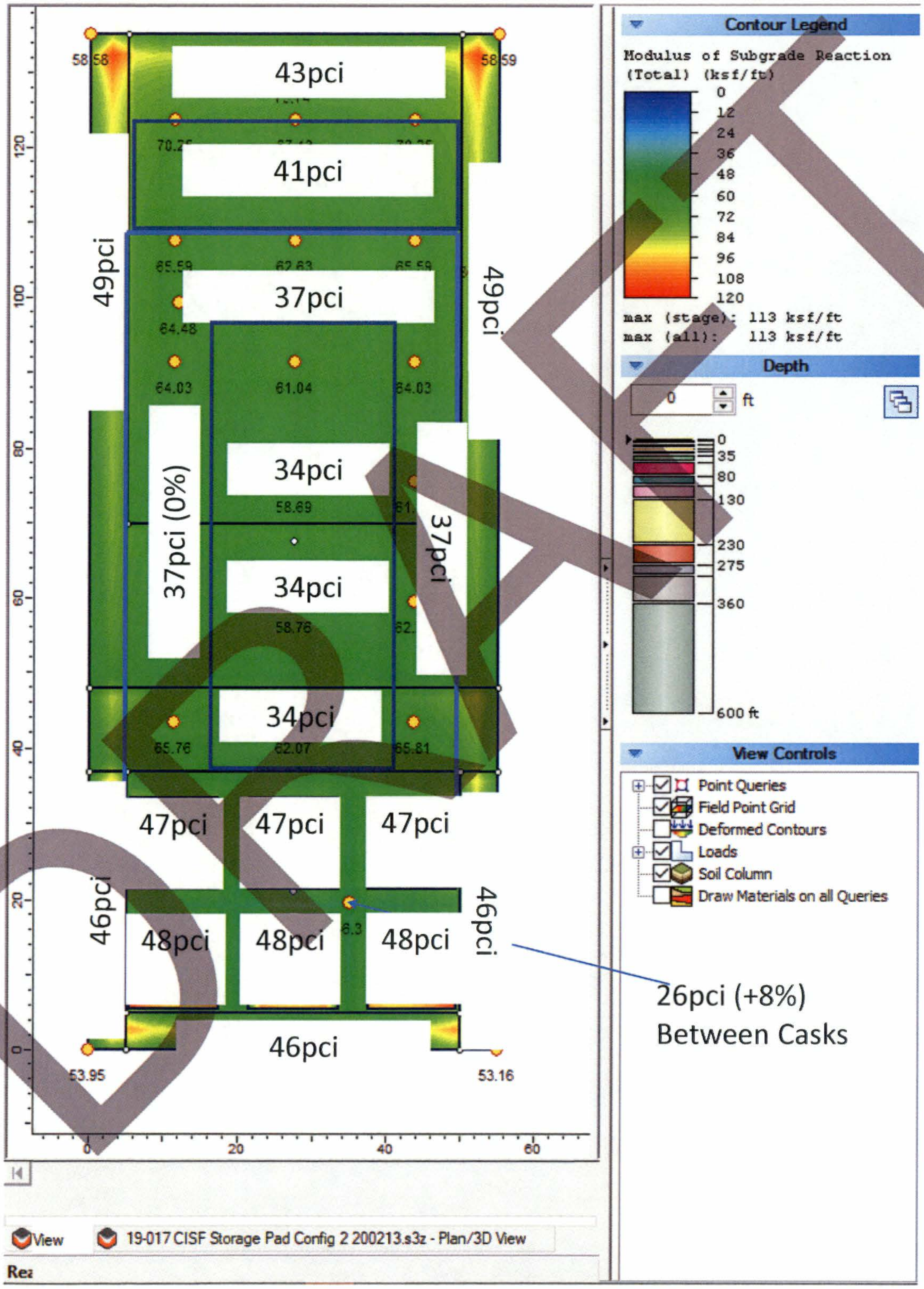
Configuration 2-Bearing Pressures (INPUT 5 Spreadsheet Loads)



Configuration 2-Settlement Estimate(INPUT 5 Spreadsheet Loads)



- Configuration 2-k values (INPUT 5 Spreadsheet)



Configuration 2 Settle3 Inputs

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF Storage Pad Config 2 200214.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.550882
Loading Stress ZZ [ksf]	0.0164762	3.8

Loads

1. Polygonal Load: "Base Zone"

Label	Base Zone
Load Type	Flexible
Area of Load	7425 ft^2
Load	0.92 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Polygonal Load: "Zone 8, 10, and 12"

Label	Zone 8, 10, and 12
Load Type	Flexible
Area of Load	1860 ft^2
Load	0.14 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	48
55	135
50	135
50	70
5	70
5	135
0	135
0	48

3. Polygonal Load: "Zone 7"

Label	Zone 7
Load Type	Flexible
Area of Load	605 ft^2
Load	0.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	37
55	48
0	48
0	37

4. Polygonal Load: "Zone 15"

Label	Zone 15
Load Type	Flexible
Area of Load	185 ft^2
Load	1.38 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	37
0	37

5. Polygonal Load: "Zone 13"

Label	Zone 13
Load Type	Flexible
Area of Load	185 ft^2
Load	1.31 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	37
50	37

6. Polygonal Load: "Zone 14"

Label	Zone 14
Load Type	Flexible
Area of Load	225 ft^2
Load	1.78 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

7. Polygonal Load: "Zone 16"

Label	Zone 16
Load Type	Flexible
Area of Load	1440 ft^2
Load	0.64 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	5
50	5
50	37
5	37

8. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.91 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

9. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.65 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

10. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.91 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

11. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.24 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

12. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.91 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

13. Rectangular Load: "Zone 6"

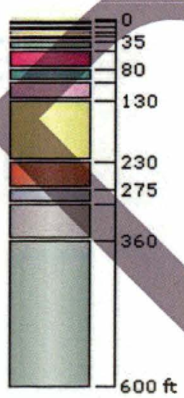
Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.65 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Footing Bottom Left	0, 0	Auto: 101
26	Footing Bottom Right	55, 0	Auto: 101
27	Footing Top Left	0, 135	Auto: 101
28	Footing Top Right	55, 135	Auto: 101
29	Footing Center	52.208, 106.8	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Query Point 31	34.932, 19.529	Auto: 101
32	Query Point 32	2.5, 18.5	Auto: 101
33	Query Point 33	27.105, 129.904	Auto: 101
34	Query Point 34	27.5, 2.5	Auto: 101

Field Point Grid

Number of points 1294
Expansion Factor 1

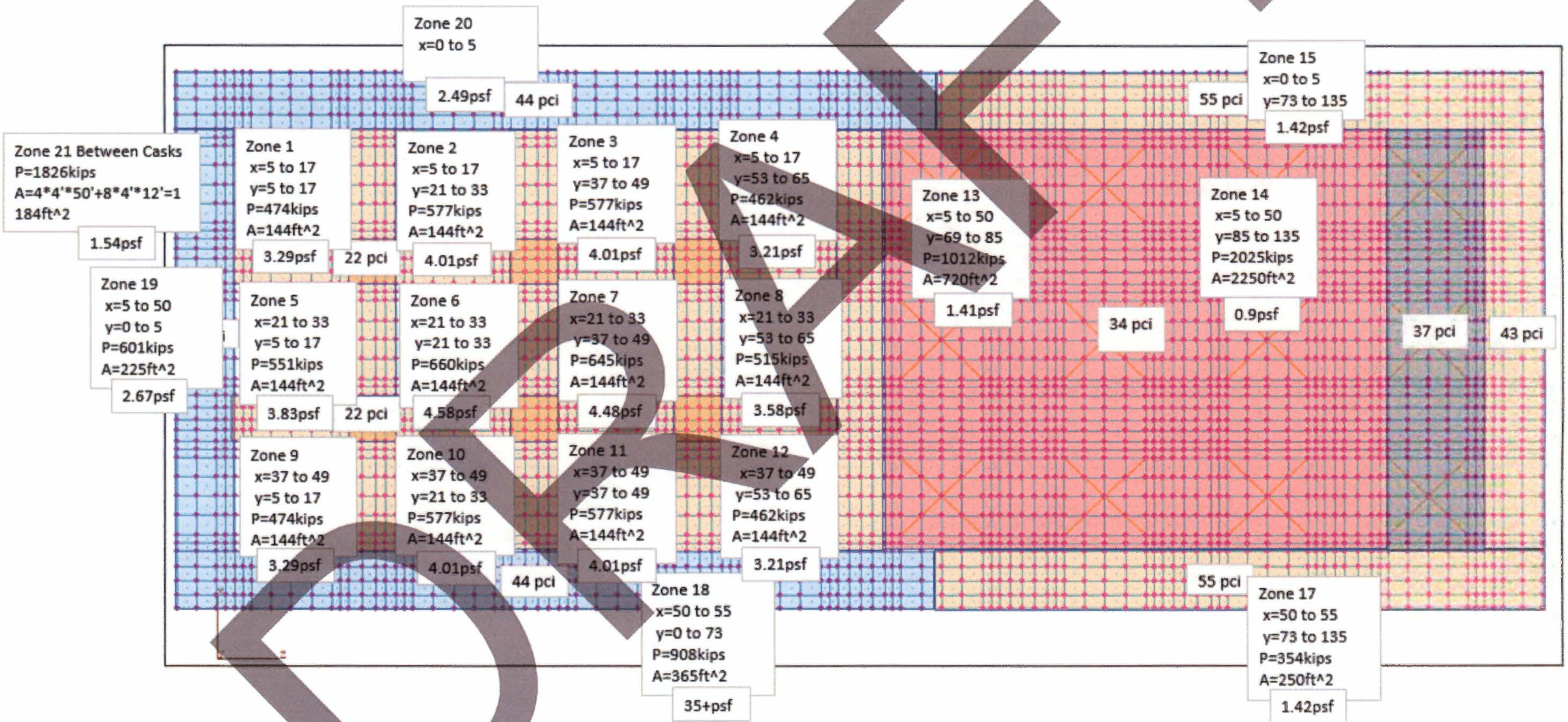
Grid Coordinates

X [ft]	Y [ft]
122.5	202.5
122.5	-67.5
-67.5	-67.5
-67.5	202.5

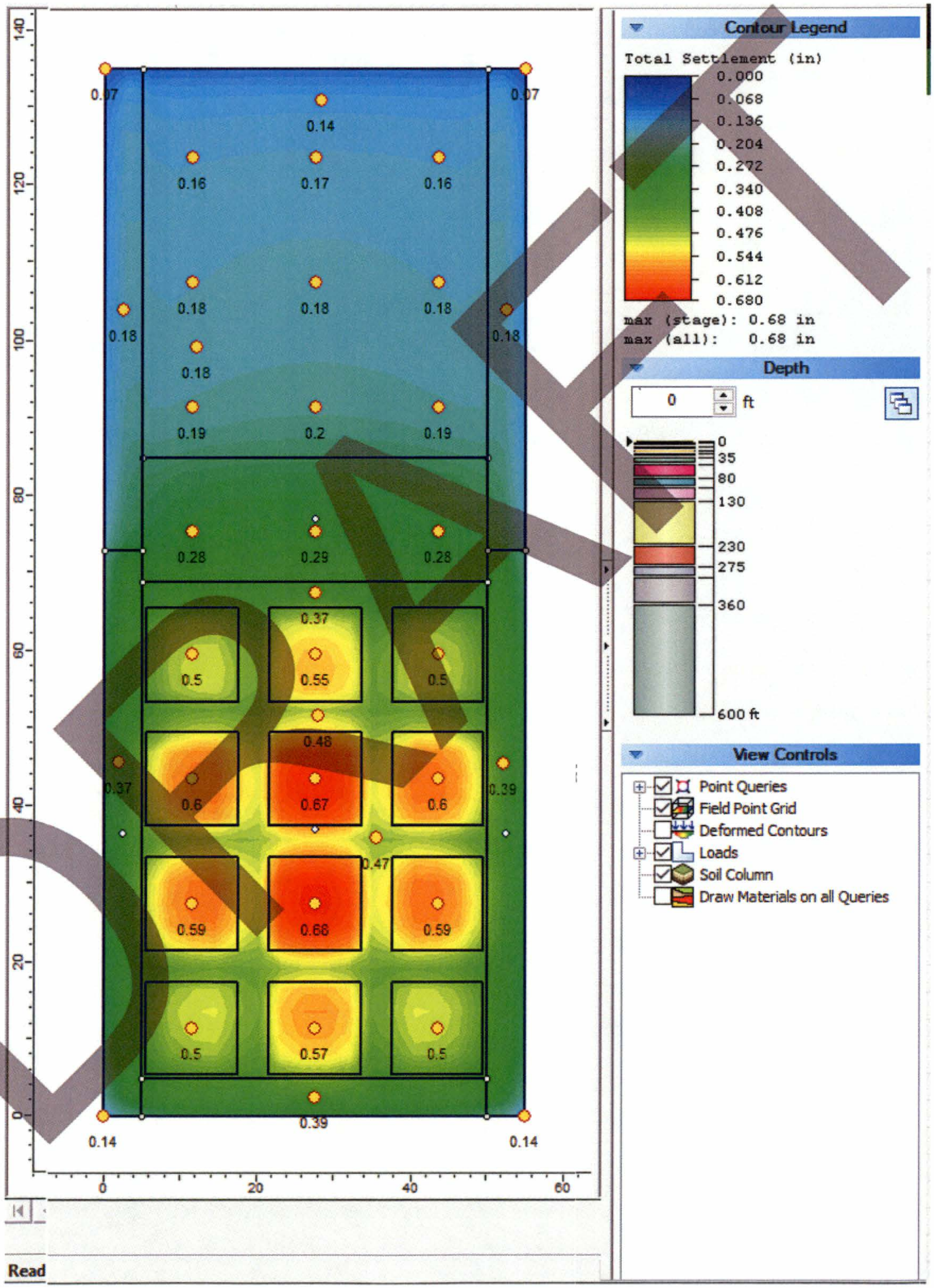
Configuration 3
Final Bearing Pressure Zones,
Settlement Results,
and Modulus of Subgrade Reaction
Values for Enercon Model

CISF Site
Single Pad Analysis
Andrews, TX

Configuration 3-Bearing Pressures (INPUT 5 Spreadsheet Loads)

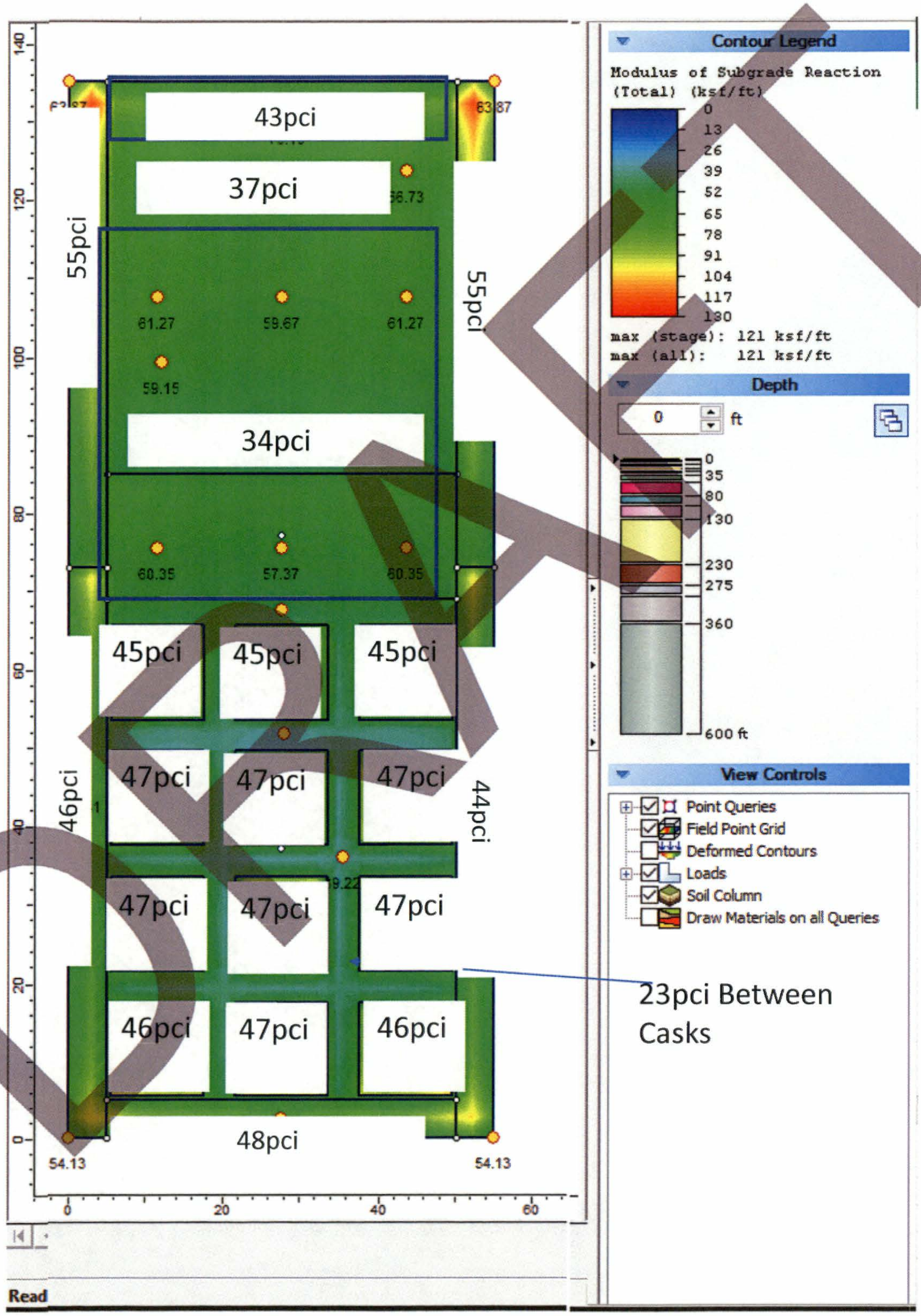


Configuration 3-Settlement Estimate (INPUT 5 Spreadsheet Loads)



CISF Site
Andrews, TX
Single Pad Analysis Results

- Configuration 3 k values (INPUT 5 Spreadsheet)



Configuration 3 Settle3 Inputs

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF Storage Pad Config 3 200214 Input 5.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.677157
Loading Stress ZZ [ksf]	0.02225	4.58

Loads

1. Polygonal Load: "Base Zone"

Label	Base Zone
Load Type	Flexible
Area of Load	7425 ft^2
Load	0.9 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

3. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

4. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.67 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	53.5
49.5	53.5
49.5	65.5
37.5	65.5

5. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.46 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	37.5
49.5	37.5
49.5	49.5
37.5	49.5

6. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.46 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

7. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.67 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	53.5
17.5	53.5
17.5	65.5
5.5	65.5

8. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.46 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	37.5
17.5	37.5
17.5	49.5
5.5	49.5

9. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.46 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

10. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.28 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

11. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	3.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

12. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.94 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	37.5
33.5	37.5
33.5	49.5
21.5	49.5

13. Rectangular Load: "Zone 8"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.03 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	53.5
33.5	53.5
33.5	65.5
21.5	65.5

14. Polygonal Load: "Zone 13"

Label	Zone 13
Load Type	Flexible
Area of Load	720 ft^2
Load	0.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	69
50	85
5	85
5	69

15. Polygonal Load: "Zone 20"

Label	Zone 20
Load Type	Flexible
Area of Load	365 ft^2
Load	1.59 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	73
0	73

16. Polygonal Load: "Zone 18"

Label	Zone 18
Load Type	Flexible
Area of Load	365 ft^2
Load	1.59 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	73
50	73

17. Polygonal Load: "Zone 19"

Label	Zone 19
Load Type	Flexible
Area of Load	225 ft^2
Load	1.77 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

18. Polygonal Load: "Zone 17"

Label	Zone 17
Load Type	Flexible
Area of Load	310 ft^2
Load	0.52 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	73
55	73
55	135
50	135

19. Polygonal Load: "Zone 15"

Label	Zone 15
Load Type	Flexible
Area of Load	310 ft^2
Load	0.52 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	73
5	73
5	135
0	135

20. Polygonal Load: "Zone 21"

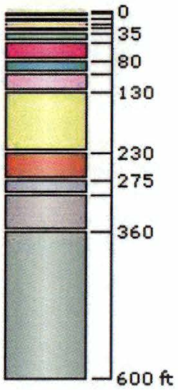
Label	Zone 21
Load Type	Flexible
Area of Load	2880 ft^2
Load	0.64 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	5
50	5
50	69
5	69

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Footing Bottom Left	0, 0	Auto: 101
26	Footing Bottom Right	55, 0	Auto: 101
27	Footing Top Left	0, 135	Auto: 101
28	Footing Top Right	55, 135	Auto: 101
29	Footing Center	27.5, 67.5	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Query Point 31	28.128, 130.851	Auto: 101
32	Query Point 32	2.012, 45.679	Auto: 101
33	Query Point 33	52.141, 45.546	Auto: 101
34	Query Point 34	2.5, 104	Auto: 101
35	Query Point 35	52.5, 104	Auto: 101
36	Query Point 36	27.808, 51.568	Auto: 101
37	Query Point 37	35.388, 35.978	Auto: 101
38	Query Point 38	27.5, 2.5	Auto: 101

Field Point Grid

Number of points 1428
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
122.5	202.5
122.5	-67.5
-67.5	-67.5
-67.5	202.5

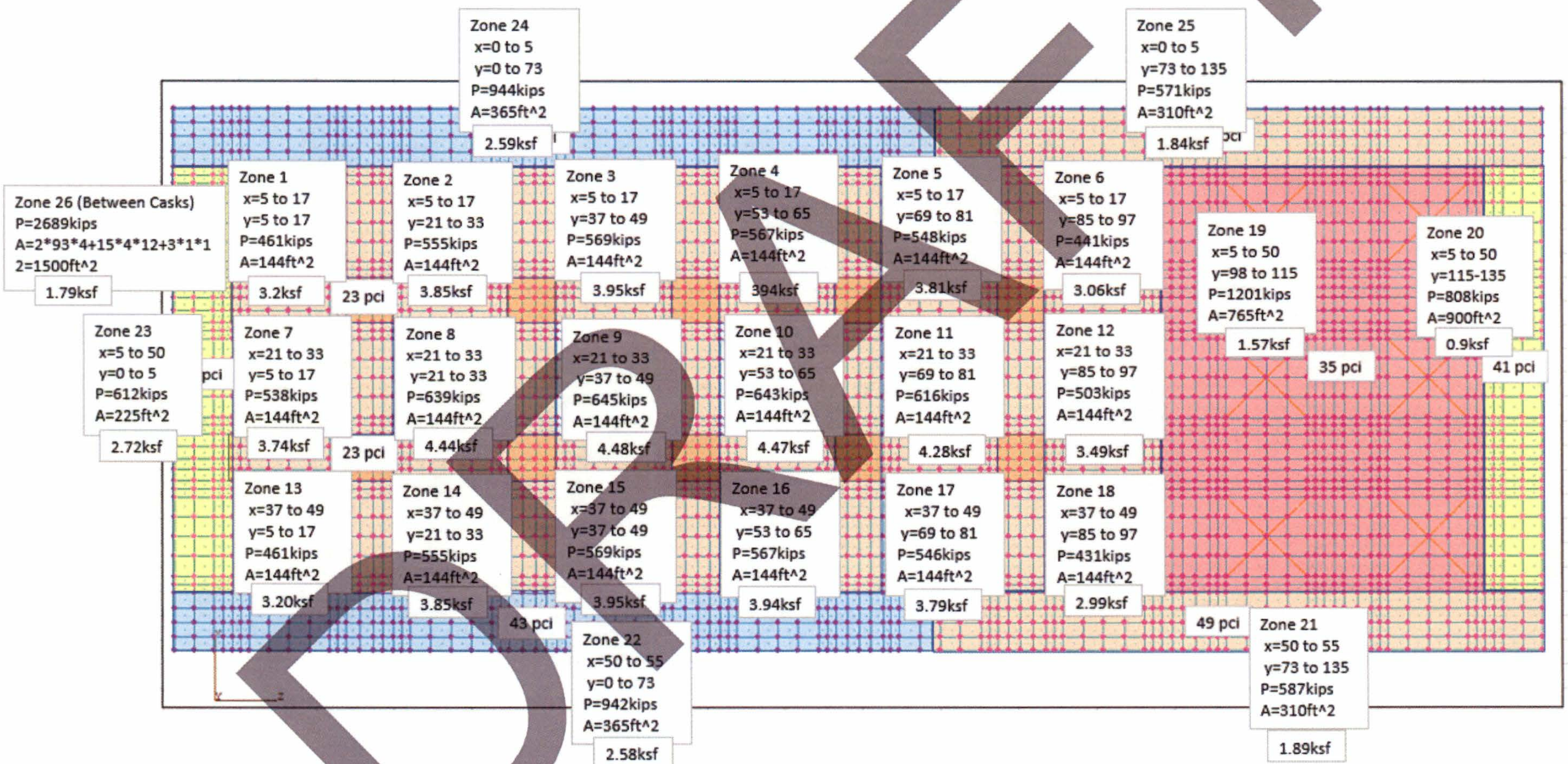
DAN BROWN AND ASSOCIATES

Configuration 4
Final Bearing Pressure Zones, Settlement Results, and Modulus of Subgrade Reaction Values for Enercon Model

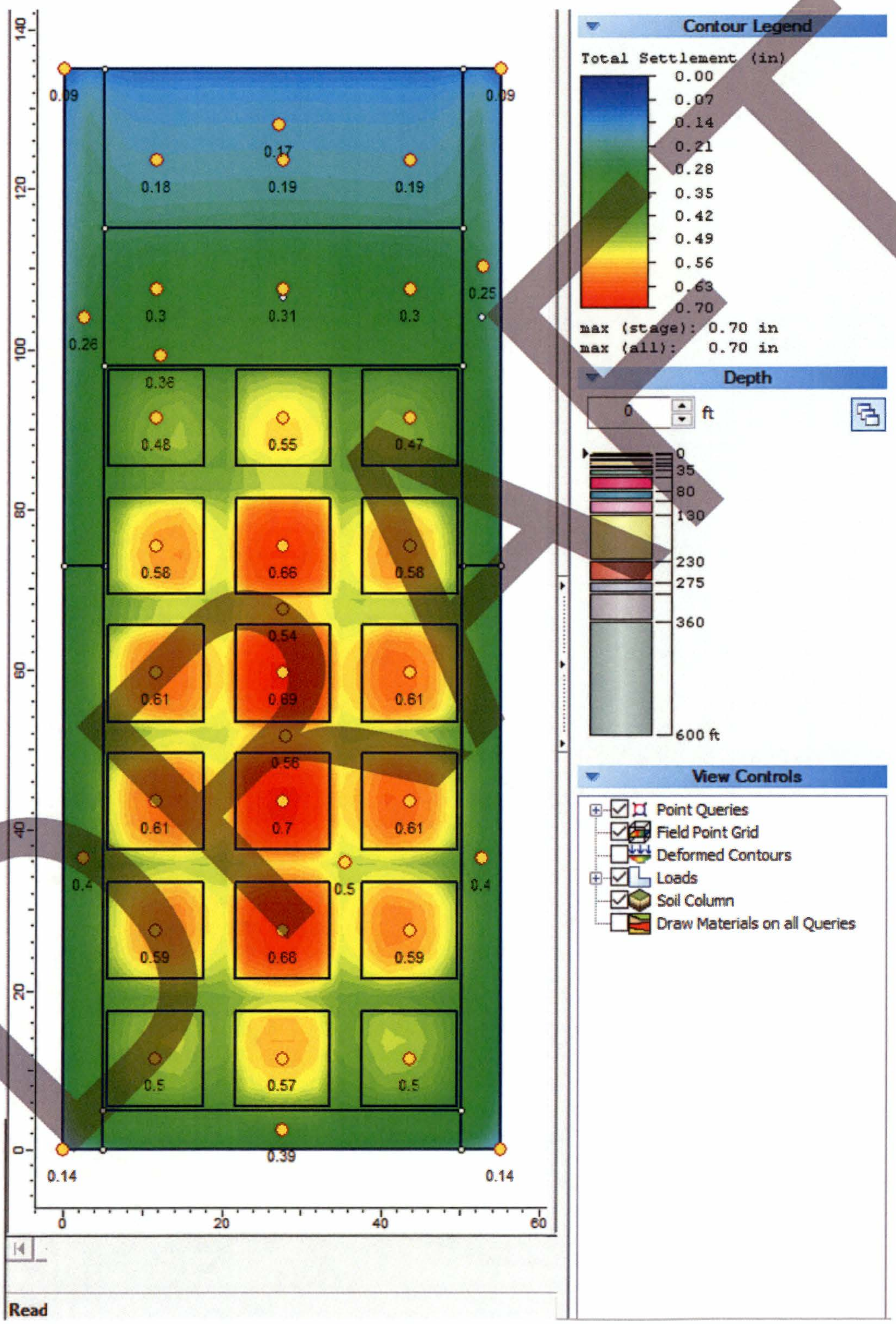
DAN BROWN AND ASSOCIATES

CISF Site
Single Pad Analysis
Andrews, TX

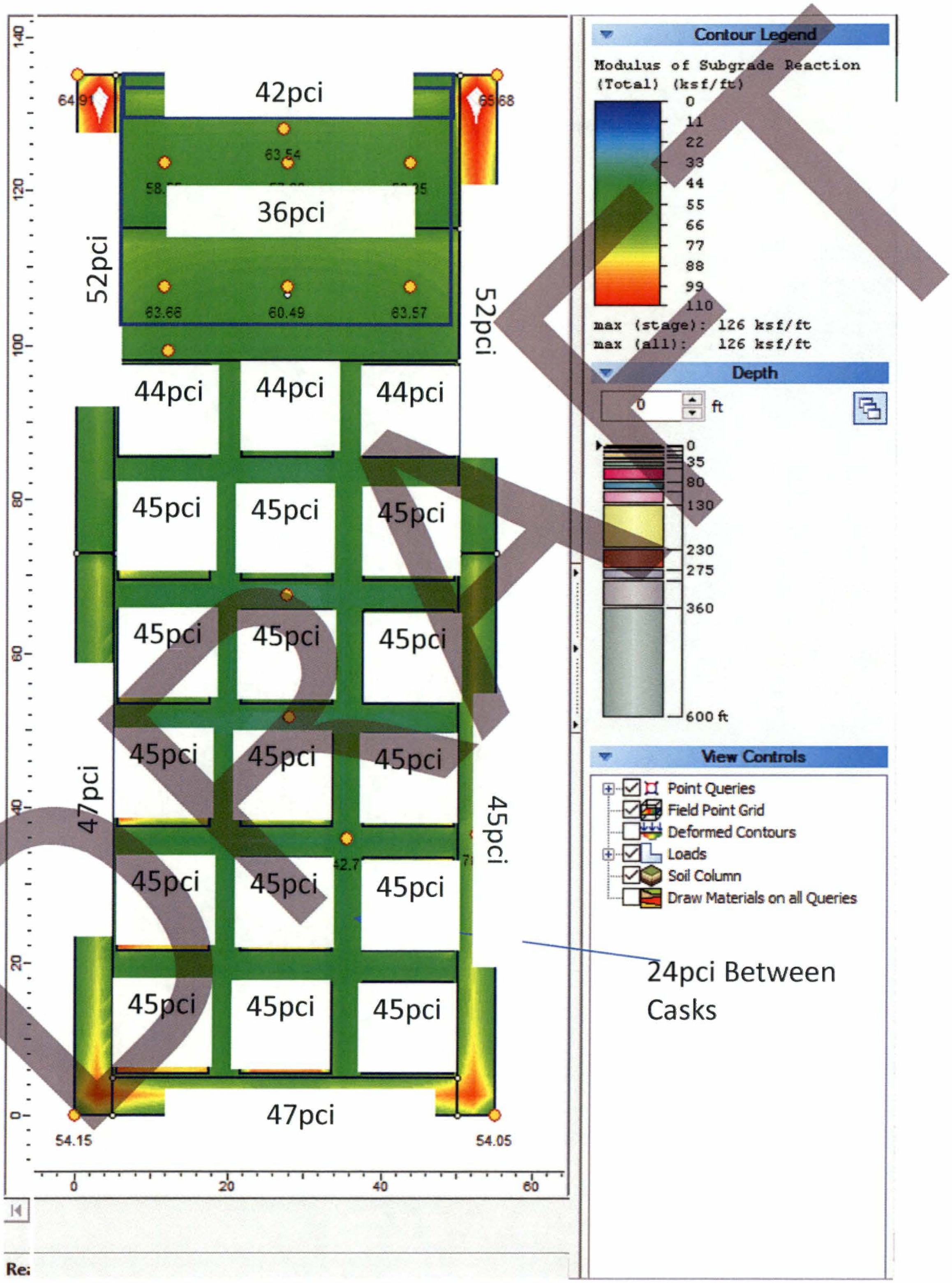
Configuration 4-Bearing Pressures (INPUT 4 Spreadsheet Loads)



Configuration 4-Settlement Estimate – Loads from INPUT4Spreadsheets)



- Configuration 4 k values – Loads from INPUT 4 Spreadsheets



DAN BROWN AND ASSOCIATES

Configuration 4 Settle3 Inputs

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF Storage Pad Config 4 200215 INPUT 4.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.695643
Loading Stress ZZ [ksf]	0.0283155	4.48

Loads

1. Polygonal Load: "Base Zone"

Label	Base Zone
Load Type	Flexible
Area of Load	7425 ft2
Load	0.9 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.41 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

3. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.41 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

4. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.14 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	53.5
49.5	53.5
49.5	65.5
37.5	65.5

5. Rectangular Load: "Zone 15"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	37.5
49.5	37.5
49.5	49.5
37.5	49.5

6. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.06 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

7. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.14 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	53.5
17.5	53.5
17.5	65.5
5.5	65.5

8. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	37.5
17.5	37.5
17.5	49.5
5.5	49.5

9. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.06 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

10. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.94 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

11. Rectangular Load: "Zone 8"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.64 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

12. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.69 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	37.5
33.5	37.5
33.5	49.5
21.5	49.5

13. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.67 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	53.5
33.5	53.5
33.5	65.5
21.5	65.5

14. Polygonal Load: "Zone 24"

Label	Zone 24
Load Type	Flexible
Area of Load	365 ft^2
Load	1.69 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	73
0	73

15. Polygonal Load: "Zone 22"

Label	Zone 22
Load Type	Flexible
Area of Load	365 ft^2
Load	1.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	73
50	73

16. Polygonal Load: "Zone 23"

Label	Zone 23
Load Type	Flexible
Area of Load	225 ft^2
Load	1.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

17. Polygonal Load: "Zone 21"

Label	Zone 21
Load Type	Flexible
Area of Load	310 ft^2
Load	1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	73
55	73
55	135
50	135

18. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	310 ft^2
Load	0.94 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	73
5	73
5	135
0	135

19. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.01 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	69.5
17.5	69.5
17.5	81.5
5.5	81.5

20. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.27 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	85.5
17.5	85.5
17.5	97.5
5.5	97.5

21. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.49 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	69.5
33.5	69.5
33.5	81.5
21.5	81.5

22. Rectangular Load: "Zone12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	85.5
33.5	85.5
33.5	97.5
21.5	97.5

23. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	69.5
49.5	69.5
49.5	81.5
37.5	81.5

24. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.2 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	85.5
49.5	85.5
49.5	97.5
37.5	97.5

25. Polygonal Load: "Zone 19"

Label	Zone 19
Load Type	Flexible
Area of Load	765 ft^2
Load	0.67 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	98
50	115
5	115
5	98

26. Polygonal Load: "Zone 26"

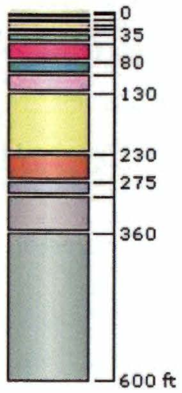
Label	Zone 26
Load Type	Flexible
Area of Load	4185 ft^2
Load	0.89 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	5
50	98
5	98
5	5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft³]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft³]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltstone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft³]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft³]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Footing Bottom Left	0, 0	Auto: 101
26	Footing Bottom Right	55, 0	Auto: 101
27	Footing Top Left	0, 135	Auto: 101
28	Footing Top Right	55, 135	Auto: 101
29	Footing Center	27.5, 67.5	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Query Point 31	26.895, 127.856	Auto: 101
32	Query Point 32	2.5, 36.5	Auto: 101
33	Query Point 33	52.5, 36.5	Auto: 101
34	Query Point 34	2.5, 104	Auto: 101
35	Query Point 35	52.794, 110.237	Auto: 101
36	Query Point 36	27.808, 51.568	Auto: 101
37	Query Point 37	35.388, 35.978	Auto: 101
38	Query Point 38	27.5, 2.5	Auto: 101

Field Point Grid

Number of points 1556
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
122.5	202.5
122.5	-67.5
-67.5	-67.5
-67.5	202.5

Overall Pad Layout from WCS
CISF Safety Analysis Report

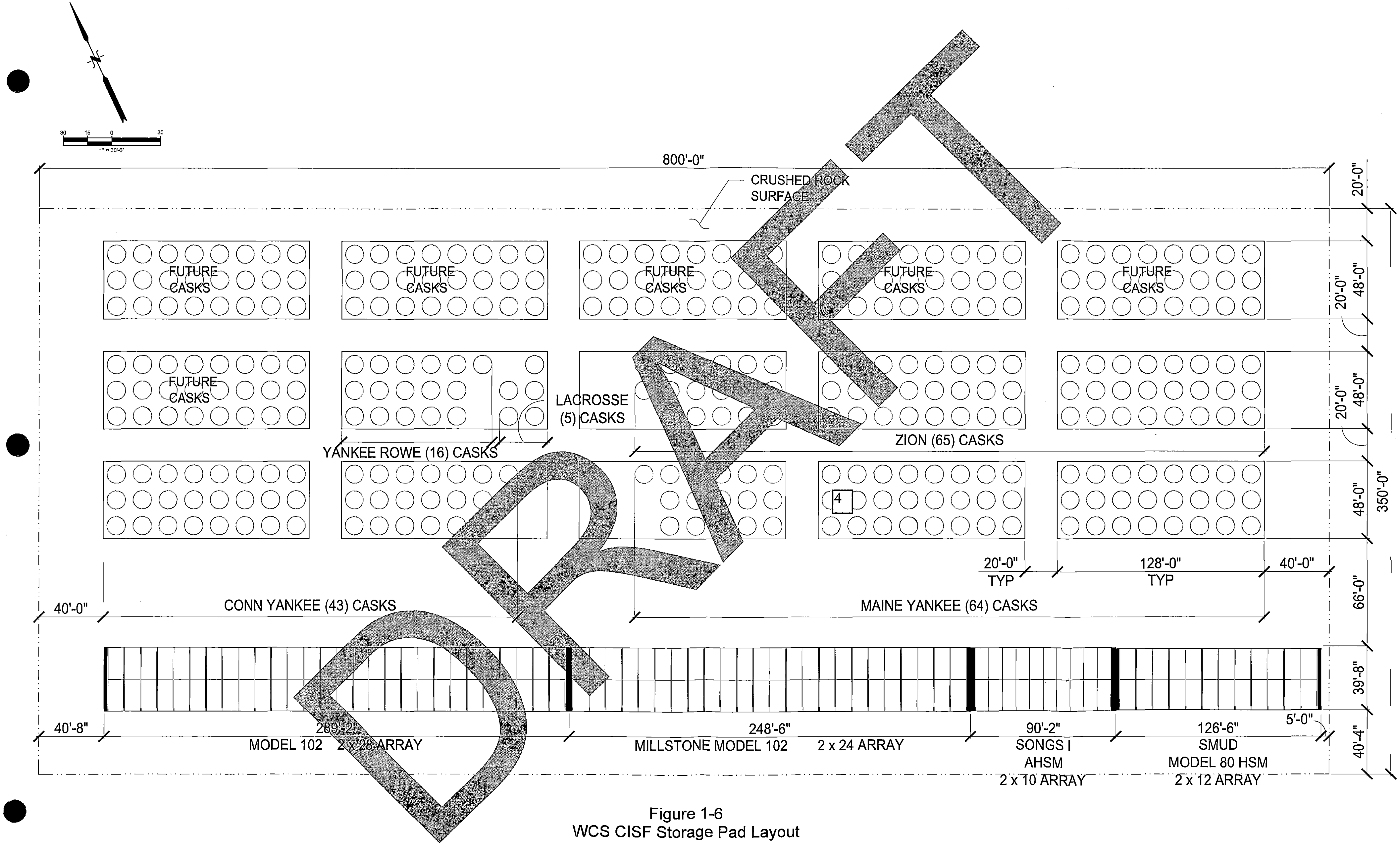


Figure 1-6
WCS CISF Storage Pad Layout

DAN BROWN AND ASSOCIATES

Four Pad Settle3 Inputs and Results

DAN BROWN AND ASSOCIATES

CISF Site
Four Storage Pads Analysis
Andrews, TX
15 February 2020

Soil Column Based on Average Shear Wave Velocity Measurements

- Interpreted from all explorations and collaboration between GEOS and DBA.
- Top 100ft contains 7 stratum in 5 materials identified by GEOS
- 100ft-600ft (approx. location of incompressible layers with sharp contrast of velocity) contains 3 average velocity values for 6 layers identified in the soil column provided by GEOS

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Layer Material (From GEOS column)	Model Layer Name
20	25	1530	Caliche - Very Hard	Caliche Hard 1
25	35	1900	Caliche - Very Hard	Caliche Hard 2
35	50	2290	Ogallala - Sand with Gravel	Ogallala 1
50	80	1840	Ogallala - Sand with Gravel	Ogallala 2
80	100	2790	Ogallala - Sand with Gravel	Ogallala 3
100	130	2300	Dockum - Claystone and Siltstone	Dockum Claystone/Siltstone
130	230	2755	Claystone and Siltstone	Claystone and Siltstone
230	275	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone1
275	300	2755	Dockum - Silty Sands	Dockum Silty/Sands
300	360	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone 2
360	600	3115	Dockum - Clay/Claystone	Dockum Clay/Claystone 3

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Unit weight (pcf)	Poisson's Ratio	Gmax (psf)
20	25	1530	125	0.33	9,087,345
25	35	1900	125	0.33	14,013,975
35	50	2290	125	0.33	20,357,531
50	80	1840	130	0.33	13,668,571
80	100	2790	130	0.33	31,426,491
100	130	2300	130	0.33	21,357,143
130	230	2755	130	0.33	30,642,958
230	275	2755	130	0.33	30,642,958
275	300	2755	130	0.33	30,642,958
300	360	2755	130	0.33	30,642,958
360	600	3115	130	0.33	39,174,511

Use Soil Column from Pad Analysis with bearing pressures from Configuration 1 Iterations (INPUT 5 final iteration).

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CISF Site
Four Storage Pads Analysis
Andrews, TX
15 February 2020

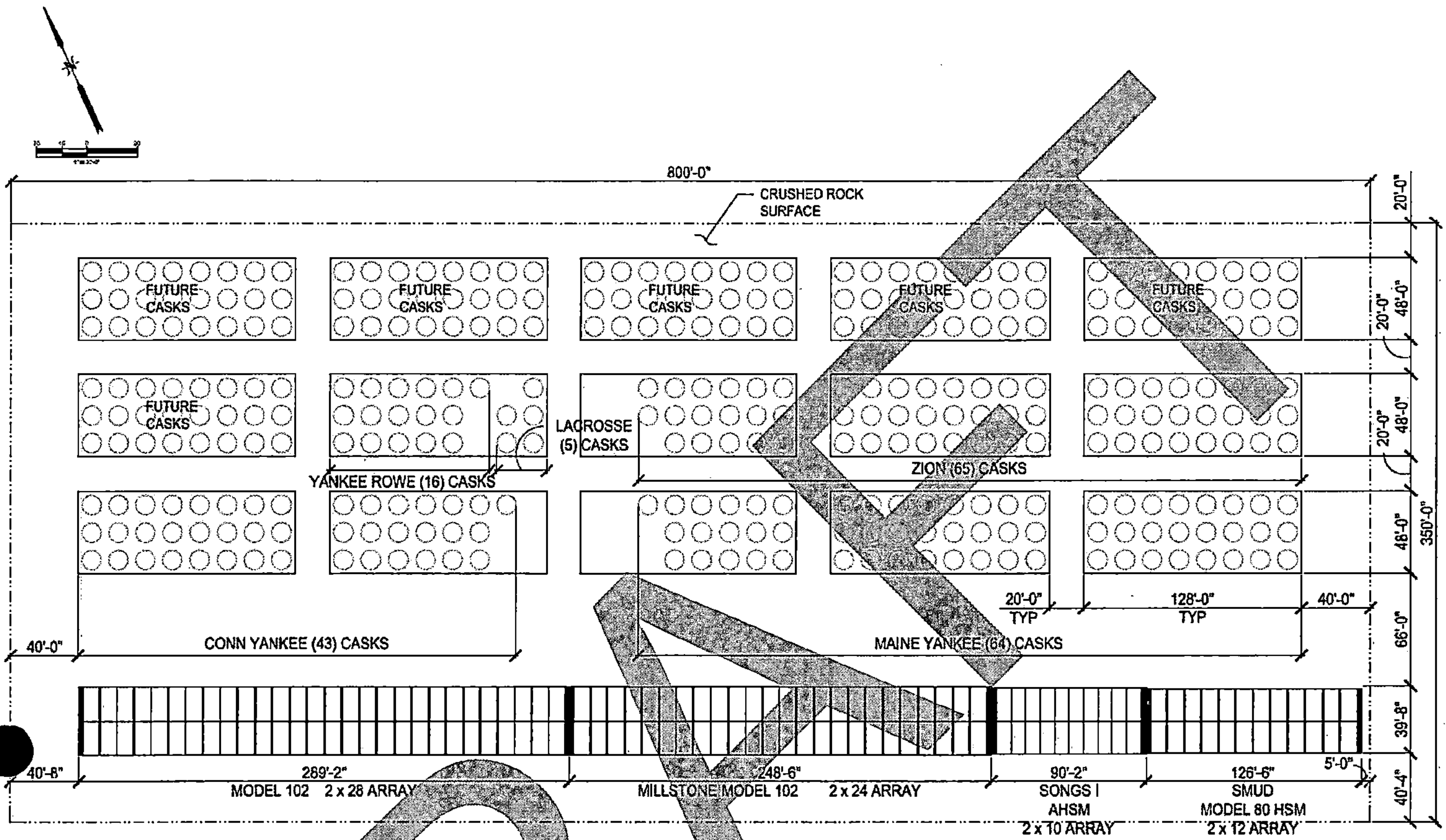


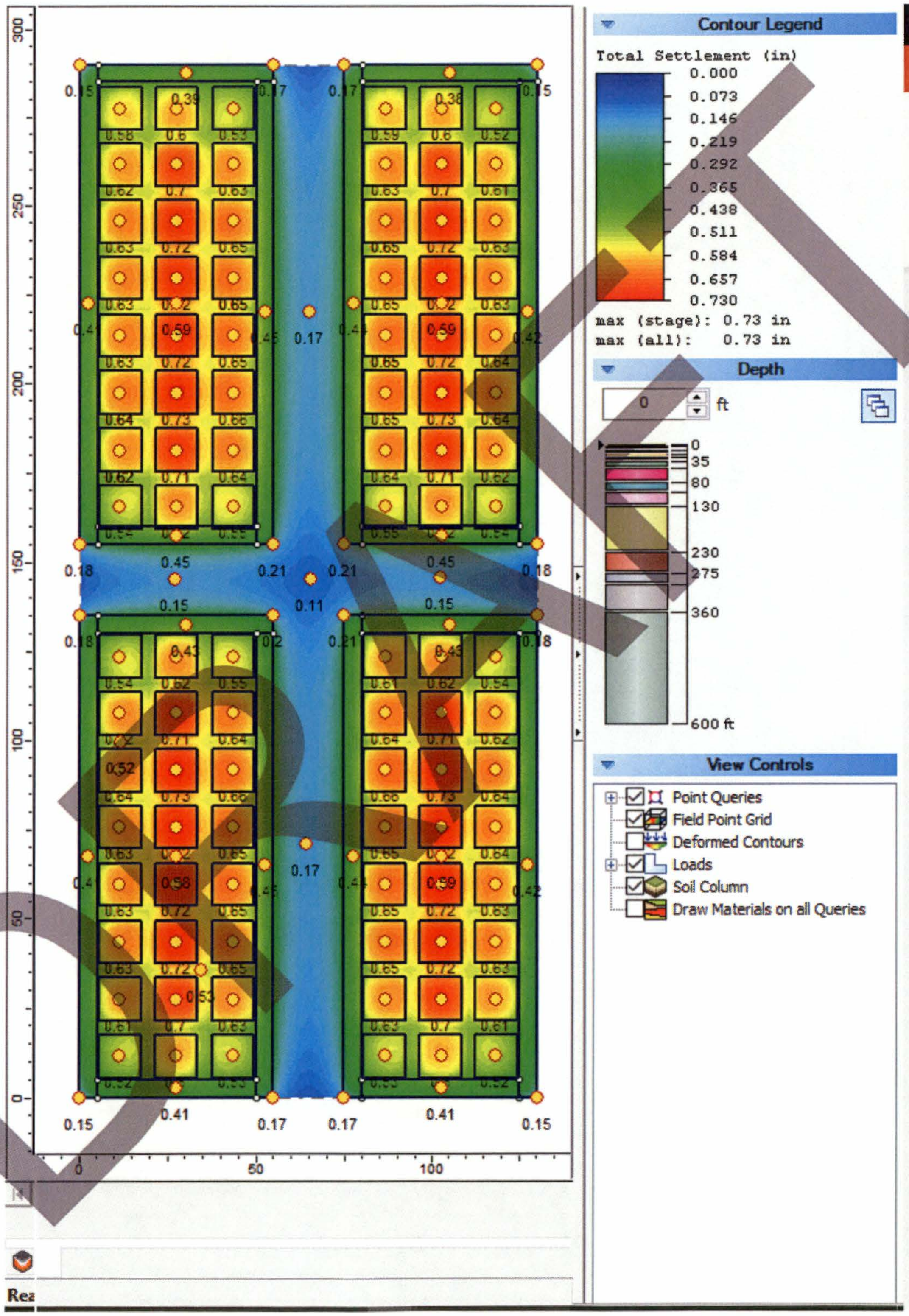
Figure 1-6

WCS Consolidated Interim Storage Facility System Safety Analysis Report, Revision 2, June 8, 2018.

Total settlement check when four pads are fully loaded simultaneously

DAN BROWN AND ASSOCIATES

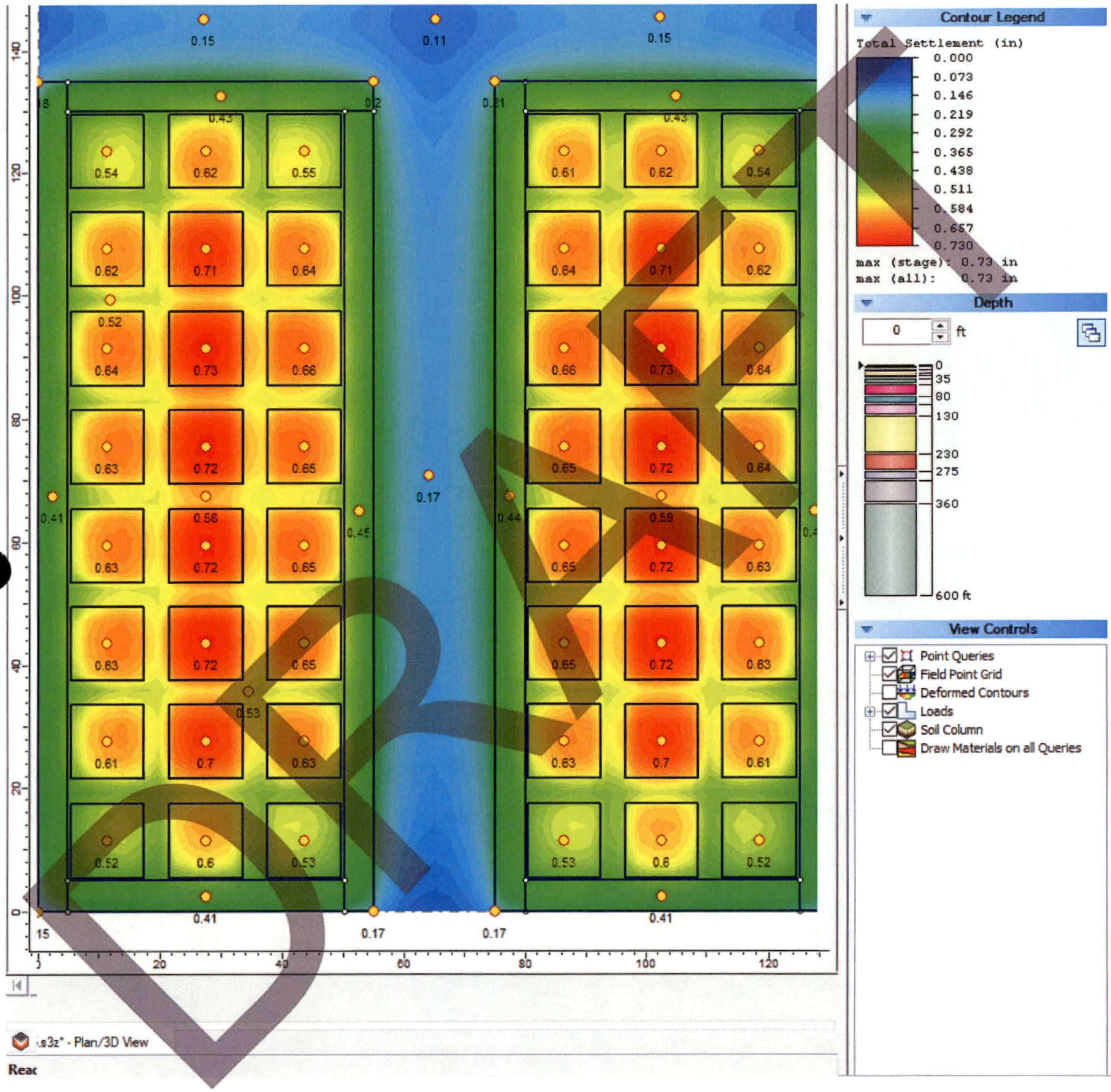
CISF Site
Four Storage Pads Analysis
Andrews, TX
15 February 2020



DAN BROWN AND ASSOCIATES

CISF Site
Four Storage Pads Analysis
Andrews, TX

15 February 2020



Stress overlap reduces differential settlement between center and corners of footings.

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF Storage Pads Four Pad 200215.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.726985
Loading Stress ZZ [ksf]	0	4.49

Loads

1. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft^2
Load	1.81 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.71 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	135
0	135

3. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	130
55	135
5	135
5	130

4. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	130
50	130

5. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

6. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

7. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

8. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	117.5
49.5	117.5
49.5	129.5
37.5	129.5

9. Rectangular Load: "Zone 8"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	117.5
17.5	117.5
17.5	129.5
5.5	129.5

10. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	101.5
49.5	101.5
49.5	113.5
37.5	113.5

11. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	85.5
49.5	85.5
49.5	97.5
37.5	97.5

12. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	69.5
49.5	69.5
49.5	81.5
37.5	81.5

13. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	53.5
49.5	53.5
49.5	65.5
37.5	65.5

14. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	37.5
49.5	37.5
49.5	49.5
37.5	49.5

15. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

16. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	117.5
33.5	117.5
33.5	129.5
21.5	129.5

17. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	101.5
17.5	101.5
17.5	113.5
5.5	113.5

18. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	85.5
17.5	85.5
17.5	97.5
5.5	97.5

19. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	69.5
17.5	69.5
17.5	81.5
5.5	81.5

20. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	53.5
17.5	53.5
17.5	65.5
5.5	65.5

21. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	37.5
17.5	37.5
17.5	49.5
5.5	49.5

22. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

23. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

24. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.58 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

25. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	37.5
33.5	37.5
33.5	49.5
21.5	49.5

26. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	53.5
33.5	53.5
33.5	65.5
21.5	65.5

27. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	69.5
33.5	69.5
33.5	81.5
21.5	81.5

28. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	85.5
33.5	85.5
33.5	97.5
21.5	97.5

29. Rectangular Load: "Zone 15"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.57 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	101.5
33.5	101.5
33.5	113.5
21.5	113.5

30. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.71 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
75	0
80	0
80	135
75	135

31. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
130	130
130	135
80	135
80	130

32. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
125	0
130	0
130	130
125	130

33. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80	0
125	0
125	5
80	5

34. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft^2
Load	1.81 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
75	0
130	0
130	135
75	135

35. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft^2
Load	1.81 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	155
55	155
55	290
0	290

36. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft^2
Load	1.81 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
75	155
130	155
130	290
75	290

37. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.71 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
75	155
80	155
80	290
75	290

38. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.71 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	155
5	155
5	290
0	290

39. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
125	155
130	155
130	285
125	285

40. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
130	285
130	290
80	290
80	285

41. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	285
55	290
5	290
5	285

42. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.75 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	155
55	155
55	285
50	285

43. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80	155
125	155
125	160
80	160

44. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.82 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	155
50	155
50	160
5	160

45. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	5.5
92.5	5.5
92.5	17.5
80.5	17.5

46. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	159.5
92.5	159.5
92.5	171.5
80.5	171.5

47. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	159.5
17.5	159.5
17.5	171.5
5.5	171.5

48. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	21.5
92.5	21.5
92.5	33.5
80.5	33.5

49. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	175.5
92.5	175.5
92.5	187.5
80.5	187.5

50. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	175.5
17.5	175.5
17.5	187.5
5.5	187.5

51. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	191.5
17.5	191.5
17.5	203.5
5.5	203.5

52. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	191.5
92.5	191.5
92.5	203.5
80.5	203.5

53. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	37.5
92.5	37.5
92.5	49.5
80.5	49.5

54. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	207.5
17.5	207.5
17.5	219.5
5.5	219.5

55. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	207.5
92.5	207.5
92.5	219.5
80.5	219.5

56. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	53.5
92.5	53.5
92.5	65.5
80.5	65.5

57. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	223.5
17.5	223.5
17.5	235.5
5.5	235.5

58. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	223.5
92.5	223.5
92.5	235.5
80.5	235.5

59. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	239.5
17.5	239.5
17.5	251.5
5.5	251.5

60. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	239.5
92.5	239.5
92.5	251.5
80.5	251.5

61. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	255.5
17.5	255.5
17.5	267.5
5.5	267.5

62. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	255.5
92.5	255.5
92.5	267.5
80.5	267.5

63. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	271.5
17.5	271.5
17.5	283.5
5.5	283.5

64. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	271.5
92.5	271.5
92.5	283.5
80.5	283.5

65. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	5.5
108.5	5.5
108.5	17.5
96.5	17.5

66. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	69.5
92.5	69.5
92.5	81.5
80.5	81.5

67. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	85.5
92.5	85.5
92.5	97.5
80.5	97.5

68. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	101.5
92.5	101.5
92.5	113.5
80.5	113.5

69. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
80.5	117.5
92.5	117.5
92.5	129.5
80.5	129.5

70. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.58 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	21.5
108.5	21.5
108.5	33.5
96.5	33.5

71. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.58 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	175.5
108.5	175.5
108.5	187.5
96.5	187.5

72. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.58 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	175.5
33.5	175.5
33.5	187.5
21.5	187.5

73. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	159.5
33.5	159.5
33.5	171.5
21.5	171.5

74. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	159.5
108.5	159.5
108.5	171.5
96.5	171.5

75. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	191.5
33.5	191.5
33.5	203.5
21.5	203.5

76. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	191.5
108.5	191.5
108.5	203.5
96.5	203.5

77. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	37.5
108.5	37.5
108.5	49.5
96.5	49.5

78. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	53.5
108.5	53.5
108.5	65.5
96.5	65.5

79. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	207.5
108.5	207.5
108.5	219.5
96.5	219.5

80. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	207.5
33.5	207.5
33.5	219.5
21.5	219.5

81. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	69.5
108.5	69.5
108.5	81.5
96.5	81.5

82. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	223.5
108.5	223.5
108.5	235.5
96.5	235.5

83. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.63 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	223.5
33.5	223.5
33.5	235.5
21.5	235.5

84. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	85.5
108.5	85.5
108.5	97.5
96.5	97.5

85. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	239.5
108.5	239.5
108.5	251.5
96.5	251.5

86. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.68 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	239.5
33.5	239.5
33.5	251.5
21.5	251.5

87. Rectangular Load: "Zone 15"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.57 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	255.5
33.5	255.5
33.5	267.5
21.5	267.5

88. Rectangular Load: "Zone 15"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.57 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	255.5
108.5	255.5
108.5	267.5
96.5	267.5

89. Rectangular Load: "Zone 15"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.57 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	101.5
108.5	101.5
108.5	113.5
96.5	113.5

90. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	271.5
33.5	271.5
33.5	283.5
21.5	283.5

91. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	271.5
108.5	271.5
108.5	283.5
96.5	283.5

92. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	271.5
49.5	271.5
49.5	283.5
37.5	283.5

93. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	271.5
124.5	271.5
124.5	283.5
112.5	283.5

94. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	117.5
124.5	117.5
124.5	129.5
112.5	129.5

95. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.04 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
96.5	117.5
108.5	117.5
108.5	129.5
96.5	129.5

96. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	101.5
124.5	101.5
124.5	113.5
112.5	113.5

97. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	255.5
124.5	255.5
124.5	267.5
112.5	267.5

98. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	255.5
49.5	255.5
49.5	267.5
37.5	267.5

99. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	85.5
124.5	85.5
124.5	97.5
112.5	97.5

100. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	239.5
124.5	239.5
124.5	251.5
112.5	251.5

101. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	239.5
49.5	239.5
49.5	251.5
37.5	251.5

102. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	69.5
124.5	69.5
124.5	81.5
112.5	81.5

103. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	223.5
124.5	223.5
124.5	235.5
112.5	235.5

104. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	223.5
49.5	223.5
49.5	235.5
37.5	235.5

105. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	53.5
124.5	53.5
124.5	65.5
112.5	65.5

106. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	207.5
124.5	207.5
124.5	219.5
112.5	219.5

107. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.1 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	207.5
49.5	207.5
49.5	219.5
37.5	219.5

108. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	37.5
124.5	37.5
124.5	49.5
112.5	49.5

109. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	191.5
124.5	191.5
124.5	203.5
112.5	203.5

110. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.16 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	191.5
49.5	191.5
49.5	203.5
37.5	203.5

111. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	175.5
49.5	175.5
49.5	187.5
37.5	187.5

112. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	175.5
124.5	175.5
124.5	187.5
112.5	187.5

113. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	159.5
49.5	159.5
49.5	171.5
37.5	171.5

114. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	159.5
124.5	159.5
124.5	171.5
112.5	171.5

115. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	1.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	5.5
124.5	5.5
124.5	17.5
112.5	17.5

116. Rectangular Load: "Zone 18"

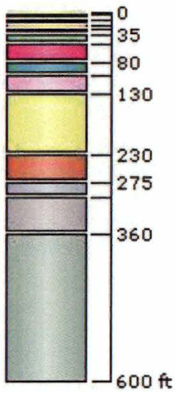
Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.09 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
112.5	21.5
124.5	21.5
124.5	33.5
112.5	33.5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Pad 1 Bottom Left	0, 0	Auto: 101
26	Pad 1 Bottom Right	55, 0	Auto: 101
27	Pad 1 Top Left	0, 135	Auto: 101
28	Pad 1 Top Right	55, 135	Auto: 101
29	Pad 1 Center	27.5, 67.5	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Pad 1 Top Edge	30, 132.5	Auto: 101
32	Pad 1 Right Edge	52.5, 65	Auto: 101
33	Query Point 33	34.643, 35.669	Auto: 101
34	Pad 1 Bottom Edge	27.5, 2.5	Auto: 101
35	Pad 2 Cask 1	11.5, 165.5	Auto: 101
36	Query Point 36	11.5, 181.5	Auto: 101
37	Query Point 37	11.5, 197.5	Auto: 101
38	Pad 2 Cask 4	11.5, 213.5	Auto: 101
39	Pad 2 Cask 5	11.5, 229.5	Auto: 101
40	Pad 2 Cask 6	11.5, 245.5	Auto: 101
41	Pad 2 Cask 7	11.5, 261.5	Auto: 101
42	Pad 2 Cask 8	11.5, 277.5	Auto: 101
43	Pad 2 Cask 9	27.5, 165.5	Auto: 101
44	Pad 2 Cask 10	27.5, 181.5	Auto: 101
45	Pad 2 Cask 11	27.5, 197.5	Auto: 101
46	Pad 2 Cask 12	27.5, 213.5	Auto: 101
47	Pad 2 Cask 13	27.5, 229.5	Auto: 101
48	Pad 2 Cask 14	27.5, 245.5	Auto: 101
49	Pad 2 Cask 15	27.5, 261.5	Auto: 101
50	Pad 2 Cask 16	27.5, 277.5	Auto: 101
51	Pad 2 Cask 17	43.5, 165.5	Auto: 101
52	Pad 2 Cask 18	43.5, 181.5	Auto: 101
53	Pad 2 Cask 19	43.5, 197.5	Auto: 101
54	Pad 2 Cask 20	43.5, 213.5	Auto: 101
55	Query Point 55	43.5, 229.5	Auto: 101
56	Pad 2 Cask 21	43.5, 229.5	Auto: 101
57	Pad 2 Cask 22	43.5, 245.5	Auto: 101

58	Pad 2 Cask 23	43.5, 261.5	Auto: 101
59	Pad 2 Cask 24	43.5, 277.5	Auto: 101
60	Pad 4 Cask 1	86.5, 11.5	Auto: 101
61	Query Point 61	11.5, 181.5	Auto: 101
62	Pad 2 Cask 3	11.5, 197.5	Auto: 101
63	Pad 2 Cask 2	11.5, 181.5	Auto: 101
64	Pad 4 Cask 2	86.5, 27.5	Auto: 101
65	Pad 4 Cask 3	86.5, 43.5	Auto: 101
66	Pad 4 Cask 4	86.5, 59.5	Auto: 101
67	Pad 4 Cask 6	86.5, 91.5	Auto: 101
68	Pad 4 Cask 5	86.5, 75.5	Auto: 101
69	Pad 4 Cask 7	86.5, 107.5	Auto: 101
70	Pad 4 Cask 8	86.5, 123.5	Auto: 101
71	Pad 4 Cask 9	102.5, 11.5	Auto: 101
72	Pad 4 Cask 10	102.5, 27.5	Auto: 101
73	Pad 4 Cask 11	102.5, 43.5	Auto: 101
74	Pad 4 Cask 12	102.5, 59.5	Auto: 101
75	Pad 4 Cask 13	102.5, 75.5	Auto: 101
76	Pad 4 Cask 14	102.5, 91.5	Auto: 101
77	Pad 4 Cask 15	102.5, 107.5	Auto: 101
78	Pad 4 Cask 16	102.5, 123.5	Auto: 101
79	Pad 4 Cask 17	118.5, 11.5	Auto: 101
80	Pad 4 Cask 18	118.5, 27.5	Auto: 101
81	Pad 4 Cask 20	118.5, 59.5	Auto: 101
82	Pad 4 Cask 19	118.5, 43.5	Auto: 101
83	Pad 4 Cask 21	118.5, 75.5	Auto: 101
84	Pad 4 Cask 22	118.5, 91.5	Auto: 101
85	Pad 4 Cask 23	118.5, 107.5	Auto: 101
86	Pad 4 Cask 24	118.5, 123.5	Auto: 101
87	Pad 3 Cask 1	86.5, 165.5	Auto: 101
88	Pad 3 Cask 2	86.5, 181.5	Auto: 101
89	Pad 3 Cask 3	86.5, 197.5	Auto: 101
90	Query Point 90	86.5, 213.5	Auto: 101
91	Pad 3 Cask 4	86.5, 213.5	Auto: 101
92	Query Point 92	86.5, 245.5	Auto: 101
93	Pad 3 Cask 6	86.5, 245.5	Auto: 101
94	Pad 3 Cask 5	86.5, 229.5	Auto: 101
95	Pad 3 Cask 7	86.5, 261.5	Auto: 101
96	Pad 3 Cask 8	86.5, 277.5	Auto: 101
97	Pad 3 Cask 9	102.5, 165.5	Auto: 101
98	Pad 3 Cask 11	102.5, 197.5	Auto: 101
99	Pad 3 Cask 10	102.5, 181.5	Auto: 101
100	Pad 3 Cask 12	102.5, 213.5	Auto: 101
101	Pad 3 Cask 13	102.5, 229.5	Auto: 101
102	Pad 3 Cask 14	102.5, 245.5	Auto: 101
103	Pad 3 Cask 15	102.5, 261.5	Auto: 101
104	Pad 3 Cask 16	102.5, 277.5	Auto: 101
105	Pad 3 Cask 17	118.5, 165.5	Auto: 101
106	Pad 3 Cask 18	118.5, 181.5	Auto: 101
107	Pad 3 Cask 19	118.5, 197.5	Auto: 101
108	Pad 3 Cask 20	118.5, 213.5	Auto: 101
109	Pad 3 Cask 21	118.5, 229.5	Auto: 101
110	Pad 3 Cask 22	118.5, 245.5	Auto: 101
111	Pad 3 Cask 23	118.5, 261.5	Auto: 101
112	Pad 3 Cask 24	118.5, 277.5	Auto: 101
113	Pad 1 Left edge	2.5, 67.5	Auto: 101
114	Query Point 114	27.5, 157.5	Auto: 101
115	Query Point 115	2.5, 222.5	Auto: 101
116	Query Point 116	52.5, 220	Auto: 101
117	Query Point 117	30, 287.5	Auto: 101
118	Query Point 118	105, 287.5	Auto: 101

119	Query Point 119	127.5, 220	Auto: 101
120	Query Point 120	77.5, 222.5	Auto: 101
121	Query Point 121	102.5, 157.5	Auto: 101
122	Query Point 122	105, 132.5	Auto: 101
123	Query Point 123	102.5, 2.5	Auto: 101
124	Query Point 124	77.5, 67.5	Auto: 101
125	Query Point 125	127.5, 65	Auto: 101
126	Query Point 126	130, 135	Auto: 101
127	Pad 4 Top Left	75, 135	Auto: 101
128	Pad 3 Bottom Left	75, 155	Auto: 101
129	Pad 3 Bottom Right	130, 155	Auto: 101
130	Pad 3 Top Left	75, 290	Auto: 101
131	Pad 3 Top Right	130, 290	Auto: 101
132	Pad 2 Top Left	0, 290	Auto: 101
133	Pad 2 Top Right	55, 290	Auto: 101
134	Pad 2 Bottom Left	0, 155	Auto: 101
135	Pad 2 Bottom Right	55, 155	Auto: 101
136	Pad 4 Bottom Left	75, 0	Auto: 101
137	Pad 4 Bottom Right	130, 0	Auto: 101
138	Pad 4 Top Right	130, 135	Auto: 101
139	Center of Pads	65.278, 145.171	Auto: 89
140	Between Pad 1 and Pad 2	27.143, 145.171	Auto: 89
141	Between Pad 3 and Pad 4	102.269, 145.552	Auto: 89
142	Between Pad 1 and Pad 4	64.134, 70.808	Auto: 89
143	Between Pad 2 and Pad 3	64.897, 220.297	Auto: 89
144	Pad 2 Center	27.5, 222.5	Auto: 101
145	Pad 3 Center	102.5, 222.5	Auto: 101
146	Pad 4 Center	102.5, 67.5	Auto: 101

Field Point Grid

Number of points 7876
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
197.5	357.5
197.5	-67.5
-67.5	-67.5
-67.5	357.5

DAN BROWN AND ASSOCIATES

Cask Handling Building Settle3 Analysis, Inputs and Results

CISF Site
Cask Handling Building
Andrews, TX

Cask Handling Building: Two-bay Category B steel struction measuring approximately 175feet by 193 feet in plan dimension with a height of 72 feet.

SERVICE LEVEL MAXIMUM BEARING PRESSURES

Load Combination	Foundations for Main Columns		Foundations for Wind Columns	
	Gross Bearing Pressure (ksf)	Net Bearing Pressure (ksf)	Bearing Pressure (ksf)	Net Bearing Pressure (ksf)
Dead 1.0D	1.79	0.66	1.67	0.55
Operating Wind 1.0D + 0.6W	2.59	1.47	1.85	0.73
Seismic 1.0D + 0.7E	3.34	2.21	1.86	0.74

LIMIT STATE MAXIMUM BEARING PRESSURES

Load Combination	Foundations for Main Columns		Foundations for Wind Columns	
	Gross Bearing Pressure (ksf)	Net Bearing Pressure (ksf)	Bearing Pressure (ksf)	Net Bearing Pressure (ksf)
Tornado 1.2D + 1.0W_t	4.67	2.51	5.44	4.32
Seismic 1.2D + 1.0E	4.21	3.09	2.23	1.11

DAN BROWN AND ASSOCIATES

CISF Site
Cask Handling Building
Andrews, TX

Use Soil Column from Pad Analysis with bottom of footing 10ft below GS.

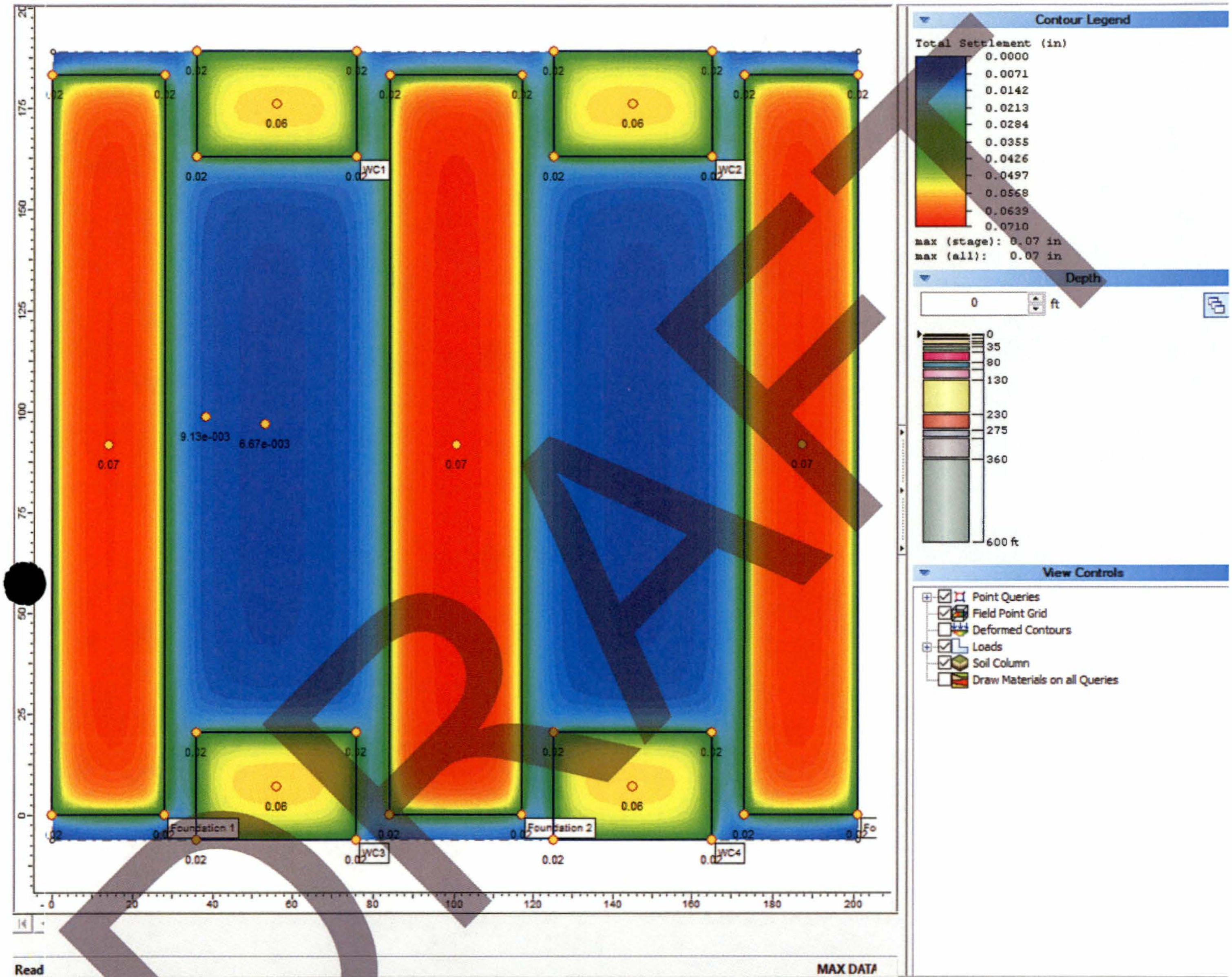
Soil Column Based on Average Shear Wave Velocity Measurements

- Interpreted from all explorations and collaboration between GEOS and DBA.
- Top 100ft contains 7 stratum in 5 materials identified by GEOS
- 100ft-600ft (approx. location of incompressible layers with sharp contrast of velocity) contains 3 average velocity values for 6 layers identified in the soil column provided by GEOS

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Layer Material (From GEOS column)	Model Layer Name
20	25	1530	Caliche - Very Hard	Caliche Hard 1
25	35	1900	Caliche - Very Hard	Caliche Hard 2
35	50	2290	Ogallala - Sand with Gravel	Ogallala 1
50	80	1840	Ogallala - Sand with Gravel	Ogallala 2
80	100	2790	Ogallala - Sand with Gravel	Ogallala 3
100	130	2300	Dockum - Claystone and Siltstone	Dockum Claystone/Siltstone
130	230	2755	Claystone and Siltstone	Claystone and Siltstone
230	275	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone1
275	300	2755	Dockum - Silty Sands	Dockum Silty/Sands
300	360	2755	Dockum - Clay/Claystone	Dockum Clay/Claystone 2
360	600	3115	Dockum - Clay/Claystone	Dockum Clay/Claystone 3

Top (ft)	Bottom (ft)	Avg. Layer Shear Wave Velocity (ft/s)	Unit weight (pcf)	Poisson's Ratio	Gmax (psf)
20	25	1530	125	0.33	9,087,345
25	35	1900	125	0.33	14,013,975
35	50	2290	125	0.33	20,357,531
50	80	1840	130	0.33	13,668,571
80	100	2790	130	0.33	31,426,491
100	130	2300	130	0.33	21,357,143
130	230	2755	130	0.33	30,642,958
230	275	2755	130	0.33	30,642,958
275	300	2755	130	0.33	30,642,958
300	360	2755	130	0.33	30,642,958
360	600	3115	130	0.33	39,174,511

CISF Site
Cask Handling Building
Andrews, TX



Net Bearing for DL Case per AECOM Request (0.660ksf on Foundations 1-3 and 0.550ksf on Wind Column Footings). 0.10" or less reported at foundation centers. Likely outside of the reasonable bounds of the calculation. Recommend considering 0.25" or less

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF CHB 200214 Net DL Only.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 17.0959 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.0706837
Loading Stress ZZ [ksf]	0	0.66002

Loads

1. Polygonal Load: "Foundation 1"

Label	Foundation 1
Load Type	Flexible
Area of Load	5126.8 ft^2
Load	0.66 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
28	0
28	183.1
0	183.1

2. Polygonal Load: "Foundation 2"

Label	Foundation 2
Load Type	Flexible
Area of Load	6042.3 ft^2
Load	0.66 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
84	0
117	0
117	183.1
84	183.1

3. Polygonal Load: "Foundation 3"

Label	Foundation 3
Load Type	Flexible
Area of Load	5126.8 ft^2
Load	0.66 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
173.1	0
201.1	0
201.1	183.1
173.1	183.1

4. Polygonal Load: "WC3"

Label	WC3
Load Type	Flexible
Area of Load	1064 ft^2
Load	0.55 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
76	-6.3
76	20.3
36	20.3
36	-6.3

5. Polygonal Load: "WC1"

Label	WC1
Load Type	Flexible
Area of Load	1052 ft^2
Load	0.55 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
76	162.8
76	189.1
36	189.1
36	162.8

6. Polygonal Load: "WC4"

Label	WC4
Load Type	Flexible
Area of Load	1064 ft^2
Load	0.55 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
165	-6.3
165	20.3
125	20.3
125	-6.3

7. Polygonal Load: "WC2"

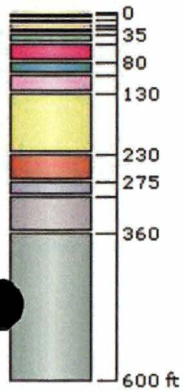
Label	WC2
Load Type	Flexible
Area of Load	1052 ft^2
Load	0.55 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
165	162.8
165	189.1
125	189.1
125	162.8

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Query Point 1	0, 0	Auto: 101
2	Query Point 2	0, 183.1	Auto: 101
3	Query Point 3	28, 183.1	Auto: 101
4	Query Point 4	28, 0	Auto: 101
5	Query Point 5	14, 91.55	Auto: 101
6	Query Point 6	36, -6.3	Auto: 101
7	Query Point 7	36, 20.3	Auto: 101
8	Query Point 8	76, 20.3	Auto: 101
9	Query Point 9	76, -6.3	Auto: 101
10	Query Point 10	56, 7	Auto: 101
11	Query Point 11	36, 162.8	Auto: 101
12	Query Point 12	36, 189.1	Auto: 101
13	Query Point 13	76, 189.1	Auto: 101
14	Query Point 14	76, 162.8	Auto: 101
15	Query Point 15	56, 175.95	Auto: 101
16	Query Point 16	84, 0	Auto: 101
17	Query Point 17	84, 183.1	Auto: 101
18	Query Point 18	117, 183.1	Auto: 101
19	Query Point 19	117, 0	Auto: 101
20	Query Point 20	100.5, 91.55	Auto: 101
21	Query Point 21	125, -6.3	Auto: 101
22	Query Point 22	125, 20.3	Auto: 101
23	Query Point 23	165, 20.3	Auto: 101
24	Query Point 24	165, -6.3	Auto: 101
25	Query Point 25	145, 7	Auto: 101
26	Query Point 26	125, 162.8	Auto: 101
27	Query Point 27	125, 189.1	Auto: 101
28	Query Point 28	165, 189.1	Auto: 101
29	Query Point 29	165, 162.8	Auto: 101
30	Query Point 30	145, 175.95	Auto: 101
31	Query Point 31	173.1, 0	Auto: 101
32	Query Point 32	173.1, 183.1	Auto: 101
33	Query Point 33	201.1, 183.1	Auto: 101
34	Query Point 34	201.1, 0	Auto: 101
35	Query Point 35	187.1, 91.55	Auto: 101
36	Query Point 36	38.215, 98.684	Auto: 89
37	Query Point 37	53.261, 96.899	Auto: 89

Field Point Grid

Number of points 5512
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
286.65	274.65
286.65	-91.85
-85.55	-91.85
-85.55	274.65

CISF Site
Cask Handling Building
Andrews, TX

Sustained Dead Loading Pressure Only (1.79ksf on Foundations 1-3 and 1.67ksf on Wind Column Footings). 0.25" or less at foundation centers

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF CHB 200214 DL Only.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.375565
Loading Stress ZZ [ksf]	0	3.51092

Loads

1. Polygonal Load: "Foundation 1"

Label	Foundation 1
Load Type	Flexible
Area of Load	5126.8 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
28	0
28	183.1
0	183.1

2. Polygonal Load: "Foundation 2"

Label	Foundation 2
Load Type	Flexible
Area of Load	6042.3 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
84	0
117	0
117	183.1
84	183.1

3. Polygonal Load: "Foundation 3"

Label	Foundation 3
Load Type	Flexible
Area of Load	5126.8 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
173.1	0
201.1	0
201.1	183.1
173.1	183.1

4. Polygonal Load: "WC3"

Label	WC3
Load Type	Flexible
Area of Load	1064 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
76	-6.3
76	20.3
36	20.3
36	-6.3

5. Polygonal Load: "WC1"

Label	WC1
Load Type	Flexible
Area of Load	1052 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
76	162.8
76	189.1
36	189.1
36	162.8

6. Polygonal Load: "WC4"

Label	WC4
Load Type	Flexible
Area of Load	1064 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
165	-6.3
165	20.3
125	20.3
125	-6.3

7. Polygonal Load: "WC2"

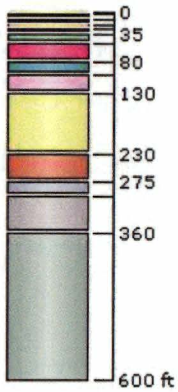
Label	WC2
Load Type	Flexible
Area of Load	1052 ft^2
Load	3.5 ksf
Depth	10 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
165	162.8
165	189.1
125	189.1
125	162.8

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

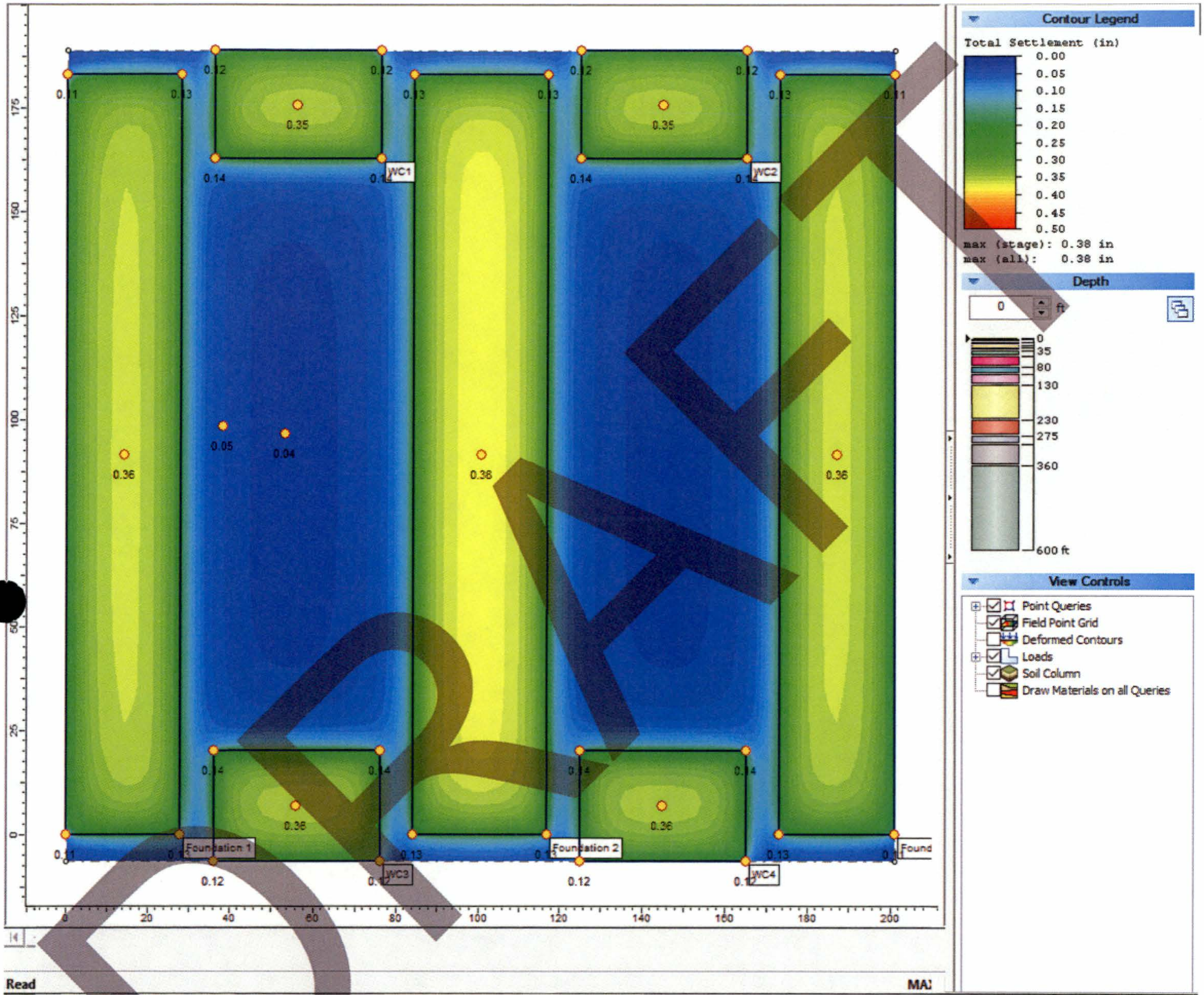
Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Query Point 1	0, 0	Auto: 101
2	Query Point 2	0, 183.1	Auto: 101
3	Query Point 3	28, 183.1	Auto: 101
4	Query Point 4	28, 0	Auto: 101
5	Query Point 5	14, 91.55	Auto: 101
6	Query Point 6	36, -6.3	Auto: 101
7	Query Point 7	36, 20.3	Auto: 101
8	Query Point 8	76, 20.3	Auto: 101
9	Query Point 9	76, -6.3	Auto: 101
10	Query Point 10	56, 7	Auto: 101
11	Query Point 11	36, 162.8	Auto: 101
12	Query Point 12	36, 189.1	Auto: 101
13	Query Point 13	76, 189.1	Auto: 101
14	Query Point 14	76, 162.8	Auto: 101
15	Query Point 15	56, 175.95	Auto: 101
16	Query Point 16	84, 0	Auto: 101
17	Query Point 17	84, 183.1	Auto: 101
18	Query Point 18	117, 183.1	Auto: 101
19	Query Point 19	117, 0	Auto: 101
20	Query Point 20	100.5, 91.55	Auto: 101
21	Query Point 21	125, -6.3	Auto: 101
22	Query Point 22	125, 20.3	Auto: 101
23	Query Point 23	165, 20.3	Auto: 101
24	Query Point 24	165, -6.3	Auto: 101
25	Query Point 25	145, 7	Auto: 101
26	Query Point 26	125, 162.8	Auto: 101
27	Query Point 27	125, 189.1	Auto: 101
28	Query Point 28	165, 189.1	Auto: 101
29	Query Point 29	165, 162.8	Auto: 101
30	Query Point 30	145, 175.95	Auto: 101
31	Query Point 31	173.1, 0	Auto: 101
32	Query Point 32	173.1, 183.1	Auto: 101
33	Query Point 33	201.1, 183.1	Auto: 101
34	Query Point 34	201.1, 0	Auto: 101
35	Query Point 35	187.1, 91.55	Auto: 101
36	Query Point 36	38.215, 98.684	Auto: 89
37	Query Point 37	53.261, 96.899	Auto: 89

Field Point Grid

Number of points 5512
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
286.65	274.65
286.65	-91.85
-85.55	-91.85
-85.55	274.65



Max Pressure 3.5 ksf on all footings. 0.5" or less at foundation centers

Settle3 Analysis Information

Project Settings

Document Name	19-017 CISF CHB 200214.s3z
Date Created	1/16/2020, 3:00:31 PM
Stress Computation Method	Westergaard
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name
1	Stage 1

Results

Time taken to compute: 0 seconds

Stage: Stage 1

Data Type	Minimum	Maximum
Total Settlement [in]	0	0.709451
Loading Stress ZZ [ksf]	0.0346785	4.58

Loads

1. Polygonal Load: "Zone 29"

Label	Zone 29
Load Type	Flexible
Area of Load	7425 ft^2
Load	1.64 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
55	0
55	135
0	135

2. Polygonal Load: "Zone 26"

Label	Zone 26
Load Type	Flexible
Area of Load	675 ft^2
Load	0.61 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
0	0
5	0
5	135
0	135

3. Polygonal Load: "Zone 27"

Label	Zone 27
Load Type	Flexible
Area of Load	250 ft^2
Load	0.65 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
55	130
55	135
5	135
5	130

4. Polygonal Load: "Zone 28"

Label	Zone 28
Load Type	Flexible
Area of Load	650 ft^2
Load	0.64 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
50	0
55	0
55	130
50	130

5. Polygonal Load: "Zone 25"

Label	Zone 25
Load Type	Flexible
Area of Load	225 ft^2
Load	0.7 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5	0
50	0
50	5
5	5

6. Rectangular Load: "Zone 1"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.01 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	5.5
17.5	5.5
17.5	17.5
5.5	17.5

7. Rectangular Load: "Zone 17"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.01 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	5.5
49.5	5.5
49.5	17.5
37.5	17.5

8. Rectangular Load: "Zone 24"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.01 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	117.5
49.5	117.5
49.5	129.5
37.5	129.5

9. Rectangular Load: "Zone 8"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.01 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	117.5
17.5	117.5
17.5	129.5
5.5	129.5

10. Rectangular Load: "Zone 23"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	101.5
49.5	101.5
49.5	113.5
37.5	113.5

11. Rectangular Load: "Zone 22"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.6 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	85.5
49.5	85.5
49.5	97.5
37.5	97.5

12. Rectangular Load: "Zone 21"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.55 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	69.5
49.5	69.5
49.5	81.5
37.5	81.5

13. Rectangular Load: "Zone 20"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.6 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	53.5
49.5	53.5
49.5	65.5
37.5	65.5

14. Rectangular Load: "Zone 19"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.6 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	37.5
49.5	37.5
49.5	49.5
37.5	49.5

15. Rectangular Load: "Zone 18"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
37.5	21.5
49.5	21.5
49.5	33.5
37.5	33.5

16. Rectangular Load: "Zone 16"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.48 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	117.5
33.5	117.5
33.5	129.5
21.5	129.5

17. Rectangular Load: "Zone 7"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	101.5
17.5	101.5
17.5	113.5
5.5	113.5

18. Rectangular Load: "Zone 6"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.6 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	85.5
17.5	85.5
17.5	97.5
5.5	97.5

19. Rectangular Load: "Zone 5"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.55 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	69.5
17.5	69.5
17.5	81.5
5.5	81.5

20. Rectangular Load: "Zone 4"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.55 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	53.5
17.5	53.5
17.5	65.5
5.5	65.5

21. Rectangular Load: "Zone 3"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.6 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	37.5
17.5	37.5
17.5	49.5
5.5	49.5

22. Rectangular Load: "Zone 2"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.51 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
5.5	21.5
17.5	21.5
17.5	33.5
5.5	33.5

23. Rectangular Load: "Zone 9"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.48 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	5.5
33.5	5.5
33.5	17.5
21.5	17.5

24. Rectangular Load: "Zone 10"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.83 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	21.5
33.5	21.5
33.5	33.5
21.5	33.5

25. Rectangular Load: "Zone 11"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.94 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	37.5
33.5	37.5
33.5	49.5
21.5	49.5

26. Rectangular Load: "Zone 12"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.91 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	53.5
33.5	53.5
33.5	65.5
21.5	65.5

27. Rectangular Load: "Zone 13"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.91 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	69.5
33.5	69.5
33.5	81.5
21.5	81.5

28. Rectangular Load: "Zone 14"

Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.94 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	85.5
33.5	85.5
33.5	97.5
21.5	97.5

29. Rectangular Load: "Zone 15"

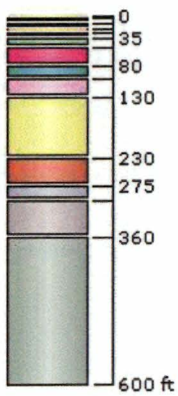
Length	12 ft
Width	12 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	144 ft^2
Load	2.83 ksf
Depth	0 ft
Installation Stage	Stage 1

Coordinates

X [ft]	Y [ft]
21.5	101.5
33.5	101.5
33.5	113.5
21.5	113.5

Soil Layers

Layer #	Type	Thickness [ft]	Depth [ft]
1	Cover Sands	2	0
2	Caliche/Sand 1	8	2
3	Caliche/Sand 2	10	10
4	Caliche Hard 1	5	20
5	Caliche Hard 2	10	25
6	Ogallala 1	15	35
7	Ogallala 2	30	50
8	Ogallala 3	20	80
9	Dockum Claystone/Siltsone	30	100
10	Claystone and Siltstone	100	130
11	Dockum Clay/Claystone 1	45	230
12	Dockum Silty/Sands	25	275
13	Dockum Clay/Claystone 2	60	300
14	Dockum Clay/Claystone 3	240	360



Soil Properties

Property	Cover Sands	Caliche/Sand 1	Caliche/Sand 2	Caliche Hard 1
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.12	0.12
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	890	1200	1200	35815
Esur [ksf]	890	1200	1200	35815
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Caliche Hard 2	Ogallala 1	Ogallala 2	Ogallala 3
Color				
Unit Weight [kips/ft^3]	0.12	0.12	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	55232	80233	53870	123857
Esur [ksf]	55232	80233	53870	123857
Undrained Su A [kips/ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Claystone/ Siltsone	Claystone and Siltstone	Dockum Clay/ Claystone 1	Dockum Silty/ Sands
Color				
Unit Weight [kips/ft^3]	0.13	0.13	0.13	0.13
K0	1	1	1	1
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
Es [ksf]	84172	120769	120769	120769
Esur [ksf]	84172	120769	120769	120769
Undrained Su A [kips/ ft2]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8

Property	Dockum Clay/Claystone 2	Dockum Clay/Claystone 3
Color		
Unit Weight [kips/ft^3]	0.13	0.13
K0	1	1
Immediate Settlement	Enabled	Enabled
Es [ksf]	120769	154394
Esur [ksf]	120769	154394
Undrained Su A [kips/ft2]	0	0
Undrained Su S	0.2	0.2
Undrained Su m	0.8	0.8

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Cask Point 1	11.5, 11.5	Auto: 101
2	Cask Point 2	11.5, 27.5	Auto: 101
3	Cask Point 3	11.5, 43.5	Auto: 101
4	Cask Point 4	11.5, 59.5	Auto: 101
5	Cask Point 5	11.5, 75.5	Auto: 101
6	Cask Point 6	11.5, 91.5	Auto: 101
7	Cask Point 7	11.5, 107.5	Auto: 101
8	Cask Point 8	11.5, 123.5	Auto: 101
9	Cask Point 9	27.5, 11.5	Auto: 101
10	Cask Point 10	27.5, 27.5	Auto: 101
11	Cask Point 11	27.5, 43.5	Auto: 101
12	Cask Point 12	27.5, 59.5	Auto: 101
13	Cask Point 13	27.5, 75.5	Auto: 101
14	Cask Point 14	27.5, 91.5	Auto: 101
15	Cask Point 15	27.5, 107.5	Auto: 101
16	Cask Point 16	27.5, 123.5	Auto: 101
17	Cask Point 17	43.5, 11.5	Auto: 101
18	Cask Point 18	43.5, 27.5	Auto: 101
19	Cask Point 19	43.5, 43.5	Auto: 101
20	Cask Point 20	43.5, 59.5	Auto: 101
21	Cask Point 21	43.5, 75.5	Auto: 101
22	Cask Point 22	43.5, 91.5	Auto: 101
23	Cask Point 23	43.5, 107.5	Auto: 101
24	Cask Point 24	43.5, 123.5	Auto: 101
25	Footing Bottom Left	0, 0	Auto: 101
26	Footing Bottom Right	55, 0	Auto: 101
27	Footing Top Left	0, 135	Auto: 101
28	Footing Top Right	55, 135	Auto: 101
29	Footing Center	27.5, 67.5	Auto: 101
30	Query Point 30	11.973, 99.283	Auto: 101
31	Query Point 31	30, 132.5	Auto: 101
32	Query Point 32	2.5, 67.5	Auto: 101
33	Query Point 33	34.543, 35.669	Auto: 101
34	Query Point 34	27.5, 2.5	Auto: 101

Field Point Grid

Number of points 1672
Expansion Factor 1

Grid Coordinates

X [ft]	Y [ft]
122.5	202.5
122.5	-67.5
-67.5	-67.5
-67.5	202.5

Settle3

Software Validation and Verification

DAN BROWN AND ASSOCIATES

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TECHNICAL MEMORANDUM

Software Verification: Settle3 v5.001
CISF Site
Andrews, TX
DBA Project No. 19-017

To: Derek Kilday, P.E./GEOServices

From: Timothy C. Siegel, P.E, G.E., D.GE
Tayler J. Day, P.E.

Date: 17 February 2020

STATE OF TEXAS
TIMOTHY C. SIEGEL
117477
LICENSED
PROFESSIONAL ENGINEER

1. Introduction

Dan Brown and Associates, P.C. (DBA) was contracted to perform settlement analyses for the subject project. As part of our scope, DBA was to perform and provide verification of the efficacy of the Settle3 v5.001 software used for the analyses. This TM explains the basis of the verification and DBA's conclusion that the software is valid and appropriate for the provided analyses.

2. Selection of Software

DBA routinely performs settlement analyses as a function of our role as industry leaders in foundation and ground improvement design. It is our experience that Settle3 software by Rocscience (currently version 5.001) is an effective tool that uses sound geotechnical and mechanics principles to produce settlement results that can be interpreted by technical and non-technical personnel. Additionally, Settle3 v5.001 (and other Rocscience geotechnical softwares) are routinely used in standard geotechnical engineering practice.

3. Critical Characteristics of Software

Settle3 v5.001 calculates settlement by interpreting the way applied stress is distributed with depth in a soil column using the Westergaard solution. The resulting stress distribution and the user input soil parameters are used to calculate strain/displacement.

The analyses required for the CISF site in Andrews, TX are ultimately concerned with the settlement of a single footing and a group of footings which requires calculations that effectively satisfy the following critical characteristics:

- Calculate stress with depth below the corner of a single rectangular loaded area
- Calculates vertical stress beneath a point within a rectangularly loaded area using the principle of superposition of stresses

- Considers multiple bearing pressure areas and calculates the vertical stress beneath a point outside of a rectangularly loaded area (i.e. between footings) using the principle of superposition.

4. Verification Procedure and Acceptance

DBA established the attached calculations to verify the critical characteristics identified in the previous section using hand calculations based on the methods used by Settle3 v5.001. The acceptance criteria established by DBA for each critical characteristic stress calculation was an error of 1% or less to account for any differences in rounding of trigonometric function results.

The hand calculations utilize a closed form solution of the Westergaard Stress Computation Formula found in Taylor (1948)[1]. The final result of the evaluation of the critical characteristics are summarized in Table 1.

Critical Characteristic	Method of Comparison	Acceptable Error Criteria	Measured Error
Vertical stress beneath corner of rectangularly loaded area	Slide3 v5.001 software vs Hand Calculation (attached)	+/- 1%	0%
Vertical stress beneath a point within rectangularly loaded area	Slide3 v5.001 software vs Hand Calculation (attached)	+/- 1%	0%
Vertical stress beneath a between multiple rectangular areas	Slide3 v5.001 software vs Hand Calculation (attached)	+/- 1%	0%

5. Concluding Remarks

Calculations were performed to verify the efficacy of Settle3 v5.001 for the types of analyses required for the CISF storage pads in Andrews, TX. DBA concludes the software is appropriate and accurate for use on the subject project. Please contact the following if you would like to discuss this document or this project.

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[1] Taylor, D.W. (1948). *Fundamentals of Soil Mechanics.* John Wiley, New York. Pg 259.

Attachments

Settle3 v5.001 Verification Analysis

DAN BROWN AND ASSOCIATES

Project WCS Storage Facility
Project No. 19-017
Sheet 1 of 9
Date 01 / 14 / 2020
Engineer WPS
Checked by TJD

Settle3 by Rocscience Stress Calculation Verification

This calculation set is for the verification of the Westergaard stress computation method in Settle3 Version 5.001 by Rocscience. The Westergaard stress computation method is a common stress computation method for layered strata. The raw form of the Westergaard stress computation method is developed to calculate the vertical stress beneath a point load. This raw form of Westergaard's stress computation method is analytically integrated to calculate the vertical stress beneath the corner of a rectangularly loaded area, and using the principle of superposition, the vertical stress beneath any point within or beyond the loaded area can be calculated. So, the critical characterisitcs to be checked are as follows: 1) Settle3 correctly calculates the vertical stress beneath the corner of a rectangularly loaded area, 2) Settle3 correctly calculates the vertical stress beneath a point within a rectangularly loaded area using the principle of superposition, and 3) Settle3 correctly considers multiple bearing pressures and calculates the vertical stress beneath a point outside of a rectangularly loaded area using the principle of superposition.

Westergaard Stress Computation Formula:

$$\sigma_{z\_Q} := \left(\frac{Q}{z^2}\right)\cdot\frac{\left(\frac{1}{2\pi}\right)\cdot\sqrt{\frac{1-2\cdot\upsilon}{2-2\upsilon}}}{\left[\left(\frac{1-2\upsilon}{2-2\upsilon}\right)+\left(\frac{r}{z}\right)^2\right]^{\frac{3}{2}}}$$

Where:

σ_z = Vertical Stress

Q = Point Load

z = Depth

υ = Poisson's Ratio

r = Horizontal Distance from Point Load

The Westergaard Stress Computation Formula shown above is listed for a point load, however, it is more common to calculate stress for a bearing pressure. The vertical stress beneath the corner of a given rectangular bearing pressure or footing can be obtained by integrating the above equation and the resulting equation is shown below.

Westergaard Stress Computation Formula:

$$\sigma_z := \left(\frac{q}{2\cdot\pi}\right)\cdot\operatorname{acot}\left[\sqrt{\left(\frac{1-2\nu}{2-2\upsilon}\right)\cdot\left(\frac{1}{m^2}+\frac{1}{n^2}\right)+\left(\frac{1-2\cdot\nu}{2-2\cdot\nu}\right)^2\cdot\left(\frac{1}{m^2\cdot n^2}\right)}\right]$$

Where:

σ_z = Vertical Stress

q = Bearing Pressure

υ = Poisson's Ratio

m = L/z

n = W/z

z = Depth

L = Length of Footing or Bearing Pressure Considered

W = Width of Footing or Bearing Pressure Considered

Source: Taylor, D. W. (1948). *Fundamentals of Soil Mechanics.* (pg. 259). John Wiley. New York.

The above equation is formulated to calculate the vertical stress beneath the corner of a rectangular bearing pressure or footing. So to calculate the stress at the center of a rectangular bearing pressure or footing, the rectangular area can be segmented into four equal areas and using the principle of superposition, the stress can be calculated at the center. The following two pages show calculations for stress beneath the corner and the center of a square footing.

Calculation of the Vertical Stress Beneath the Corner of a Square Footing:

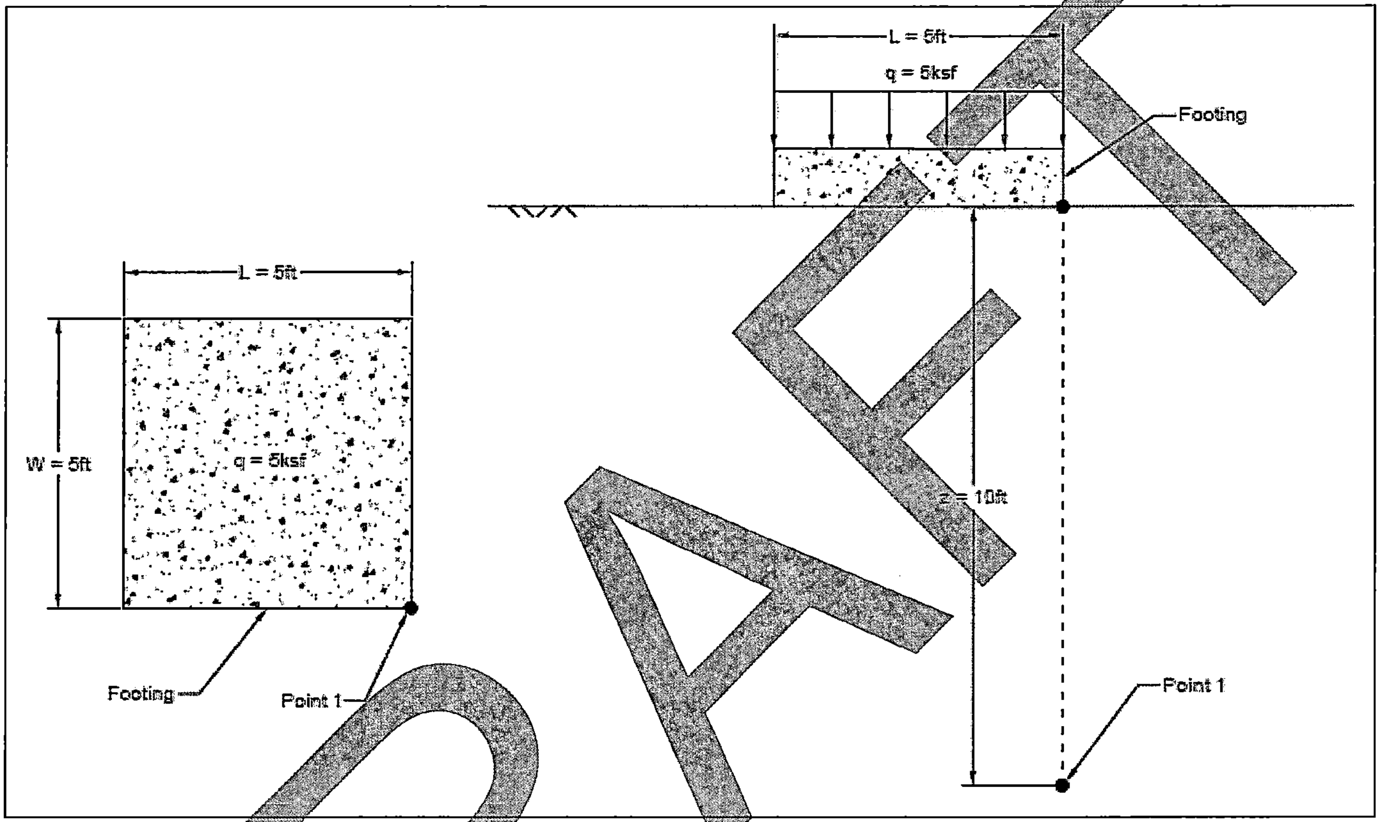

Figure 1 - Stress Beneath Corner of Square Footing Diagram (Left - Plan View, Right - Cross Section)

Footing Length: $L_1 := 5ft$

Footing Width: $W_1 := 5ft$

Bearing Pressure: $q_1 := 5ksf$

Poisson's Ratio: $\upsilon := 0.33$

Depth of Interest: $z_1 := 10ft$

Length-to-Depth Ratio: $m_1 := \frac{L_1}{z_1} = 0.5$

Width-to-Depth Ratio: $n_1 := \frac{W_1}{z_1} = 0.5$

$$\sigma_{z1} := \left(\frac{q_1}{2\cdot\pi}\right)\cdot \text{acot}\left[\sqrt{\left(\frac{1-2\cdot\upsilon}{2-2\upsilon}\right)\cdot\left(\frac{1}{m_1^{\ 2}}+\frac{1}{n_1^{\ 2}}\right)+\left(\frac{1-2\cdot\upsilon}{2-2\cdot\upsilon}\right)^2\cdot\left(\frac{1}{m_1^{\ 2}\cdot n_1^{\ 2}}\right)}\right] = 413\cdot psf$$

Calculation of the Vertical Stress Beneath the Center of a Square Footing:

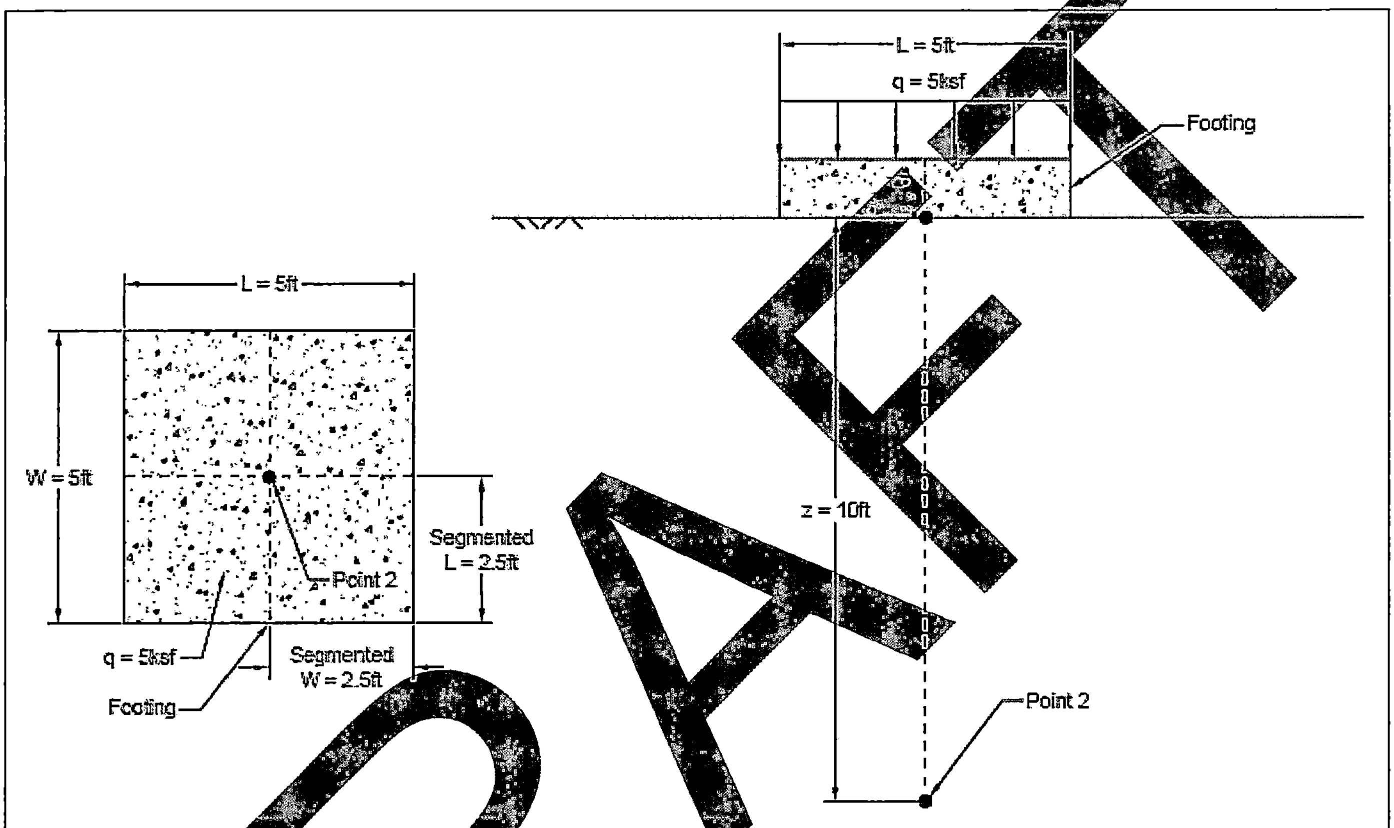

Figure 2 - Stress Beneath Center of Square Footing Diagram (Left - Plan View, Right - Cross Section)

Segmented Footing Length: $L_2 := 2.5\text{ft}$

Segmented Footing Width: $W_2 := 2.5\text{ft}$

Bearing Pressure: $q_2 := 5\text{ksf}$

Poisson's Ratio: $\upsilon := 0.33$

Depth of Interest: $z_2 := 10\text{ft}$

Length-to-Depth Ratio: $m_2 := \frac{L_2}{z_2} = 0.25$

Width-to-Depth Ratio: $n_2 := \frac{W_2}{z_2} = 0.25$

$$\sigma_{z2} := 4\left(\frac{q_2}{2\cdot\pi}\right)\cdot \text{acot}\left[\sqrt{\left(\frac{1-2\cdot\upsilon}{2-2\upsilon}\right)\cdot\left(\frac{1}{m_2^{\,2}}+\frac{1}{n_2^{\,2}}\right)+\left(\frac{1-2\cdot\upsilon}{2-2\cdot\upsilon}\right)^2\cdot\left(\frac{1}{m_2^{\,2}\cdot n_2^{\,2}}\right)}\right] = 633\cdot\text{psf}$$

Note: Using the principle of superposition, the equation for the stress beneath the corner of a rectangularly loaded area is multiplied by 4 in this case account for all 4, 2.5ft x 2.5ft loaded areas.

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Calculation of the Vertical Stress Beneath a Point Between Two Rectangular Footings:

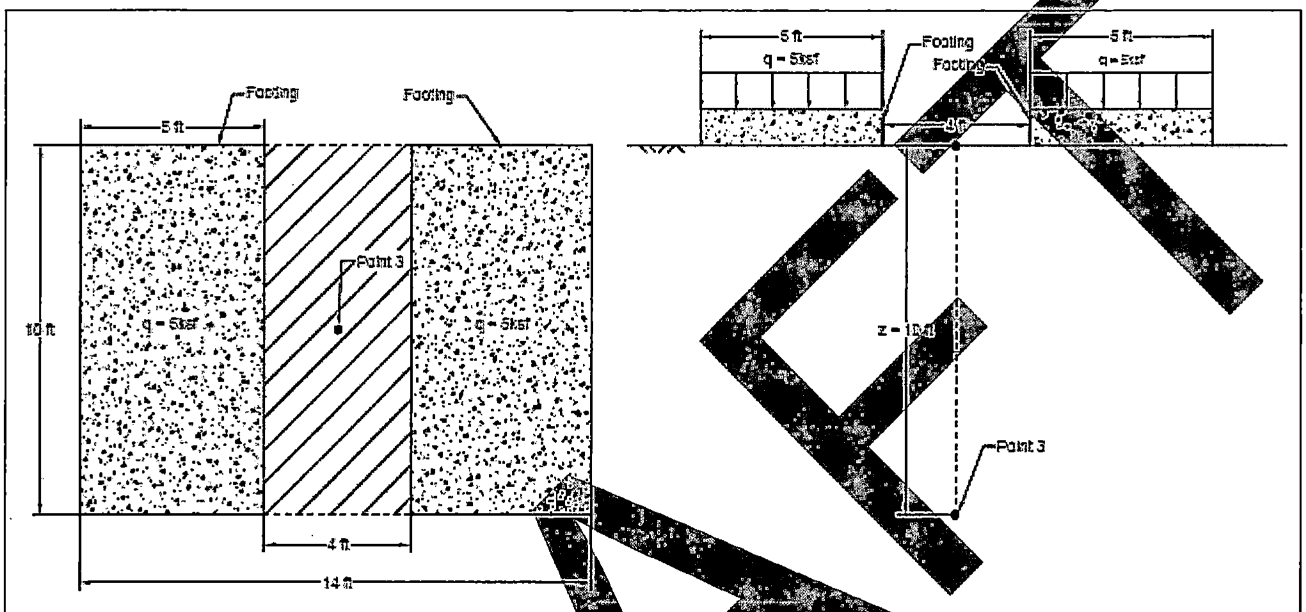

Figure 3 - Vertical Stress Beneath a Point Between Two Rectangular Footings Diagram
(Left - Plan View, Right - Cross Section)

In this case, the principle of superposition is used to add the stress from 4 rectangles with a common point at Point 3 and then subtract out the stress from the areas that are not loaded. See Figure 4 and commentary below for details.

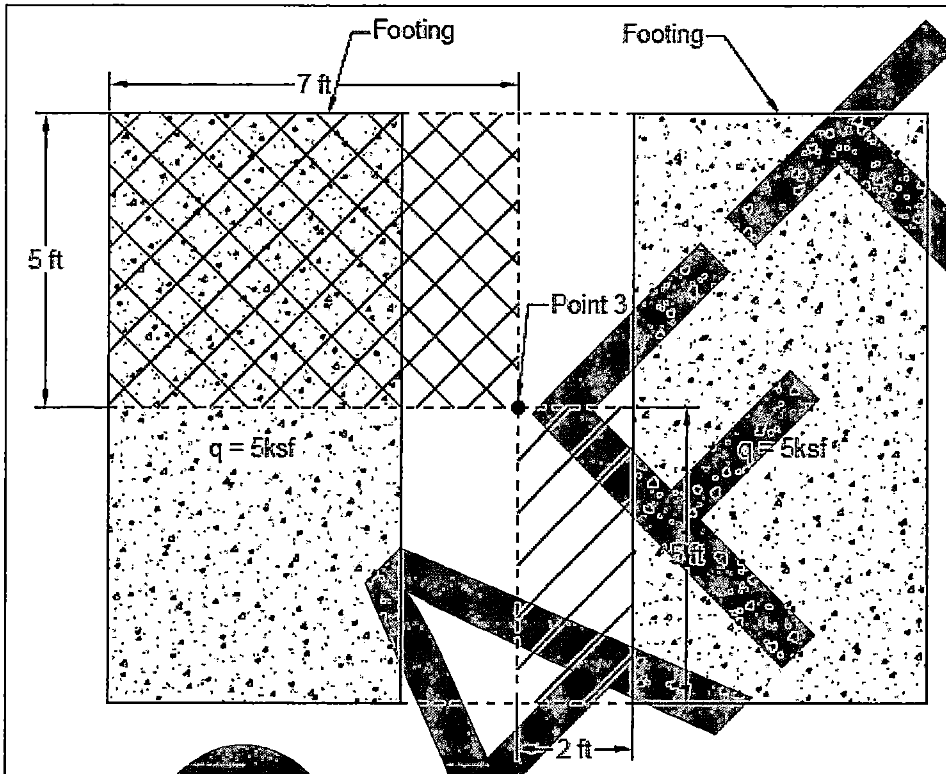

Figure 4 - Vertical Stress Beneath a Point Between Two Rectangular Footings Diagram. The Stress Computation Areas are Hatched and Cross-Hatched (Plan View)

To calculate the stress beneath Point 3, consider the entire cross-hatched area is loaded with a bearing pressure of 5ksf - just like the footings. Four times the vertical stress resulting from Westergaard's equation for the cross-hatched area will be equal to the vertical stress if the whole area, even the area between the two footings, is loaded with a bearing pressure of 5ksf. But the whole area is not loaded; the area between the footings has no load. So using the principle of superposition and Westergaard's equation for the vertical stress beneath the corner of a rectangularly loaded area, four times the vertical stress from the hatched area is subtracted from four times the vertical stress from the cross-hatched area to get the stress at Point 3. Calculations are shown below.

Vertical Stress at Point 3: Cross-Hatched Area

Segmented Footing Length: $L_{3a} := 7ft$

Segmented Footing Width: $W_{3a} := 5ft$

Bearing Pressure: $q_3 := 5ksf$

Poisson's Ratio: $\upsilon := 0.33$

Depth of Interest: $z_3 := 10ft$

Length-to-Depth Ratio: $m_{3a} := \frac{L_{3a}}{z_3} = 0.7$

Width-to-Depth Ratio: $n_{3a} := \frac{W_{3a}}{z_3} = 0.5$

$$\sigma_{z3a} := 4\left(\frac{q_3}{2\cdot\pi}\right)\cdot \text{acot}\left[\sqrt{\left(\frac{1-2\cdot\upsilon}{2-2\upsilon}\right)\cdot\left(\frac{1}{m_{3a}^{\ 2}}+\frac{1}{n_{3a}^{\ 2}}\right)+\left(\frac{1-2\cdot\upsilon}{2-2\cdot\upsilon}\right)^2\cdot\left(\frac{1}{m_{3a}^{\ 2}\cdot n_{3a}^{\ 2}}\right)}\right] = 1938\cdot psf$$

Vertical Stress at Point 3: Hatched Area

Segmented Footing Length: $L_{3b} := 2\text{ft}$

Segmented Footing Width: $W_{3b} := 5\text{ft}$

Bearing Pressure: $q_3 := 5\text{ksf}$

Poisson's Ratio: $\upsilon := 0.33$

Depth of Interest: $z_3 := 10\text{ft}$

Length-to-Depth Ratio: $m_{3b} := \dfrac{L_{3b}}{z_3} = 0.2$

Width-to-Depth Ratio: $n_{3b} := \dfrac{W_{3b}}{z_3} = 0.5$

$$\sigma_{z3b} := 4\left(\frac{q_3}{2\cdot\pi}\right)\cdot \text{acot}\left[\sqrt{\left(\frac{1-2\cdot\upsilon}{2-2\upsilon}\right)\cdot\left(\frac{1}{m_{3b}^{\ 2}}+\frac{1}{n_{3b}^{\ 2}}\right)+\left(\frac{1-2\cdot\upsilon}{2-2\cdot\upsilon}\right)^2\cdot\left(\frac{1}{m_{3b}^{\ 2}\cdot n_{3b}^{\ 2}}\right)}\right] = 837\cdot\text{psf}$$

Vertical Stress at Point 3 - Subtract Hatched Area Vertical Stress from Cross-Hatched Area Vertical Stress:

Vertical Stress at Point 3: $\sigma_{z3} := \sigma_{z3a} - \sigma_{z3b} = 1100\cdot\text{psf}$

Plots of Vertical Stress vs Depth Comparing Hand Calculations to Settle3 Output:

The above calculations using Westergaard's equation for the vertical stress beneath the corner of a 5ft x 5ft footing, the center of a 5ft x 5ft footing, and a point between two 10ft x 5ft footings, where all footings are loaded with a 5ksf bearing pressure, were carried out in Microsoft Excel from 0 to 100ft depth. This data is plotted in Figures 5, 6, and 7 and compared with the output from the same problems modelled in *Settle3 Version 5.001 by Rocscience*. As shown in Figures 5, 6, and 7, the results from the hand calculations and Settle3 are identical, thus, the critical characteristics of Settle3 are confirmed.

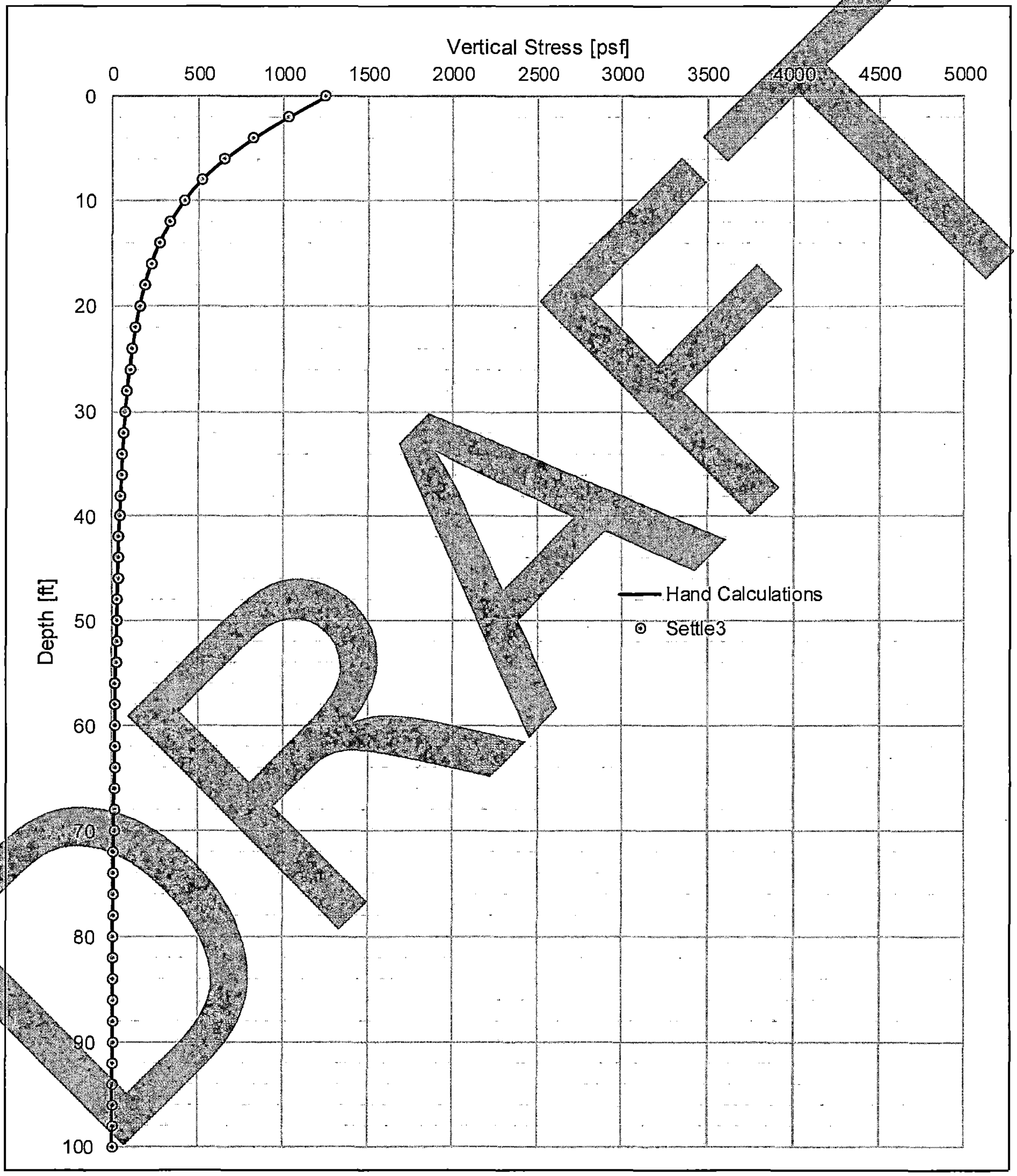

Figure 5 - Vertical Stress Beneath the Corner of a 5ft x 5ft Footing with a Bearing Pressure of 5ksf
Note: The results from hand calculations and Settle3 are compared and shown to be the same

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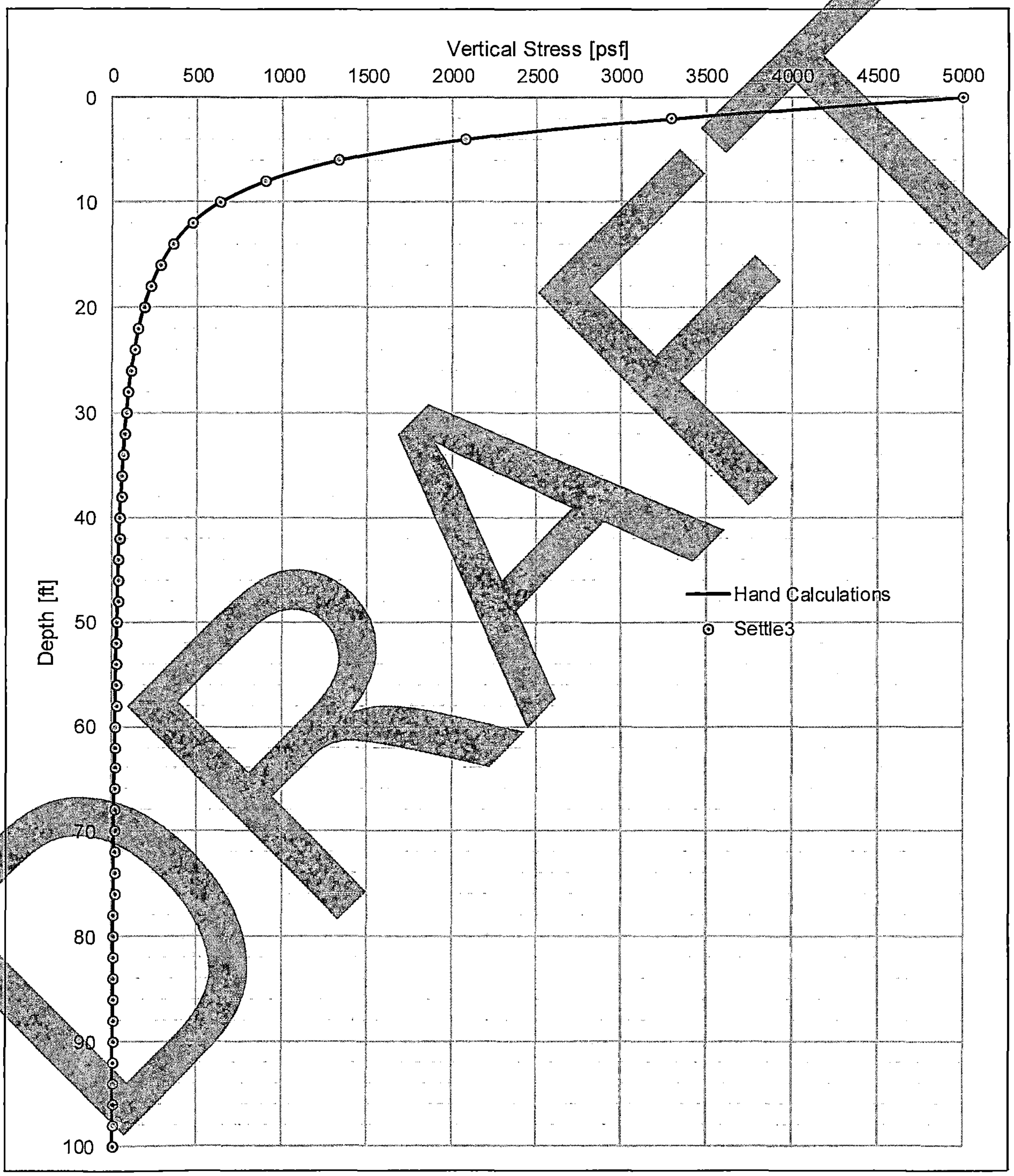

Figure 6 - Vertical Stress Beneath the Center of a 5ft x 5ft Footing with a Bearing Pressure of 5ksf
Note: The results from hand calculations and Settle3 are compared and shown to be the same

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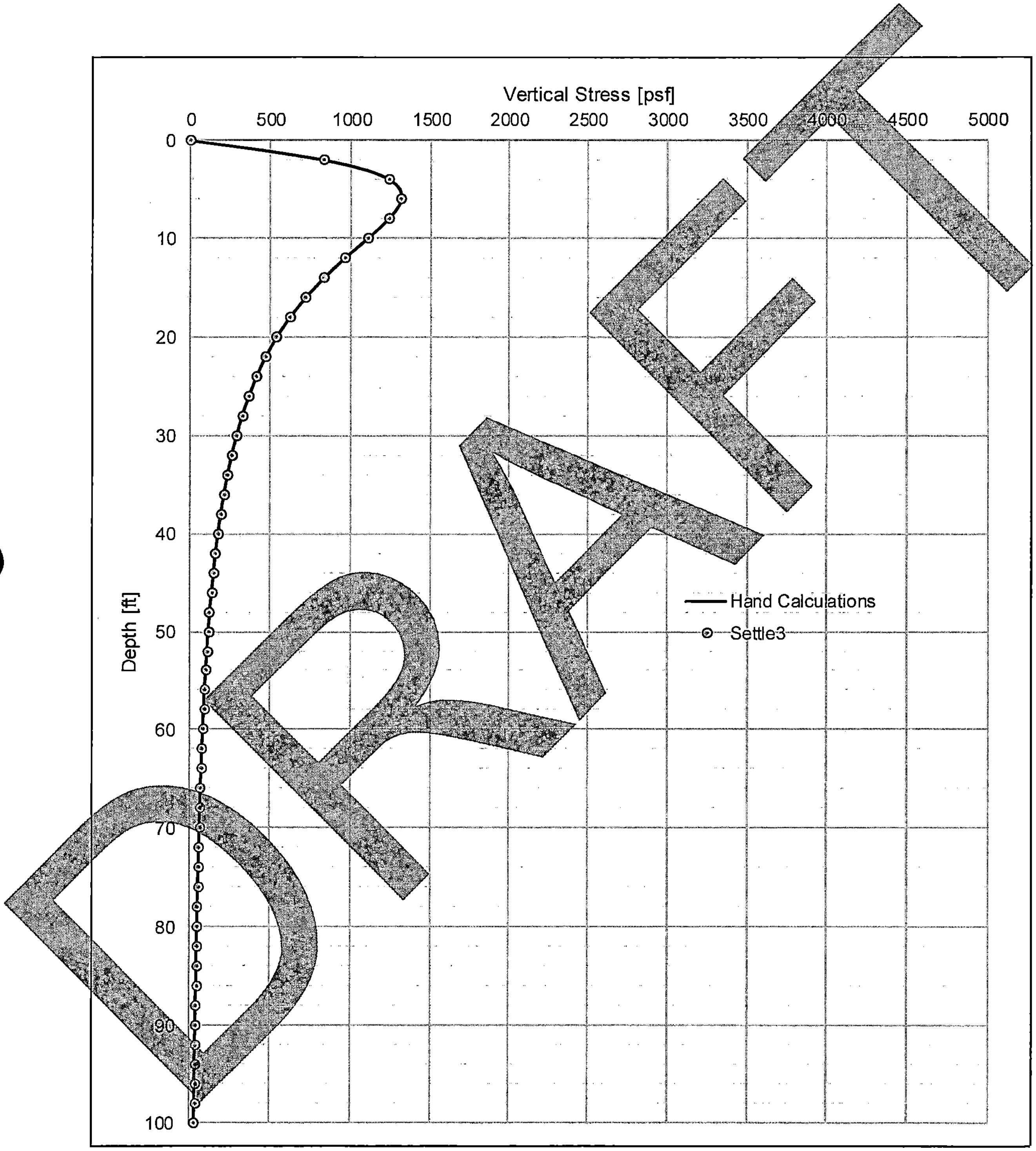

Figure 7 - Vertical Stress Beneath a Point Between Two, 10ft x 5ft Footings with a Bearing Pressure of 5ksf
Note: The results from hand calculations and Settle3 are compared and shown to be the same

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
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 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.

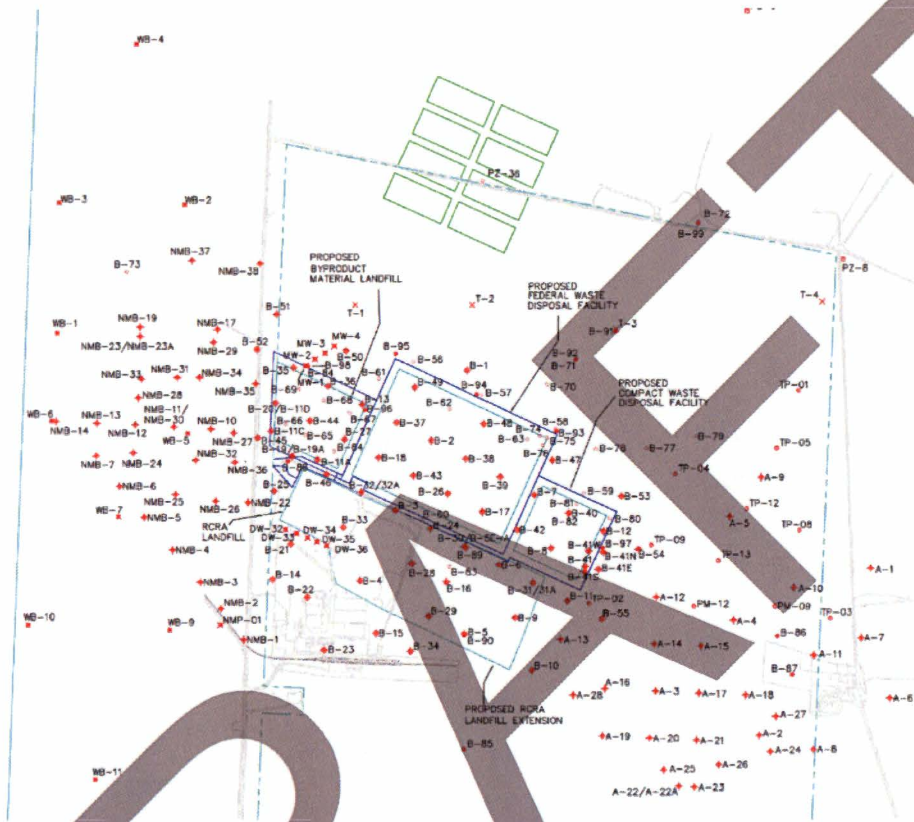

Figure NP-2.6-4-1
Historical Borings at WCS Site

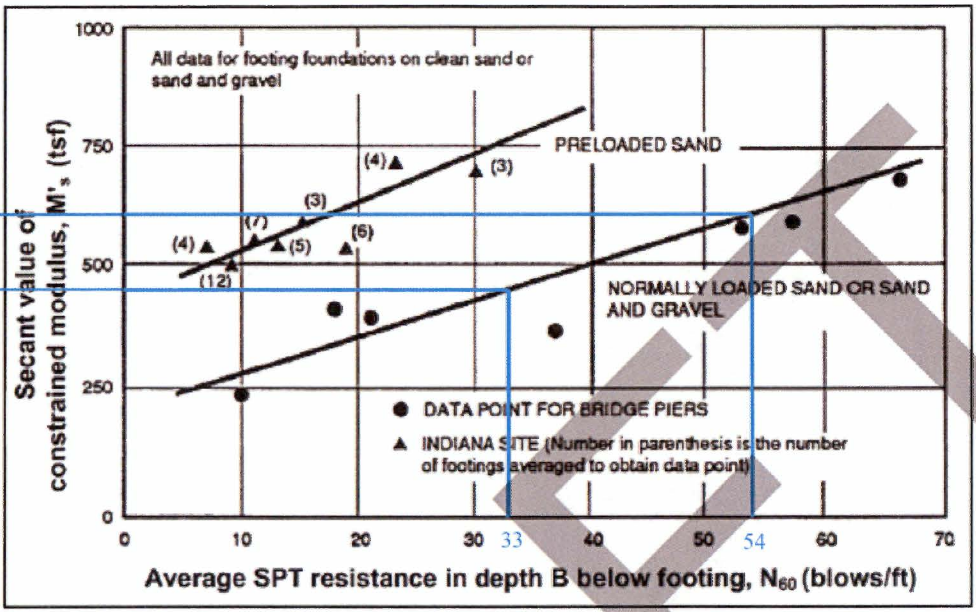

Figure NP-2.6-4-2
Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]

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.

<u>Methodology 1:</u>

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.

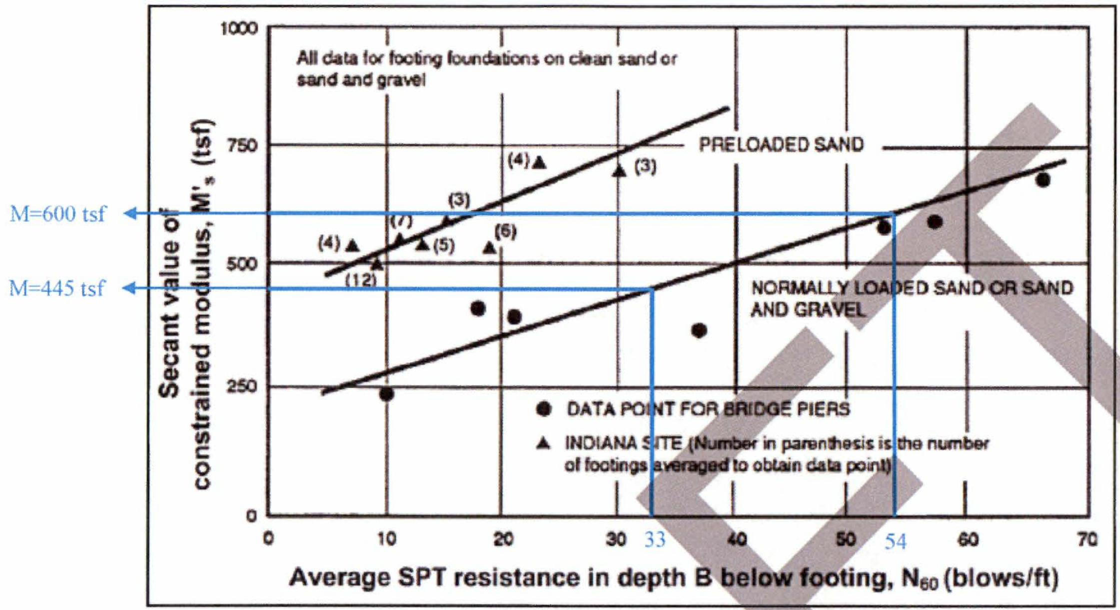

Figure NP-2.6-5-1
Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]

Methodology 2:

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
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 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:

WCS CISF SAR Figure 4-4 has been updated to show the VCT that has the full range of motion required to offload transportation casks from the railcar and upright them under the CTS. The VCT has also been enhanced to address the following functional requirements:

1. VCT deck height allows driving over a flat-bed railcar with deck heights up to a 42 inches.

2. VCT can span flat-bed railcar bed widths of up to 10 feet

3. Tower height can readily address the transport cask vertical height to clear rear rotation trunnions.

4. Added dual towers to provide better control of the transition of the transport cask to the vertical position.

Impact:

SAR Figure 4-4 has been revised as described in the response.

Proprietary Information on Pages 4-48 through 4-50
Withheld Pursuant to 10 CFR 2.390

All Indicated Changes are in response to RAI NP-4-4

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.

3.2.3.3 Design Response Spectra Derivation

The seismic analysis for the CISF swas performed to be consistent with 10 CFR 72.103 [3-23], U.S. Nuclear Regulatory Commission's NUREG- 0800 "Standard Review Plan (SRP) for the Review of Safety Analyses Reports for Nuclear Power Plants: LWR Edition" [3-3] and NUREG/CR-6728 "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines" [3-25].

3.2.3.4 Design Time History

Consistent with NRC requirements, horizontal and vertical DRS for a 10,000 year return period and associated strain-compatible properties were developed and provided for the SSI analysis. Three three-component sets of time histories were developed through spectral matching. A final report was produced that describes and summarizes the above analyses in Chapter 2, Attachment D. All calculations were performed in accordance with AECOM's NQA-1 Program. Detailed calculations are contained in calculation WCS-12-05-200-001 in Chapter 2 Attachment D.

Design time histories are used to verify all required components are considered acceptable. Chapter 7 includes further details.

3.2.3.5 Use of Equivalent Static Loads

Chapter 7 of the SAR details the load analyses used in the seismic design and analysis.

RAI NP-7-3

For the Vertical Storage Systems storage pad *and the NUHOMS® NITS storage pad*, the soil material properties used are the static properties, equal to or lower than the dynamic soil properties and, therefore, conservative for use in an equivalent static analysis. The soil properties used in the equivalent static *analyses* for the Vertical Storage System storage pads *and the NUHOMS® NITS storage pads* are given in Appendix *D* of [3-33] and are listed in Table 7-38.

The design criteria used for the Canister Transfer System (CTS) is specified in ASME NOG-1, Section 4000 [*3-34*]. All of the load combinations identified in paragraph 4140 have been evaluated. Controlling load combinations have been used to determine component stresses and then are compared to applicable allowable stresses. The sum of simultaneously applied loads (static and dynamic) do not result in stress levels which would cause any permanent deformation, and thus, the CTS fully meets the requirements of ASME NOG-1 [*3-34*].

CHB structural steel components are analyzed and designed *using static* analysis methods for determining forces and moments on structural steel members as a result of applied service loading conditions. Dynamic analysis methods are used for determining structural steel member forces and moments for factored loading conditions where structural components are subjected to seismic loads.

7.6 Other Structures, Systems, and Components Subject to NRC Approval

This section describes the structural design, design criteria and design analysis for the storage pads for the NUHOMS® and NAC Systems.

7.6.1 Storage Pads for VCCs

The WCS CISF storage pads are conventional cast-in-place reinforced concrete mat foundation structures. They provide a level and stable surface for placement and storage of VCCs. The pads are designed for normal operating loads, severe environmental loads and extreme environmental loads as referenced by NUREG-1567 [7-28]. The storage pads for the NAC VCCs are designed as ITS structures as described below.

The purpose of this evaluation is to structurally qualify the WCS CISF Storage Pad designs for the vertical systems. The licensing-basis WCS CISF VCC configuration is a 3x8 array of MAGNASTOR casks, which envelopes the other NAC International casks to be stored at the WCS CISF. The qualification is conducted in accordance with the NUREG-0800 [7-43], NUREG-1536 [7-42] and NUREG-1567 [7-28]. A geotechnical liquefaction *analysis* and elastic settlement analysis is performed as part of *the Geotechnical Exploration Report [7-32] and* Calculation NAC004-CALC-02 [7-48].

7.6.1.1 Design Inputs

Material Properties

[ ]

Soil Properties

Modulus of subgrade reaction, K_s

The modulus of subgrade reaction is calculated based on an iterative process conducted between the geotechnical model of the substrate underneath the pad and the structural model of the concrete ISFSI pad. Further details regarding the calculation of the modulus of subgrade reaction below the different areas of the ISFSI pad are documented in Section 4.3.3 and Appendix H of Reference [7-32].

Foundation friction coefficient (between concrete and bearing soils),
$\mu = 0.35$ (Ref. [7-32], Section 4.4)

All Indicated Changes are in response to RAI NP-7-3

Subsurface water was not observed at the WCS CISF. (Ref. [7-32], Section 3.2.2)

Design Loads

Dead Load:

The design dead loads are the weight of the concrete.

Live Load:

Live load includes the weight of loaded VCCs *and VCT*, operational loads of handling equipment and occupancy load *as listed below*.

[ ]

Snow Load:

Snow load is considered to be enveloped by the live load to simplify the load combinations. Per ASCE 7- 05 (Ref. [7-34], Figure 7-1) the ground snow load is 10 psf at the WCS CISF, which is small relative to the live load.

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[ ]

Seismic Inertia Load

The seismic accelerations for the 10,000-year return period earthquake response spectrum were developed as part of the site-specific seismic hazard evaluation in reference [7-33]. The site-specific seismic hazard evaluation presents the safe-shutdown earthquake (SSE) as the 10,000-year period earthquake. The site-specific seismic hazard evaluation [7-33] was used as an input to the subsequent soil-structure interaction (SSI).

[ ]

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7.6.1.2 Design Basis

The design of WCS CISF is based on NUREG-1567 [7-28] with reference to NUREG-1536 [7-42] and NUREG-0800 [7-43]. Guidance from NUREG-1567 is utilized for this design. Codes of record and regulatory guides referenced in NUREG-1567 are utilized throughout the design. The code and regulatory guide years/revisions are based on the reference year for IBC 2009 [7-45] (building code for Texas) and the newest revision of the regulatory guides. The codes of record and regulatory guides used for the design, where applicable, are as follows:

- ACI 318-08 [7-39]
- ACI 349-06 (latest revision endorsed by the NRC) [7-31]
- ASCE 7-05 [7-34]
- ASCE 43-05 [7-44]
- Regulatory Guide 1.61, Rev. 1 [7-38]
- Regulatory Guide 1.76, Rev. 1 [7-35]

All Indicated Changes are in response to RAI NP-7-3

7.6.1.3 Design Load Considerations

Thermal Load

Thermal loading of the storage pad is not considered in detail given that the heat transferred to the storage pad is very small and is only in relatively small localized areas. Furthermore, the local cask concrete elevated temperature, which occurs only near the cask top, is less than the ACI 349-06 (Ref. [7-31], Section E.4) accident temperature limits of 350 °F for the concrete surface.

Cask Drop Load

The cask drop accident has been considered with respect to the structural integrity of the cask as part of the MAGNASTOR FSAR [7-40]. The cask drop impact to the storage pad is not considered here because such an accident would result in localized damage to the storage pad, but not result in a loss of stability of the storage pad. In the case of such an accident, the storage pad would need to be evaluated and repaired as needed.

Tornado-Missile Impact Load

Tornado-missile impact load has been considered with respect to the structural integrity of the cask as part of the MAGNASTOR FSAR [7-40]. Tornado-missile impact to the directly to the storage pad or to a cask on the storage pad is not considered here because such an extreme condition would result in localized damage to the storage pad, but not result in a loss of stability of the storage pad. In the case of such an accident, the storage pad would need to be evaluated and repaired as needed. This is consistent with NUREG-1536 (Ref. [7-42], Table 3-3), which states for the tornado load case "[t]he load combination (capacity/demand >1.00 for all sections) shall be satisfied without missile loadings. Missile loadings are additive (concurrent) to the loads caused by wind pressure and other loads; however, local damage may be permitted at the point of impact if there is no loss of intended function of any structure important to safety."

Seismic Inertia Loading

The seismic load case includes various cask layouts, but not does not consider short-term configurations (e.g., VCT in operation). All three directions of seismic excitation are conservatively considered simultaneously.

Wind Load on Transporter:

The operational wind load of 25.34 kip is determined in Section 7.6.1.1. Because the transporter is not included in the GTSTRUDL model, the loads are applied directly as nodal joint forces representing the self-weight. The operational wind load creates an overturning moment on the VCT that is resolved into a vertical force couple.

All Indicated Changes are in response to RAI NP-7-3

The load from the operating wind load on the VCT is very small when compared to load due to self-weight of VCT. Instead of creating additional load combinations for operating wind load on the VCT, the wind load is added to vertical load self-weight which equals

= 1.33 kip/in + 0.18 kip/in = 1.51 kip/in. Use 1.55 kip/in.

7.6.1.4 Load Combinations

Per NUREG-1567 (Ref. [7-28], Section 5.4.3.4), load combinations for reinforced concrete structures including Independent Spent Fuel Storage Installations (ISFSIs) are per NUREG-1536 (Ref. [7-42], Table 3-3) and ACI 349 [7-31]. Load combinations from the two sources are presented only with applicable load cases. Note that ACI 318-08 [7-39] are enveloped by ACI 349 [7-31] load combinations. Thermal, piping, pipe break, soil, snow and flooding load cases are not included for clarity. Vertical cask transporter loads are considered as live loads as opposed to crane loads.

ACI 349-06 Load Combinations

$U = 1.4D$ (Ref. [7-31], Eq. 9-1)

$U = 1.2D + 1.6L$ (Ref. [7-31], Eq. 9-2)

$U = 1.2D + 0.8L$ (Ref. [7-31], Eq. 9-3)

$U = 1.2D + 1.6(L + E_O)$ (Ref. [7-31], Eq. 9-4)

$U = 1.2D + 1.6(L + W)$ (Ref. [7-31], Eq. 9-5)

$U = D + 0.8L + E_{SS}$ (Ref. [7-31], Eq. 9-6)

$U = D + 0.8L + W_t$ (Ref. [7-31], Eq. 9-7)

$U = D + 0.8L$ (Ref. [7-31], Eq. 9-8)

$U = D + 0.8L + E_{SS}$ (Ref. [7-31], Eq. 9-9)

*Note: All dead loads shall be considered at 0.9 where dead load reduces the effects of other loads. Similarly, live load shall be considered zero where live load reduces the effects of other loads.

Section 9.2.2 of ACI 349-06 [7-31] states that "Where the structural effects of differential settlement, creep, shrinkage, or expansion of shrinkage-compensating concrete are significant, they shall be included with the dead load in Eq. (9-4) through (9-9)."

The maximum settlements at the different sections of the storage pad have been calculated in the revised Geotechnical Exploration Report [7-32]. The maximum differential settlement based on the settlements reported in Appendix H of reference [7-32] is calculated to be 0.70 inches. Per Table 5-7 of reference [7-54], the recommended maximum limit for the differential settlement of buildings with rafts type footings (similar to the concrete pad under consideration) on clayey soils and sandy soils is 35 mm (1.38 in.) and 25 mm (0.98 in.), respectively.

All Indicated Changes are in response to RAI NP-7-3

Since the calculated maximum differential settlements are significantly lower than the permissible limits as defined in Table 5-7 of reference [7-54], it is considered that the additional loads on the concrete due to the differential settlements of the pad would be negligible when compared with the other relatively large design loads, including various dead and live loads. Consequently, the loads due to the differential settlements have not been considered together with the dead load in the load cases listed above.

NUREG-1536 Load Combinations

$U = 1.4D + 1.7L$ (Ref. [7-42], Table 3-3)

$U = 1.05D + 1.275L$ (Ref. [7-42], Table 3-3)

$U = 1.05D + 1.275(L + W)$ $U = D + L + E_{SS}$ (Ref. [7-42], Table 3-3)

$U = D + L + W_t$ (Ref. [7-42], Table 3-3)

*Note: All dead loads shall be varied by 5% where dead load reduces the effects of other loads.

NUREG-1536 Stability Load Combinations (Overturning and Sliding)

$O/S \geq 1.5D$ (Ref. [7-42], Table 3-3)

$O/S \geq 1.1(D + E_{SS})$ (Ref. [7-42], Table 3-3)

$O/S \geq 1.1(D + W_t)$ (Ref. [7-42], Table 3-3)

Governing Load Combinations

Governing load combinations are compiled based on code load combinations, considerations for reduced dead and live load effects, and directions of seismic excitation. Furthermore, SSE seismic load is shown to envelope the tornado wind load (Section 7.6.1.6); therefore, tornado wind load combinations are not included. Because the operational wind load is applied to the transporter, but the seismic load is not considered for the transporter, the operational wind load case is included.

Strength

$U \geq 1.4D + 1.7L$

$U \geq 1.2D + 1.6(L \pm E_O)$

$U \geq 0.9D + 0.9\ L^{**} + 1.6(\pm E_O)$

$U \geq 1.2D + 1.6(L \pm W)$

All Indicated Changes are in response to RAI NP-7-3

$U \geq 0.9D + 0.9\ L^{**} + 1.6(\pm W)$

$U \geq D + L \pm E_{SS}$

$U \geq 0.9D \pm E_{SS}$

$U \geq 1.4D + 1.7L^{*}$

Notes:

L* includes the weight of the loaded vertical cask transporter L** includes the weight of the casks, but not occupancy live load.

Stability

$S/1.5 \geq D + L^{**}$

$S/1.1 \geq D + L^{**} \pm E_{SS}$

Notes:

L** includes the weight of the casks, but not occupancy live load

7.6.1.5 Cask Layout Configurations

During the life of the storage pad, several different configurations and numbers of casks are possible. The analysis has been performed on four representative enveloping configurations. The configurations are based on initially loading one of the short sides of the storage pad and then adding casks systematically across the pad. There are also several permutations of VCT locations (one VCT on pad considered per VCT load combination). The considered cask and VCT configurations are presented in Figure 7-9. The VCT locations are shown as the transporter tread locations. Note that for the VCT load combinations, the cask is considered to be supported by the VCT.

GTSTRUDL Modeling

The static GTSTRUDL model utilizes a six-degree-of-freedom plate bending and stretching element (SBHQ6) to represent the concrete pad. The slab stiffness is reduced to account for cracking (Concrete Pad Stiffness Properties). The concrete pad is supported on nonlinear (compression only) soil springs.

The casks are modeled with rigid frame members originating at the 8 contact points (between top of concrete and bottom of cask, spaced at 45 degrees around the cask outer perimeter) and terminating at the cask center of gravity (CG). The weight of the cask and the seismic forces are applied at the CG, located at the apex of the frame members. The lower ends of the rigid frame members are pin connected to the top of mat; thus, they do not transfer bending moments, but sliding and uplifting of the casks is inhibited. The cross-sectional area and moments of inertia of the frame members are large, resulting in high axial, torsional, and flexural stiffness, in order to simulate the rigid behavior of the cask. Since sliding would reduce the magnitude of the forces transmitted to the pad, this approach is conservative. Furthermore, the cask is not released vertically from its foundation as cask overturning is unlikely. The cask overturning stability evaluation is performed as part of the SSI analysis in Reference [7-26] and provides further evidence that cask overturning will not occur under the governing design loads. Since the pad is modeled by plate elements, the horizontal membrane (in- plane) stiffness added to the slab by the frame members acts at the centerline of the plate and does not affect its bending stiffness. However, the frame members do add vertical stiffness to the mat. It is estimated that this increased vertical stiffness reduces the vertical deflection of the mat, but the bending moments and shears are aggravated due to the increased mat curvature between adjacent casks and the more severe application of cask seismic moments (as opposed to the cask just "sitting" on the pad, unable to exert uplift forces). Overall, these modeling features produce conservative moments and shears in the pad. Element body forces are used to represent the self-weight of the concrete. Element surface loads are used to represent live loads on the pad. Joint forces are used for the cask and VCT loads.

Concrete Pad Stiffness Properties

From ACI 349-06 (Ref. [7-31], Section 8.6.1) and ACI 318-08 (Ref. [7-39], Section 8.7.1), "[u]se of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffness..." To approximate the effective stiffness of the storage pad, effective stiffness for reinforced concrete members provided in ASCE 43-05 (Ref. [7-44], Table 3-1) are used conservatively considering "[w]alls and diaphragms – cracked" because slabs- on-grade are not addressed. The effective stiffness' are as follows:

Effective flexural rigidity: $0.5E_cI_g$ (Ref. [7-44], Table 3-1)

Effective shear rigidity: $0.5G_cA_w$ (Ref. [7-44], Table 3-1)

Shear modulus, $G_e = 0.4E_C = 0.4 \cdot 3{,}605$ ksi = 1,442 ksi (Ref. [7-44], Table 3-1)

To represent the effective flexural rigidity and shear rigidity, 50 percent of the elastic modulus and shear modulus are used in the GTSTRUDL model and the actual 36-in thickness is used.

Effective elastic modulus, $E_{e,eff}$ = 1,803 ksi

Effective shear modulus, $G_{e,eff}$ = 721 ksi

All Indicated Changes are in response to RAI NP-7-3

Nonlinear Soil Springs

Nonlinear (compression only) springs are included at each storage pad node using the GTSTRUDL function "CALCULATE SOIL SPRING VALUES COMPRESSION ONLY DIR Y…."

[ ]

[ ]

7.6.1.6 Analysis

Comparison of Wind and Seismic Load

Wind loads and seismic loads need to be considered for the design of the storage pad.

[ ]

Cask Stability Analysis

[ ]

All Indicated Changes are in response to RAI NP-7-3

Sliding Cask Stability:

Overturning Cask Stability:

7.6.1.7 Design Forces from Analysis

Cask Configuration 1

All Indicated Changes are in response to RAI NP-7-3

Cask Configuration 2

[ ]

Cask Configuration 3

[ ]

All Indicated Changes are in response to RAI NP-7-3

[ ]

Cask Configuration 4

[ ]

Summary of Maximum Design Forces:

[ ]

All Indicated Changes are in response to RAI NP-7-3

Assessment of VCT Loading:

Multiple VCT locations were considered in the analyses. [

]

7.6.1.8 Concrete Pad Capacity

Minimum Shrinkage and Temperature Reinforcement:

[

.]

Design Flexural Capacity:

The maximum moment for both directions (across length and width) are evaluated. Note that the top and bottom reinforcement (each face) are equal; therefore, the maximum design moment is taken as the maximum of the positive and negative moments. [

]

All Indicated Changes are in response to RAI NP-7-3

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All Indicated Changes are in response to RAI NP-7-3

One-Way Shear Capacity:

All Indicated Changes are in response to RAI NP-7-3

[ ]

Punching (Two-Way) Shear Capacity:

[ ]

All Indicated Changes are in response to RAI NP-7-3

[ ]

Stability of Storage Pad:

[ ]

All Indicated Changes are in response to RAI NP-7-3

Proprietary Information on This Page
Withheld Pursuant to 10 CFR 2.390

Storage Pad Soil Bearing Evaluation:

All Indicated Changes are in response to RAI NP-7-3

[ ]

7.6.1.9 Construction and Detailing Evaluations

Various reinforced concrete detailing requirements are discussed in the subsequent sections. In all cases, it has been verified that the requirements in ACI 349-06 [7-31] are consistent with those in ACI 318-08 [7-39].

Durability Considerations:

[ ]

Bar Development and Lap Splices:

[ ]

All Indicated Changes are in response to RAI NP-7-3

Construction/Control Joint Interfaces:

7.6.1.10 Results and Conclusions

Based on the evaluations performed, it is concluded that the licensing design of the NAC storage pad for Andrews, TX meets all of the applicable structural requirements of NUREG-1567 [7-28] with reference to NUREG-1536 [7-42] and NUREG-0800 [7-43].

The WCS CISF licensing design includes consideration of four cask configurations on the pad based on systematically loading the pad with casks from one short side moving across to the other. Seismic, operational wind, and tornado wind were all considered to act on the casks. In the case of an SSE event, the VCCs do not overturn; however, the casks could slide up to *1.20* in (considering a safety factor of two). Furthermore, the concrete pad could slide up to *1.0* in (considering a safety factor of two).

All Indicated Changes are in response to RAI NP-7-3

Impact from cask drop or tornado-generated missiles was not considered with respect to the storage pad. The casks are already qualified for impact conditions and impact to the storage pad is an accident condition where damage is acceptable as long as there is no loss of function. The VCT was considered at several locations while fully supporting a cask. Operational wind load was applied to the VCT; however, seismic and tornado wind were not considered given that cask movements are infrequent evolutions.

7.6.2 Soil Liquefaction and VCC Storage Pad Settlement

The purpose of this evaluation is to determine the liquefaction potential and elastic settlement of the VCC storage pad located at the WCS CISF in Andrews, Texas.

The scope of work included:

- Review of Drawing NAC004-C-001, Rev. 0 showing the dimensions and general arrangement of the storage pad [7-30], and review of Drawing NAC004-C-002, Rev. 0 showing the structural concrete plan, sections, and details [7-37].
- Review of "Report of Geotechnical Exploration" performed by GEOServices, LLC [7-32].
- Liquefaction potential evaluation using the data from reference [7-32].
- Elastic settlement evaluation under static loading conditions using the data from reference [7-32].

7.6.2.1 Design Basis

All Indicated Changes are in response to RAI NP-7-3

7.6.2.2 Design Inputs

Soil Properties

[ ]

Relevant Concrete Pad Properties

[ ]

7.6.2.3 Analysis

Liquefaction Potential Evaluation

Liquefaction potential evaluation was based on NRC Regulatory Guide 1.198 [7-52] and widely accepted empirical methodology using Standard Penetration Test (SPT) and laboratory test data [7-53].

[ ]

7.6.2.4 Elastic Settlement Evaluation

[ ]

All Indicated Changes are in response to RAI NP-7-3

[ ]

7.6.2.5 Calculations

Liquefaction Potential

[ ]

All Indicated Changes are in response to RAI NP-7-3

[

]

7.6.2.6 Results and Conclusions

Based on the evaluation contained, it is concluded that overall, the soils below the Storage Pad are not susceptible to liquefaction.

Based on the evaluation contained in reference [7-32], the estimated maximum settlement of the Storage Pad (assuming the pad to be flexible for settlement purposes) is on the order of 0.70 inch (Section 4.2, Ref. [7-32]).

7.6.3 Soil Structure Interaction of the VCC Storage Pad

This section documents the Soil Structure Interaction (SSI) analysis to support a concrete pad design for the VCC storage pads located at the WCS CISF in Andrews Country, Texas. The analysis is conducted in accordance with NUREG-0800 [7-43].

The SSI analysis considers the concrete pad design with the MAGNASTOR VCC, which envelopes the NAC-UMS and NAC-MPC VCCs to be stored at the WCS CISF, for 4 cask load configurations, 3 soil cases, and 3 time histories, totaling 36 analysis cases to obtain enveloping maximum accelerations at the VCC center of gravities, the concrete pad center of gravity, and an evaluation for sliding and overturning of the VCCs. The SSI analysis supports structural design of the VCC storage pad system.

7.6.3.1 Design Basis

[ ]

All Indicated Changes are in response to RAI NP-7-3

Proprietary Information on This Page
Withheld Pursuant to 10 CFR 2.390

7.6.3.2 Design Inputs

The inputs used to prepare and execute the SSI analysis are as follows:

Finite Element Model Inputs:

[ ]

Soil Model Inputs:

[ ]

Time History Inputs:

[ ]

SSI Analysis Inputs:

[ ]

All Indicated Changes are in response to RAI NP-7-3

[ ]

7.6.3.3 Calculations

The following sections detail the calculations of the inputs for the SSI analysis.

SSI Soil Model

[ ]

All Indicated Changes are in response to RAI NP-7-3

7.6.3.4 Results and Conclusions

Following SSI analysis of 36 analysis cases it was found that the enveloping maximum accelerations at the MAGNASTOR Cask center of gravity are as follows:

- 0.45g in the X/E-W Direction for Case 30, Coyote Lake earthquake on UB soil at cask CG B1 for cask configuration 4
- 0.42g in the Y/N-S Direction for Case 30, Coyote Lake earthquake on UB at cask CG A2 for cask configuration 4
- 0.28g in the Z/Vertical Direction for Case 22, Norcia earthquake on LB soil at cask CG B3 for cask configuration 3

The MAGNASTOR cask envelopes all other vertical VCC types to be stored at the WCS CISF. Through examining the instantaneous coefficient of friction demand, it is deemed that cask sliding is likely to occur for at least 1 cask due to a maximum coefficient of friction demand of 0.46, which is greater than the coefficient of friction of 0.35 for cask steel-to-concrete contact for a light broom finish on the concrete pad.

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7-22 NAC International Report 630075-R-06, Rev. 3, Appendix B, Failure Modes and Effects Analysis.

7-23 NAC International Report 630075-R-06, Rev. 3, Appendix C, Crane Operations Descriptions.

7-24 NAC International Report 630075-R-06, Rev. 3, Appendix D, Kuosheng Hydraulic Gantry Crane NUREG-0554/ASME NOG-1 Conformance Matrix.

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All Indicated Changes are in response to RAI NP-7-3

Table 7-11
Enveloping Element Forces (kip/in) and Moments (kip-in/in) for Cask Configuration 1

**** MAXIMUM AND MINIMUM SUMMARY OF ABOVE ENVELOPE RESULTS ****

RESULT	MAXIMUM	LOAD	JOINT	MINIMUM	LOAD	JOINT
NXX	0.973309E+02	133	5668	-0.971351E+02	126	5225
NYY	0.175945E+03	131	1389	-0.176289E+03	129	1389
NXY	0.710638E+02	131	126	-0.710638E+02	127	1146
MXX	0.496207E+02	131	403	-0.158170E+03	109	837
MYY	0.651214E+02	132	323	-0.130053E+03	104	299
MXY	0.221102E+02	104	3355	-0.232088E+02	201	2977
VXX	0.571444E+01	109	4966	-0.571444E+01	105	6316
VYY	0.621464E+01	104	2997	-0.621597E+01	105	3402

All Indicated Changes are in response to RAI NP-7-3

Table 7-12
Enveloping Element Forces (kip/in) and Moments (kip-in/in) for Cask Configuration 2

**** MAXIMUM AND MINIMUM SUMMARY OF ABOVE ENVELOPE RESULTS ****

RESULT	MAXIMUM	LOAD	JOINT	MINIMUM	LOAD	JOINT
NXX	0.569772E+02	132	5225	-0.568082E+02	126	5225
NYY	0.885718E+02	130	1398	-0.887461E+02	128	1398
NXY	0.371079E+02	128	126	-0.371079E+02	132	1146
MXX	0.556533E+02	108	323	-0.108215E+03	108	869
MYY	0.800629E+02	108	323	-0.146712E+03	105	617
MXY	0.351331E+02	105	2635	-0.351331E+02	109	8485
VXX	0.462325E+01	104	8899	-0.462324E+01	108	2149
VYY	0.445764E+01	104	2997	-0.430292E+01	104	3078

All Indicated Changes are in response to RAI NP-7-3

Table 7-13
Enveloping Element Forces (kip/in) and Moments (kip-in/in) for Cask Configuration 3

**** MAXIMUM AND MINIMUM SUMMARY OF ABOVE ENVELOPE RESULTS ****

RESULT	MAXIMUM	LOAD	JOINT	MINIMUM	LOAD	JOINT
NXX	0.765872E+02	128	5675	-0.763796E+02	126	5225
NYY	0.125460E+03	130	1398	-0.125541E+03	132	9954
NXY	0.519504E+02	128	126	-0.519640E+02	132	1146
MXX	0.659693E+02	131	991	-0.139056E+03	108	869
MYY	0.930660E+02	104	943	-0.125794E+03	108	989
MXY	0.413002E+02	104	2743	-0.420798E+02	108	8593
VXX	0.602652E+01	108	9007	-0.594009E+01	104	2257
VYY	0.573442E+01	108	7497	-0.557302E+01	108	7686

All Indicated Changes are in response to RAI NP-7-3

Table 7-14
Enveloping Element Forces (kip/in) and Moments (kip-in/in) for Cask Configuration 4

**** MAXIMUM AND MINIMUM SUMMARY OF ABOVE ENVELOPE RESULTS ****

RESULT	MAXIMUM	LOAD	JOINT	MINIMUM	LOAD	JOINT
NXX	0.907013E+02	128	5675	-0.905094E+02	126	5225
NYY	0.153049E+03	130	1398	-0.153281E+03	132	9954
NXY	0.619094E+02	126	1146	-0.619172E+02	130	126
MXX	0.654932E+02	132	1003	-0.151329E+03	109	837
MYY	0.972217E+02	108	905	-0.138299E+03	108	1001
MXY	0.436717E+02	105	2851	-0.439850E+02	109	8701
VXX	0.678284E+01	108	9115	-0.670758E+01	104	2365
VYY	0.608036E+01	108	7497	-0.672943E+01	108	7794

All Indicated Changes are in response to RAI NP-7-3

Table 7-15

Load Combination 104 Resultant Force For Width Cut, Cask Configuration 4 (kip & kip-in)

```
==========================================================================
 LOADING - 104
==========================================================================

    LOCATION OF CENTROID (REFER TO NOTE ABOVE) =    0.9900000E+02

         IN-PLANE NORMAL FORCE,        P =   -0.3362797E+03

         IN-PLANE SHEAR FORCE
            (PARALLEL TO CUT),         V =   -0.6475717E+03

         OUT-OF-PLANE (TRANSVERSE)
            SHEAR FORCE,              FZ =    0.1912850E+03

         BENDING MOMENT (MOMENT
            VECTOR PARALLEL TO CUT),  MB =    0.7248867E+04

         TWISTING MOMENT (MOMENT
            VECTOR NORMAL TO CUT),    MT =   -0.1093596E+05

         IN-PLANE ROTATIONAL
            DEFORMATION MOMENT,       MZ =   -0.1293207E+05
```


All Indicated Changes are in response to RAI NP-7-3

Table 7-16
Load Combination 105 Resultant Force For Length Cut, Cask Configuration 2 (kip & kip-in)

```
========================================================================
 LOADING - 105
========================================================================

   LOCATION OF CENTROID (REFER TO NOTE ABOVE) =    0.9529411E+02

        IN-PLANE NORMAL FORCE,        P =    0.2532034E+01

        IN-PLANE SHEAR FORCE
           (PARALLEL TO CUT),         V =    0.4587067E+02

        OUT-OF-PLANE (TRANSVERSE)
           SHEAR FORCE,              FZ =    0.1734929E+03

        BENDING MOMENT (MOMENT
           VECTOR PARALLEL TO CUT), MB =    0.6583913E+04

        TWISTING MOMENT (MOMENT
           VECTOR NORMAL TO CUT),   MT =    0.3900859E+04

        IN-PLANE ROTATIONAL
           DEFORMATION MOMENT,       MZ =    0.9978431E+03
```


All Indicated Changes are in response to RAI NP-7-3

Table 7-17
Load Combination 301 Enveloping Reactions Cask Configuration 4 (kip & kip-in)

```
**** Summary of Global Reaction Envelopes ****

=================================================
  Type            Value         Load      Joint
=================================================

Force X  Min  -0.774047E-01  301       611
         Max   0.774047E-01  301       661
Force Y  Min   0.000000E+00  301       126
         Max   0.000000E+00  301       126
Force Z  Min  -0.189993E+00  301       126
         Max   0.189993E+00  301       1146
Moment X Min   0.000000E+00  301       126
         Max   0.000000E+00  301       126
Moment Y Min   0.000000E+00  301       126
         Max   0.000000E+00  301       126
Moment Z Min   0.000000E+00  301       126
         Max   0.000000E+00  301       126
=================================================
```


All Indicated Changes are in response to RAI NP-7-3

Table 7-18
Load Combination 302 To 309 Enveloping Reactions Cask Configuration 1
(kip & kip-in)

**** Summary of Global Reaction Envelopes ****

Type		Value	Load	Joint
Force X	Min	-0.278213E+04	303	611
	Max	0.278213E+04	307	611
Force Y	Min	0.000000E+00	302	126
	Max	0.000000E+00	302	126
Force Z	Min	-0.278292E+04	306	126
	Max	0.278292E+04	307	126
Moment X	Min	0.000000E+00	302	126
	Max	0.000000E+00	302	126
Moment Y	Min	0.000000E+00	302	126
	Max	0.000000E+00	302	126
Moment Z	Min	0.000000E+00	302	126
	Max	0.000000E+00	302	126

All Indicated Changes are in response to RAI NP-7-3

Table 7-19

Load Combination 301 Maximum Displacements Cask Configuration 1 (in)

```
****SUMMARY OF MAXIMUM GLOBAL DISPLACEMENTS****
        INDEPENDENT IN EACH COORDINATE


=============================================
* RESULT *    MAXIMUM       LOAD     JOINT   *
*========*====================================*
* X-DISP * -0.233258E-02  301       225      *
* Y-DISP * -0.393650E+00  301       627      *
* Z-DISP *  0.851354E-03  301       403      *
=============================================
```


All Indicated Changes are in response to RAI NP-7-3

Table 7-20
Load Combination 302 to 309 Maximum Displacements Cask Configuration 7 (in)

****SUMMARY OF MAXIMUM GLOBAL DISPLACEMENTS****
INDEPENDENT IN EACH COORDINATE

RESULT	MAXIMUM	LOAD	JOINT
X-DISP	-0.368355E+00	308	175
Y-DISP	-0.534379E+00	309	525
Z-DISP	-0.263145E+00	308	102

All Indicated Changes are in response to RAI NP-7-3

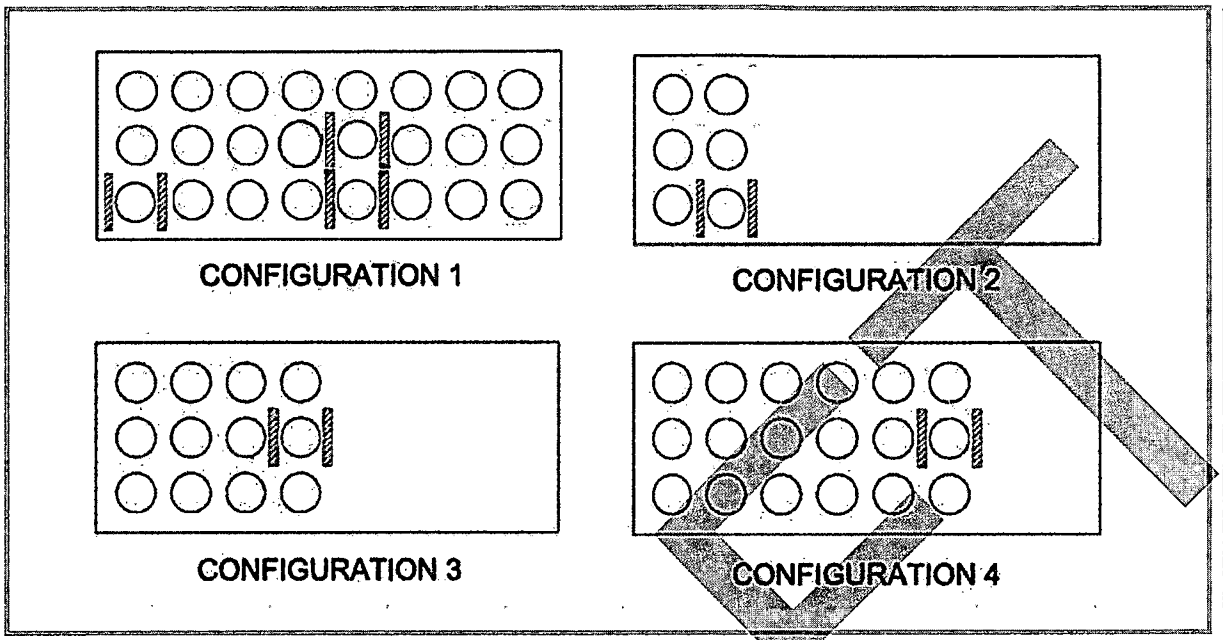

Figure 7-9
Cask Layout Design Configurations and VCT Locations

All Indicated Changes are in response to RAI NP-7-3

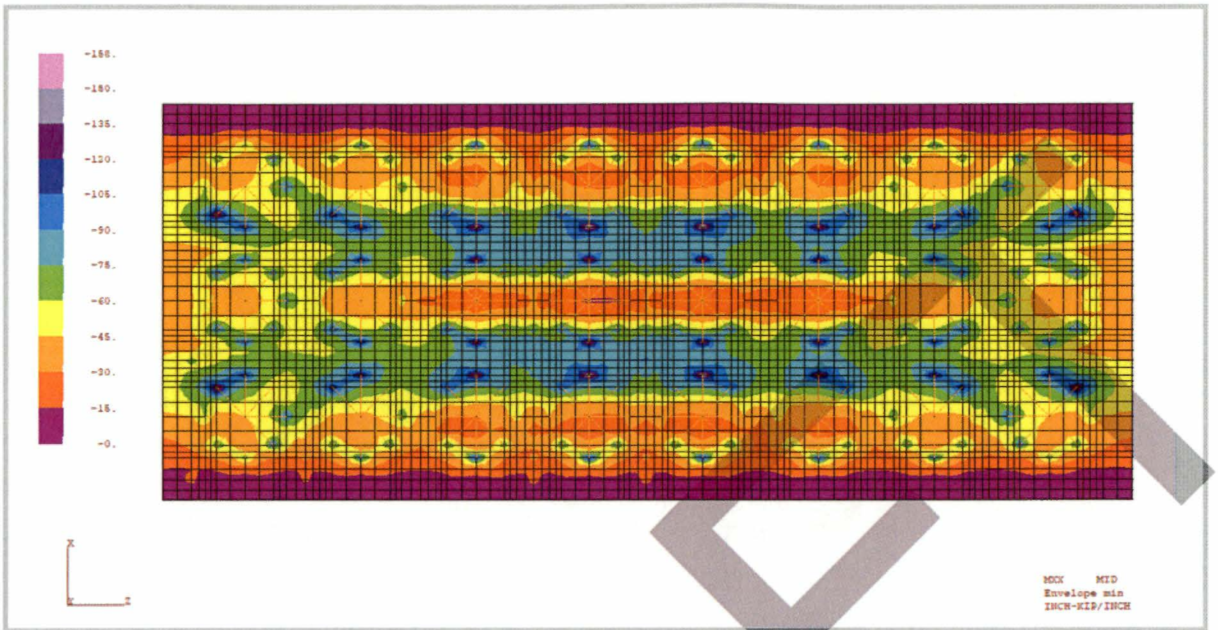

Figure 7-10
Enveloping Negative Moment Resultants (kip-in/in) About Global Z-Axis (local M_{XX}) - Configuration 1

All Indicated Changes are in response to RAI NP-7-3

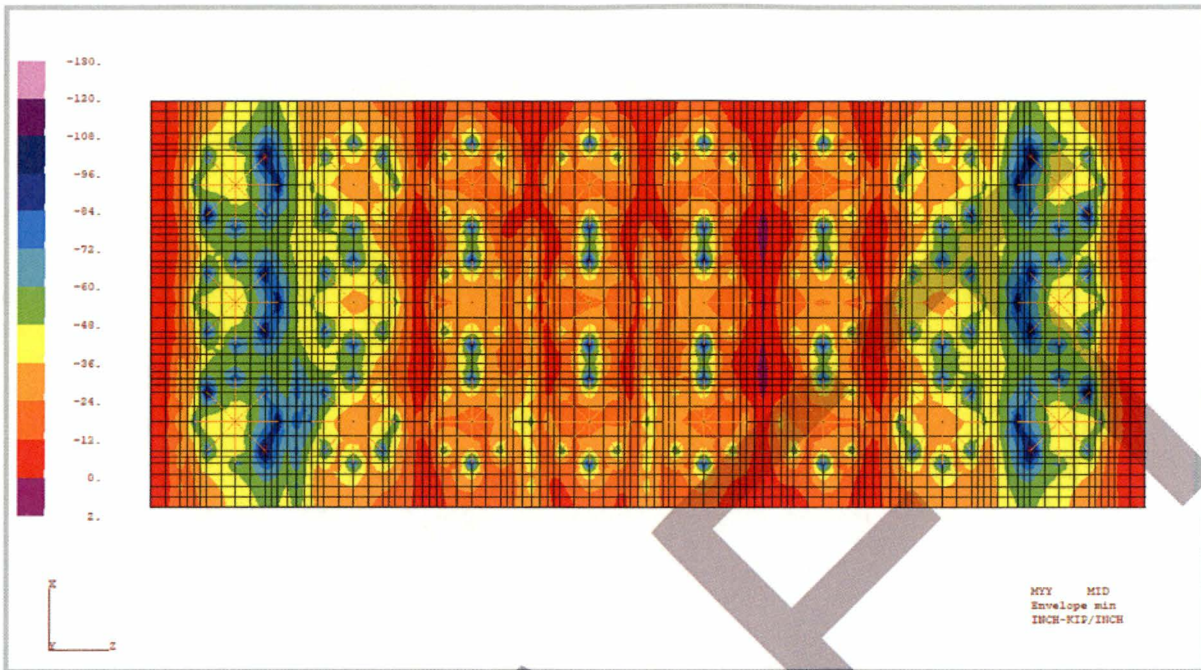

Figure 7-11
Enveloping Negative Moment Resultants (kip-in/in) About Global X-Axis (local M_{YY}) - Configuration 1

All Indicated Changes are in response to RAI NP-7-3

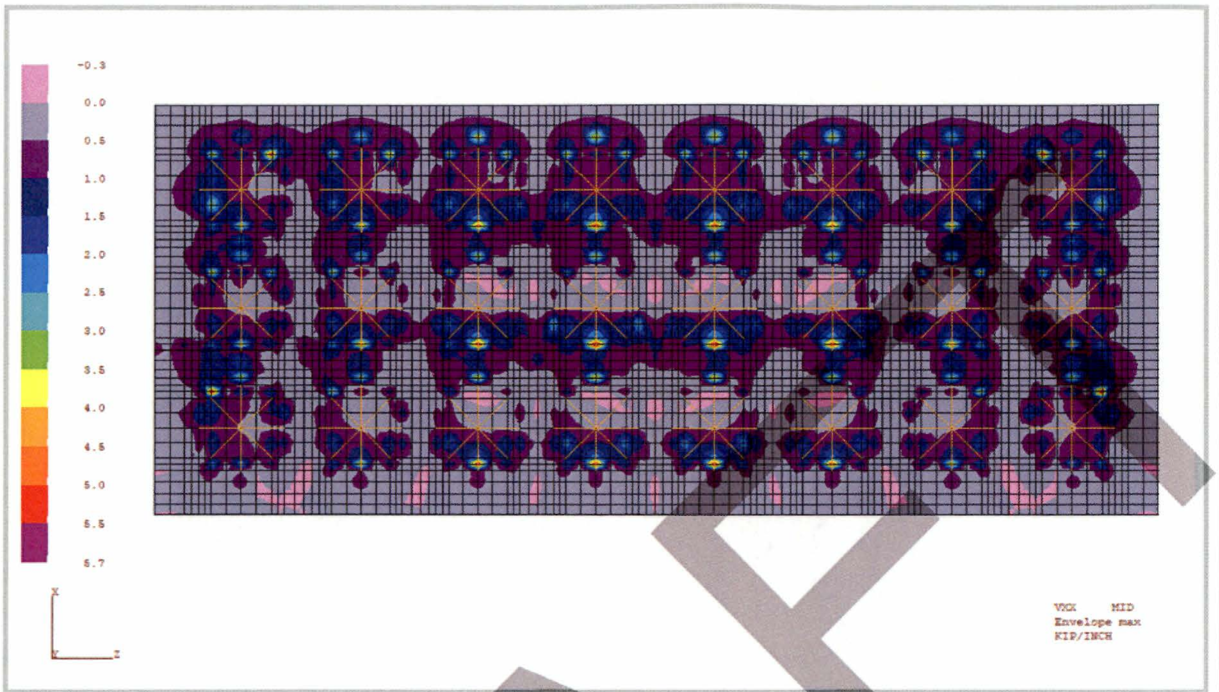

Figure 7-12
Enveloping Element Shear (kip/in) In The Global Y-Direction On The X-Face (local V_{XX}) - Configuration 1

All Indicated Changes are in response to RAI NP-7-3

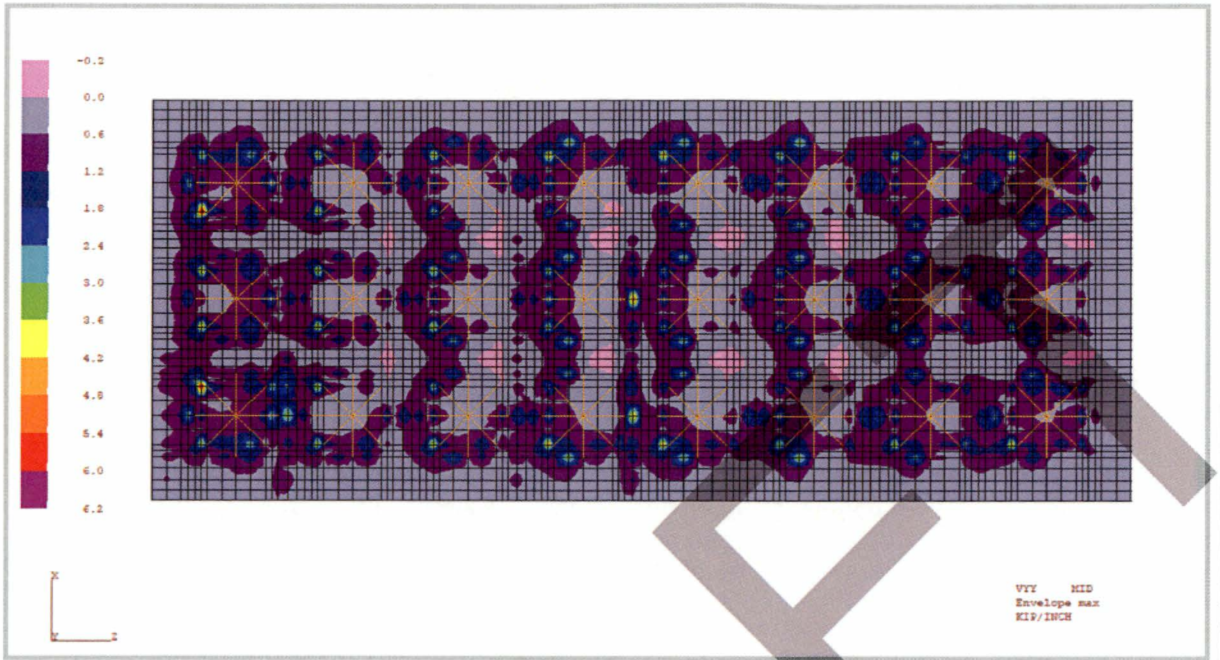

Figure 7-13
Enveloping Element Shear (kip/in) In The Global Y-Direction On The Z-Face (local V_{YY}) - Configuration 1

All Indicated Changes are in response to RAI NP-7-3

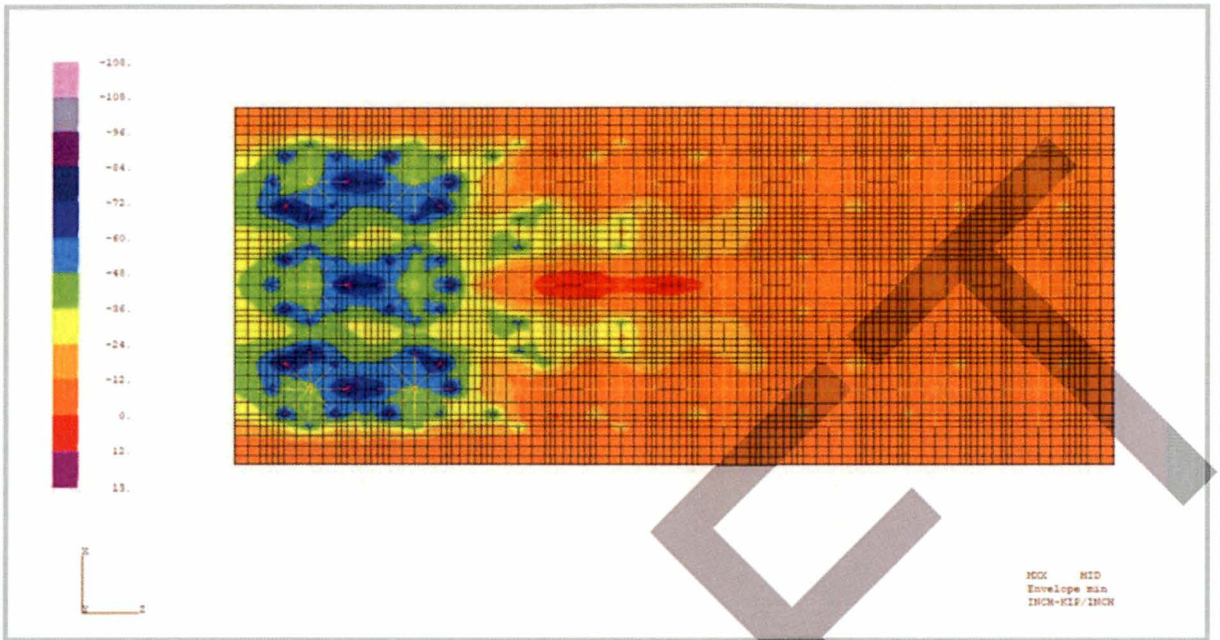

Figure 7-15
Enveloping Negative Moment Resultants (kip-in/in) About Global Z-Axis (local M_{XX}) - Configuration 2

All Indicated Changes are in response to RAI NP-7-3

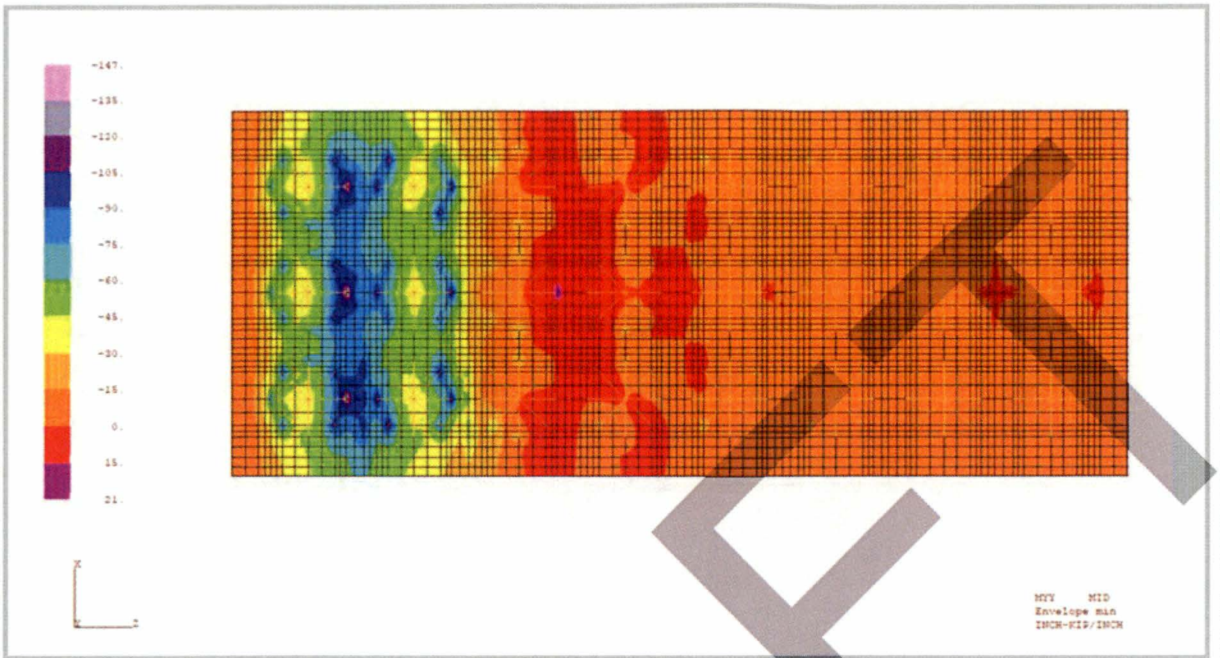

Figure 7-16
Enveloping Negative Moment Resultants (kip-in/in) About Global X-Axis (local M_{YY}) - Configuration 2

All Indicated Changes are in response to RAI NP-7-3

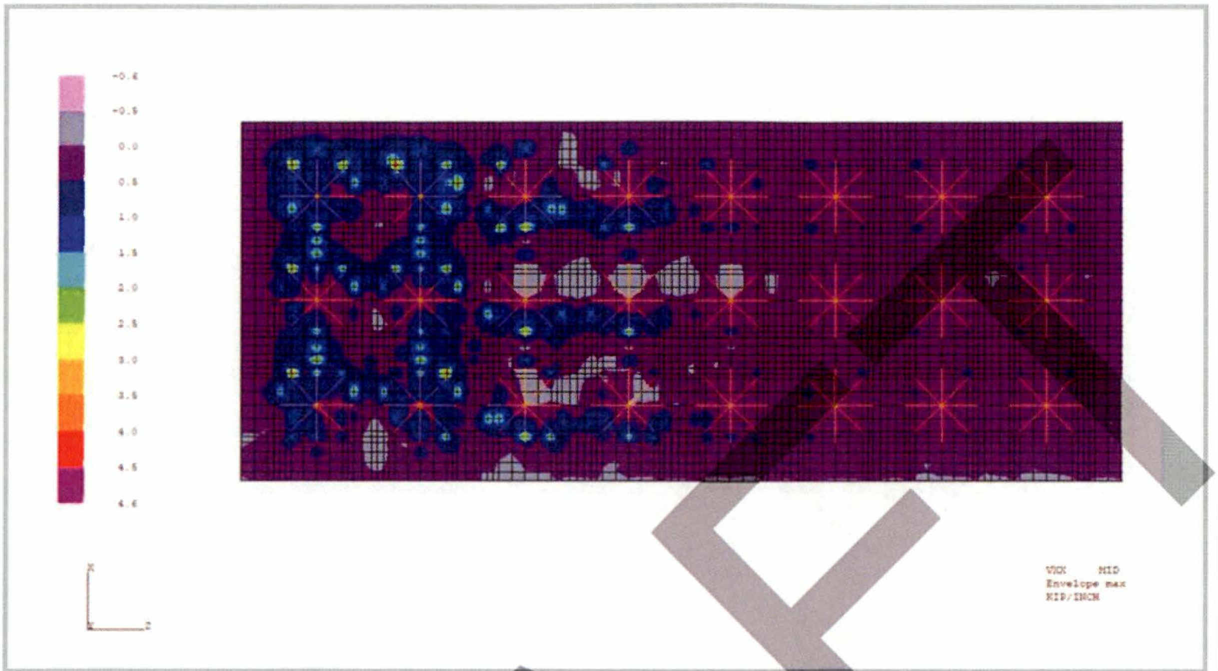

Figure 7-17
Enveloping Element Shear (kip/in) In The Global Y-Direction On The X-Face (local V_{XX}) - Configuration 2

All Indicated Changes are in response to RAI NP-7-3

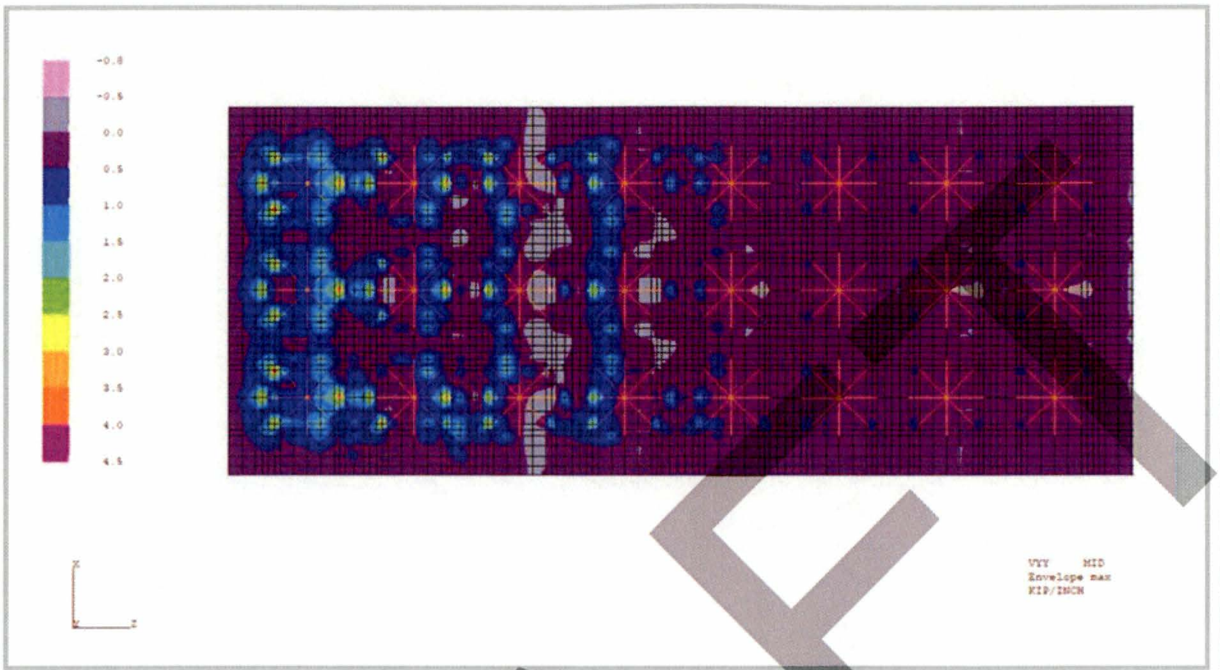

Figure 7-18
Enveloping Element Shear (kip/in) In The Global Y-Direction On The Z-Face (local V_{YY}) - Configuration 2

All Indicated Changes are in response to RAI NP-7-3

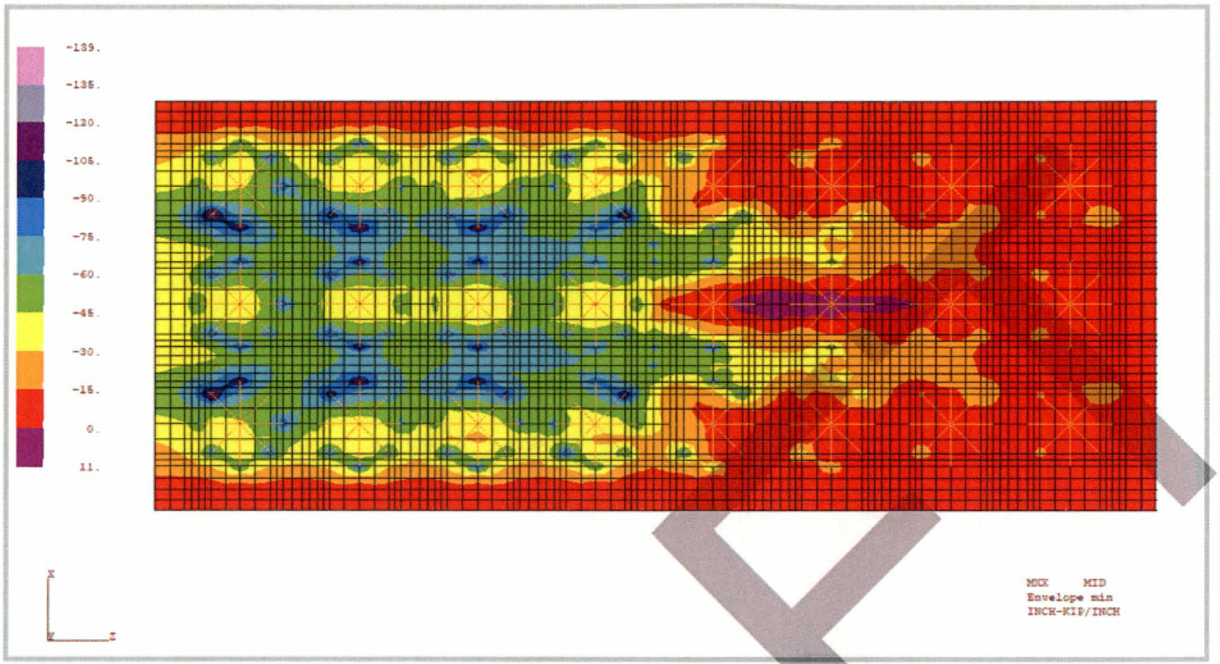

Figure 7-19
Enveloping Negative Moment Resultants (kip-in/in) About Global Z-Axis (local M_{XX}) - Configuration 3

All Indicated Changes are in response to RAI NP-7-3

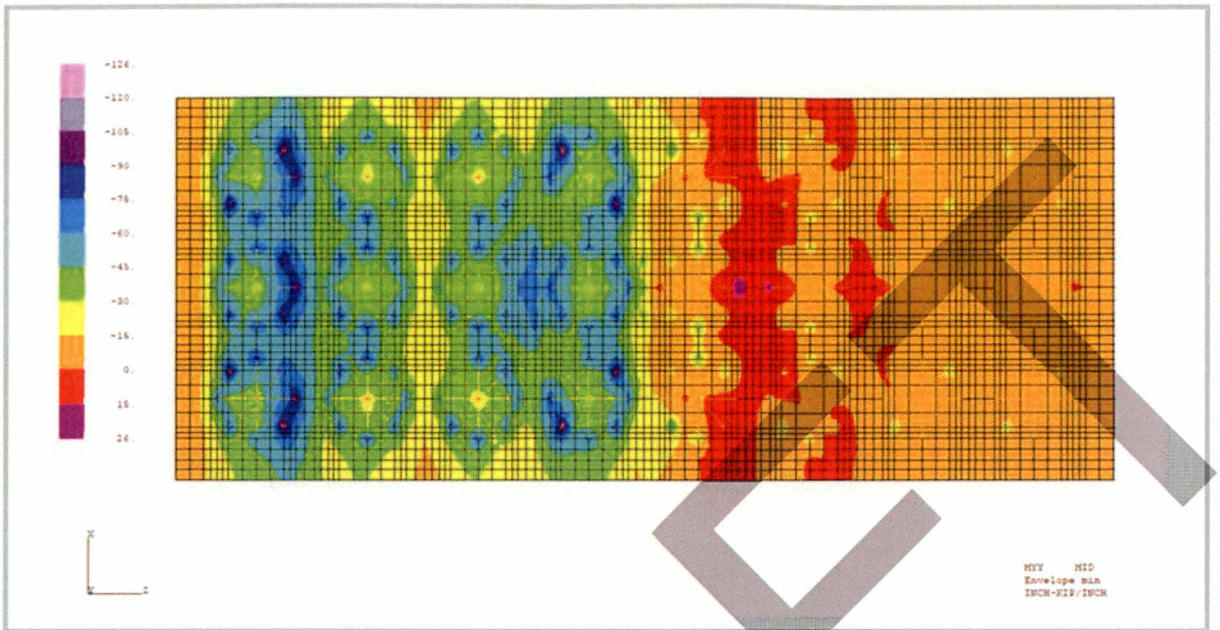

Figure 7-20
Enveloping Negative Moment Resultants (kip-in/in) About Global X-Axis (local M_{YY}) - Configuration 3

All Indicated Changes are in response to RAI NP-7-3

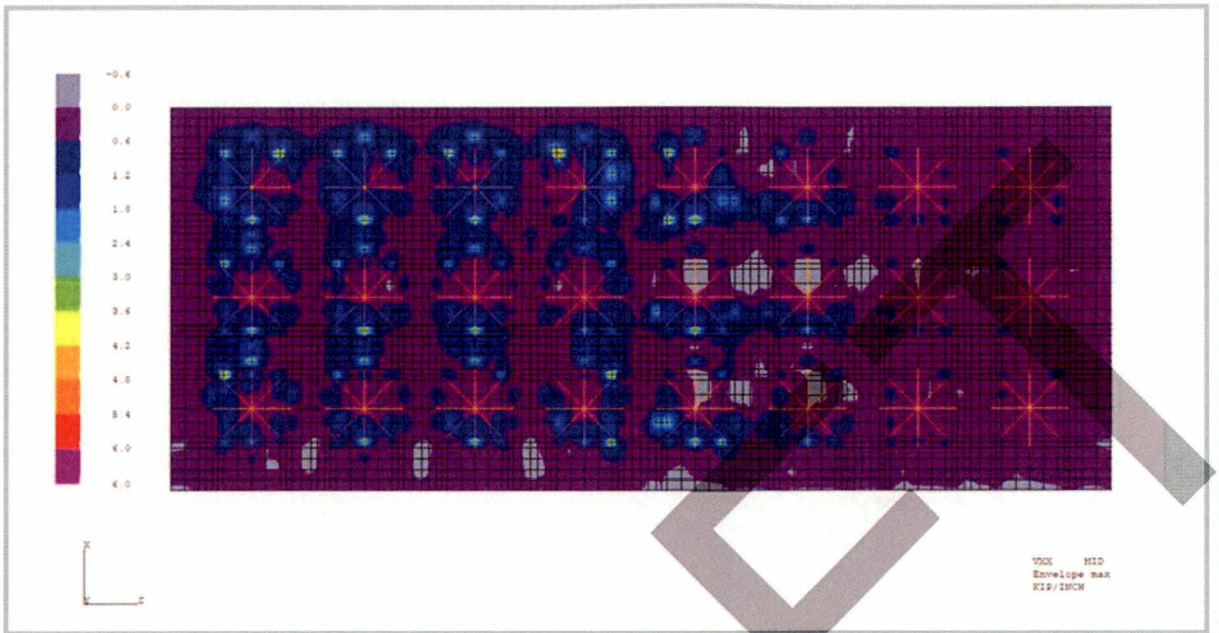

Figure 7-21
Enveloping Element Shear (kip/in) In The Global Y-Direction On The X-Face (local V_{XX}) - Configuration 3

All Indicated Changes are in response to RAI NP-7-3

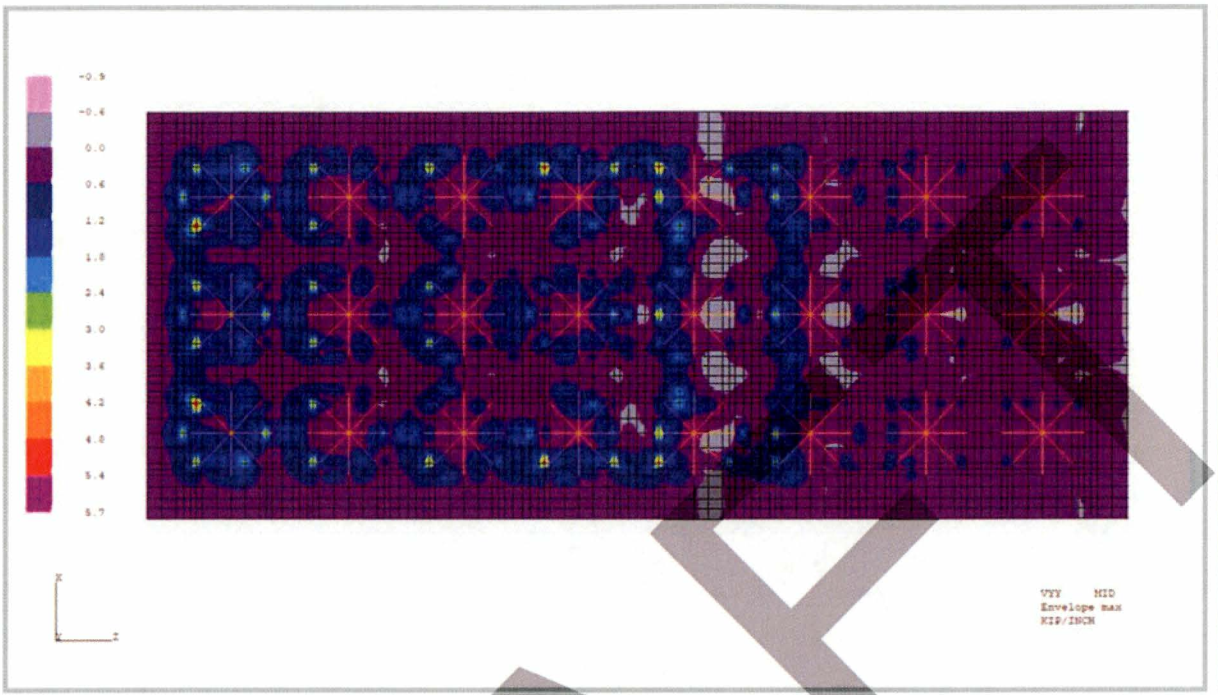

Figure 7-22
Enveloping Element Shear (kip/in) In The Global Y-Direction On The Z-Face (local V_{YY}) - Configuration 3

All Indicated Changes are in response to RAI NP-7-3

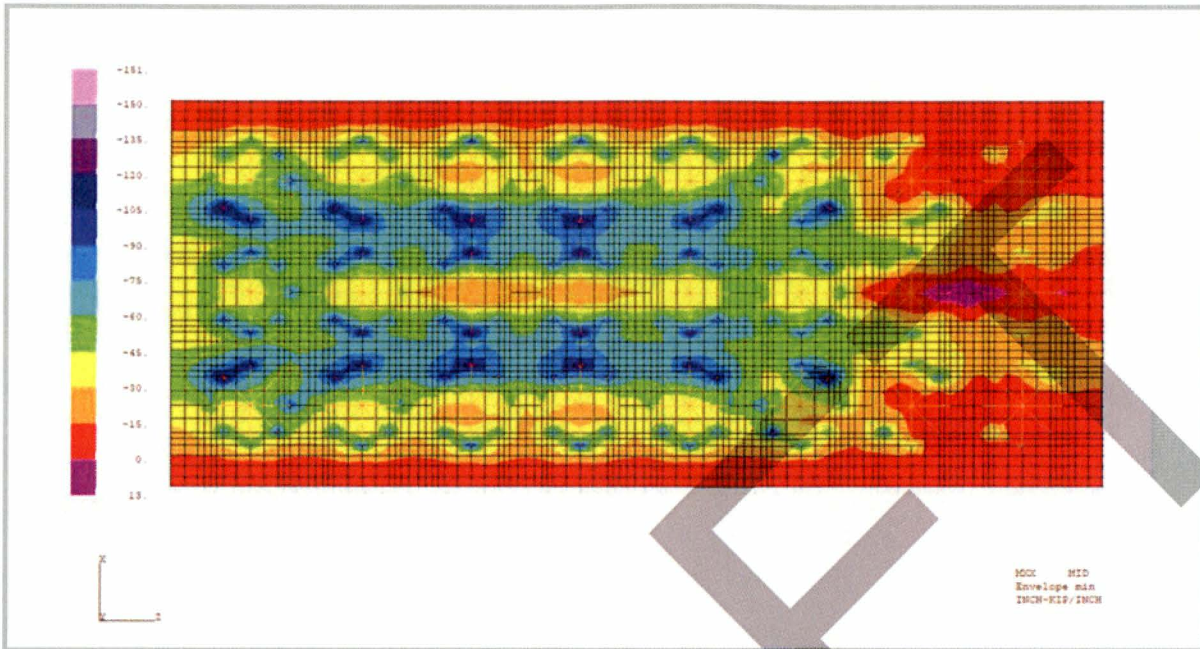

Figure 7-23
Enveloping Negative Moment Resultants (kip-in/in) About Global Z-Axis (local M_{XX}) - Configuration 4

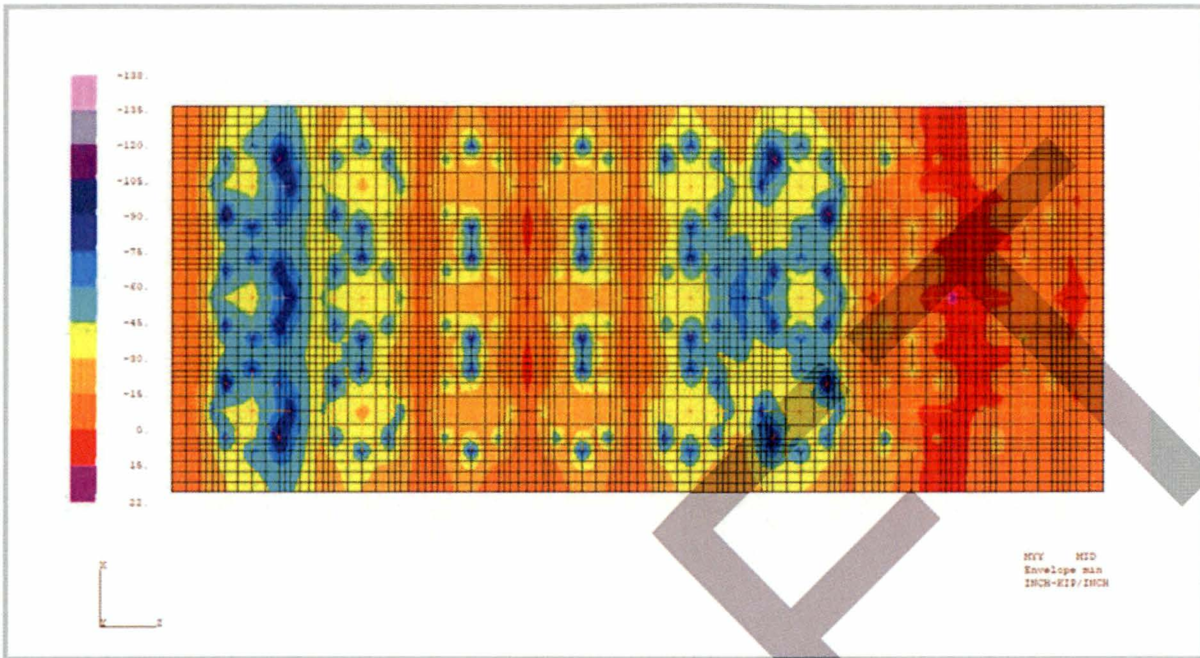

Figure 7-24
Enveloping Negative Moment Resultants (kip-in/in) About Global X-Axis (local M_{YY}) - Configuration 4

All Indicated Changes are in response to RAI NP-7-3

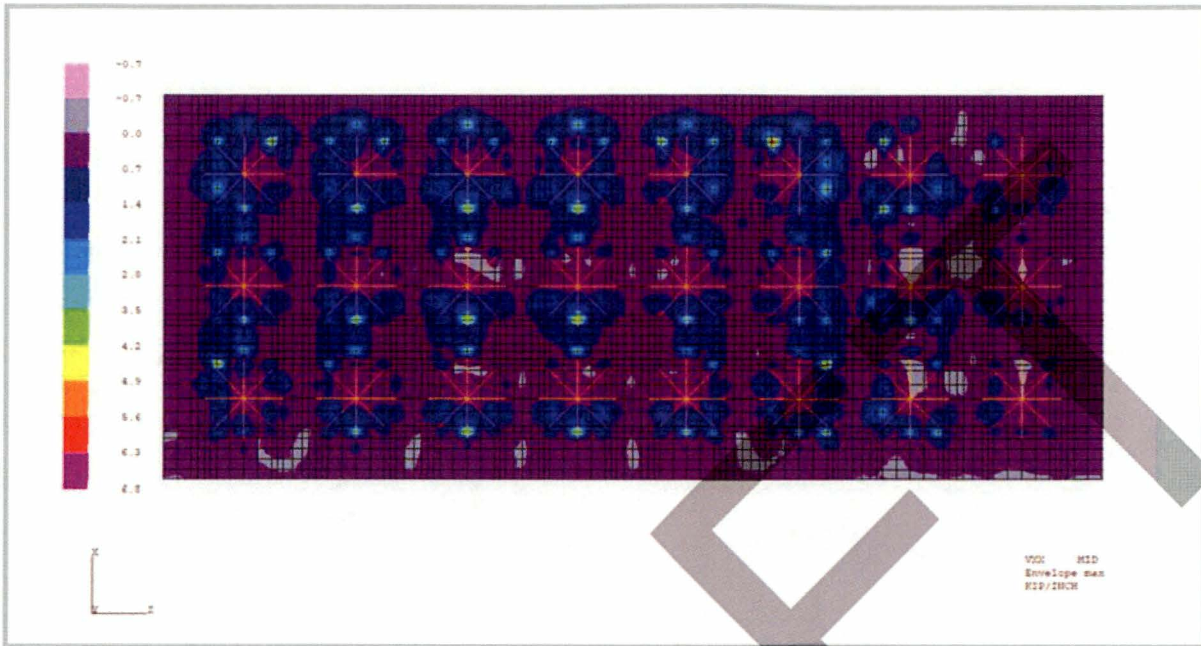

Figure 7-25
Enveloping Element Shear (kip/in) In The Global Y-Direction On The X-Face (local V_{XX}) - Configuration 4

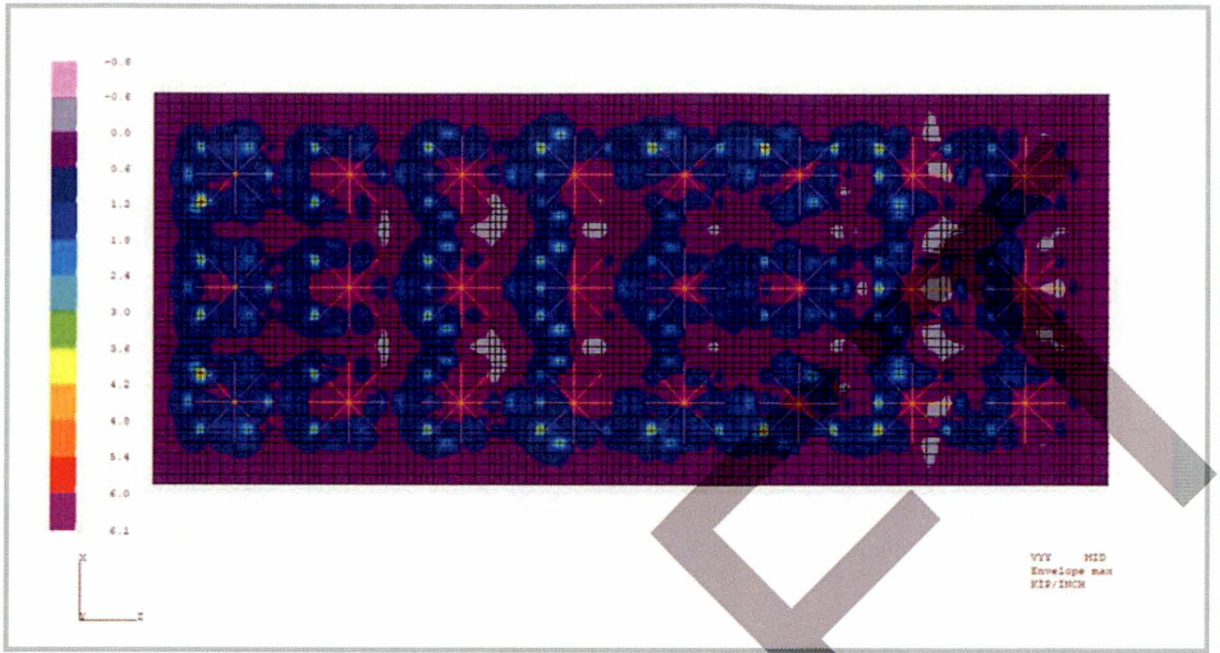

Figure 7-26
Enveloping Element Shear (kip/in) In The Global Y-Direction On The Z-Face (local V_{YY}) - Configuration 4

All Indicated Changes are in response to RAI NP-7-3

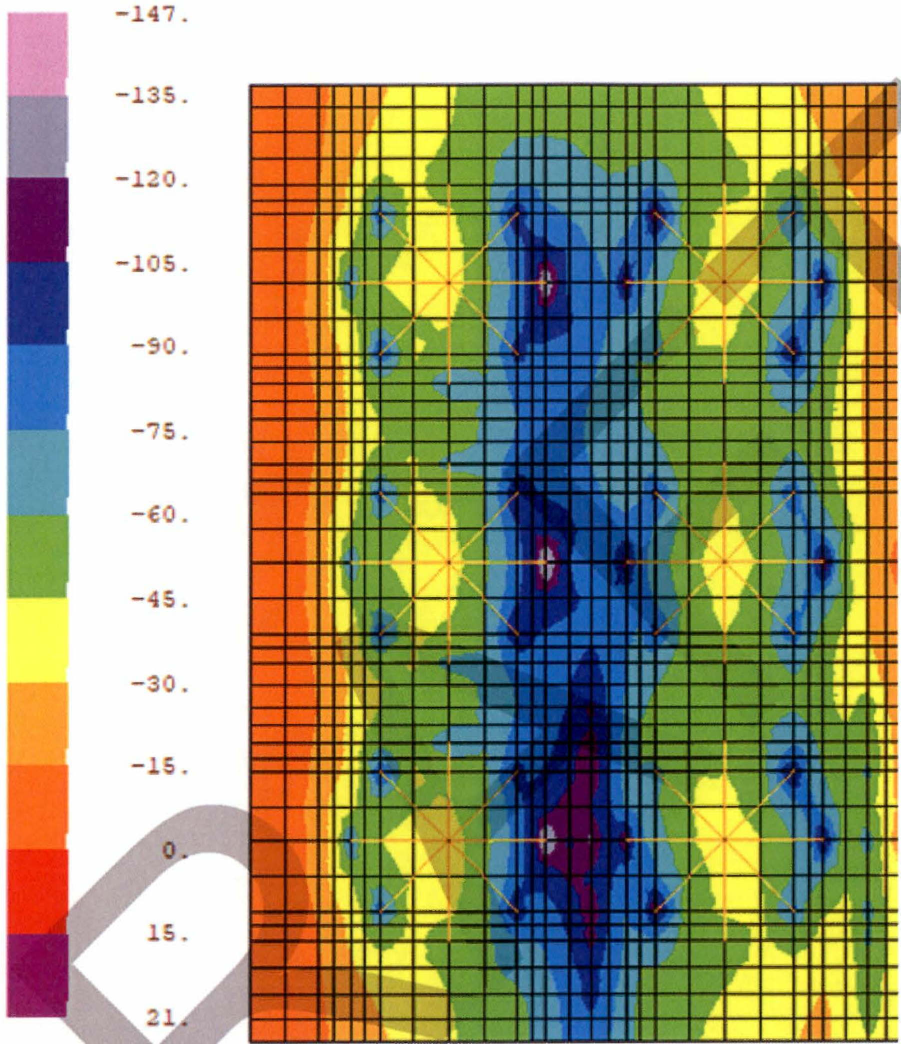

Figure 7-27
Excerpt Of Load Configuration *2* Enveloping Negative Moment Resultants *(kip-in/in)* About Global X-Axis (Local M_{YY}) Showing Effect Of VCT Loading

All Indicated Changes are in response to RAI NP-7-3

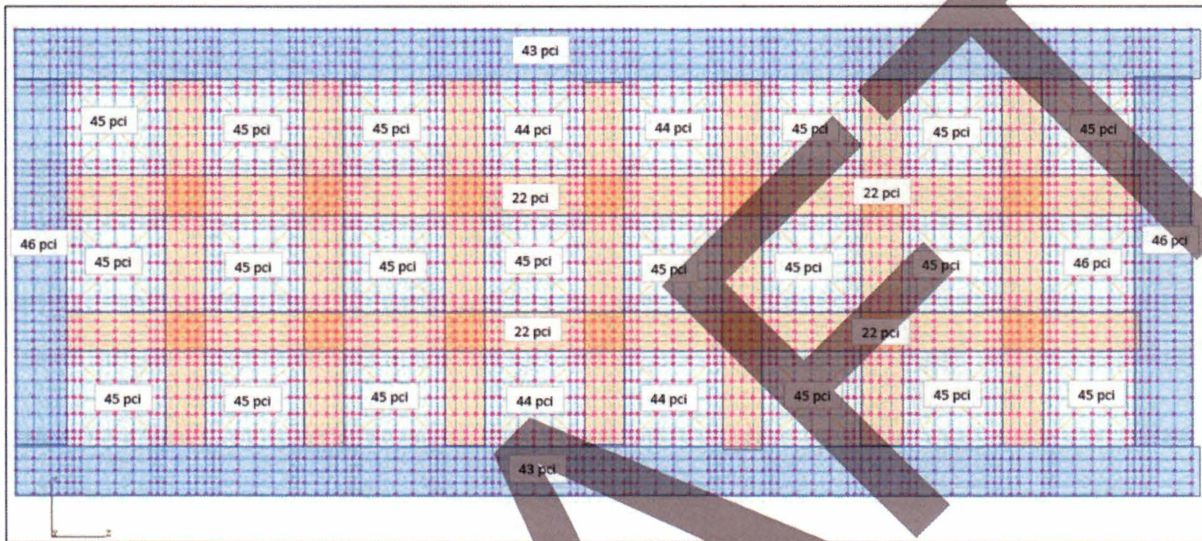

Figure 7-64
Storage Pad Cask Configuration 1 – Modulus of Sub-Grade Reactions for Soil Below the Different Sections of the Pad Used for Design Evaluation

All Indicated Changes are in response to RAI NP-7-3

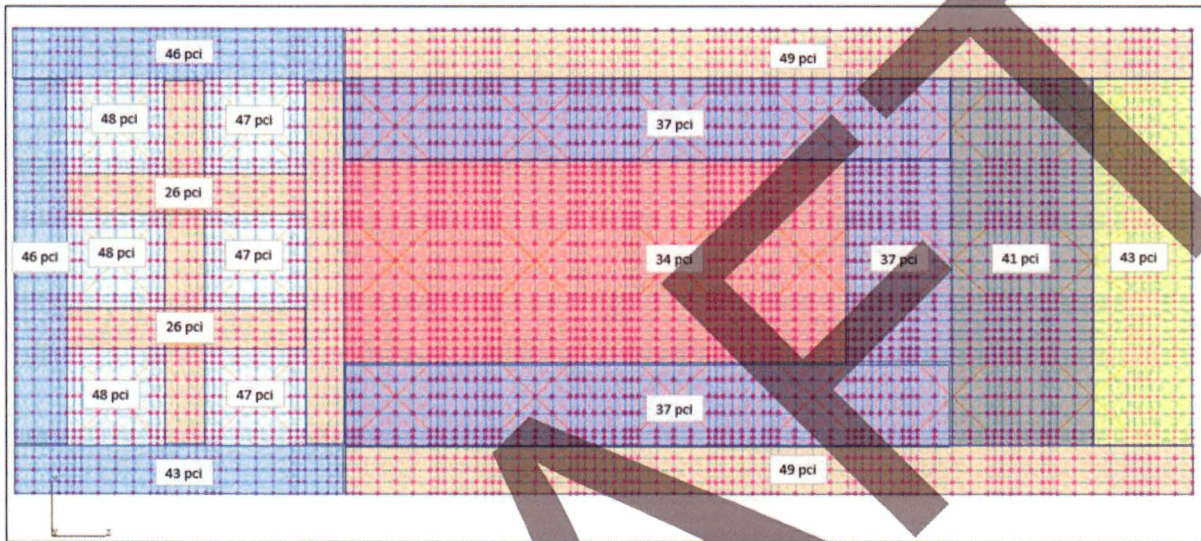

Figure 7-65
Storage Pad Cask Configuration 2 – Modulus of Sub-Grade Reactions for Soil Below the Different Sections of the Pad Used for Design Evaluation

All Indicated Changes are in response to RAI NP-7-3

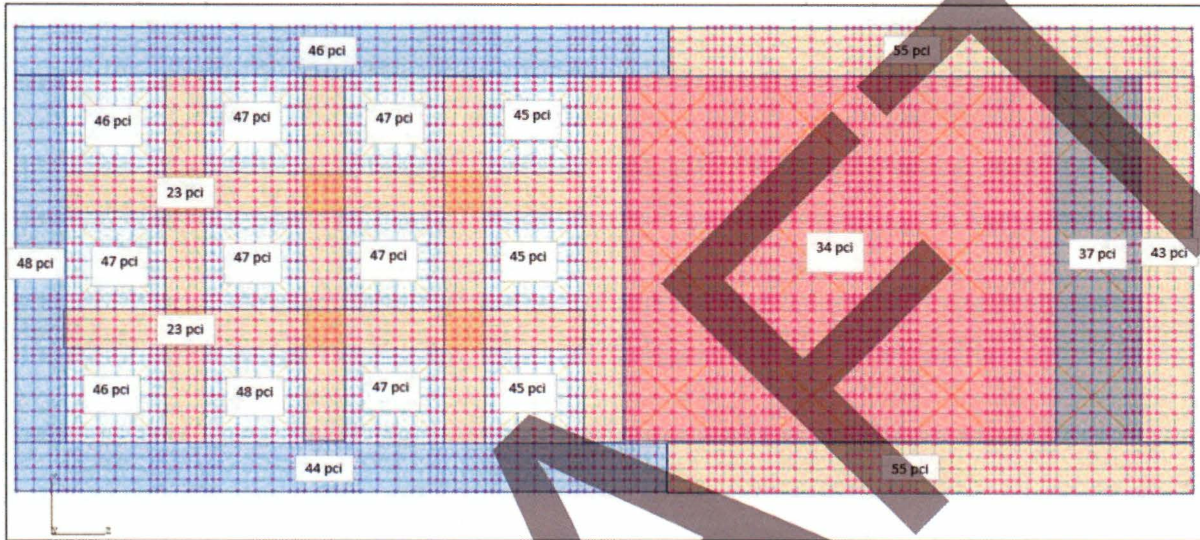

Figure 7-66
Storage Pad Cask Configuration 3 – Modulus of Sub-Grade Reactions for Soil Below the Different Sections of the Pad Used for Design Evaluation

All Indicated Changes are in response to RAI NP-7-3

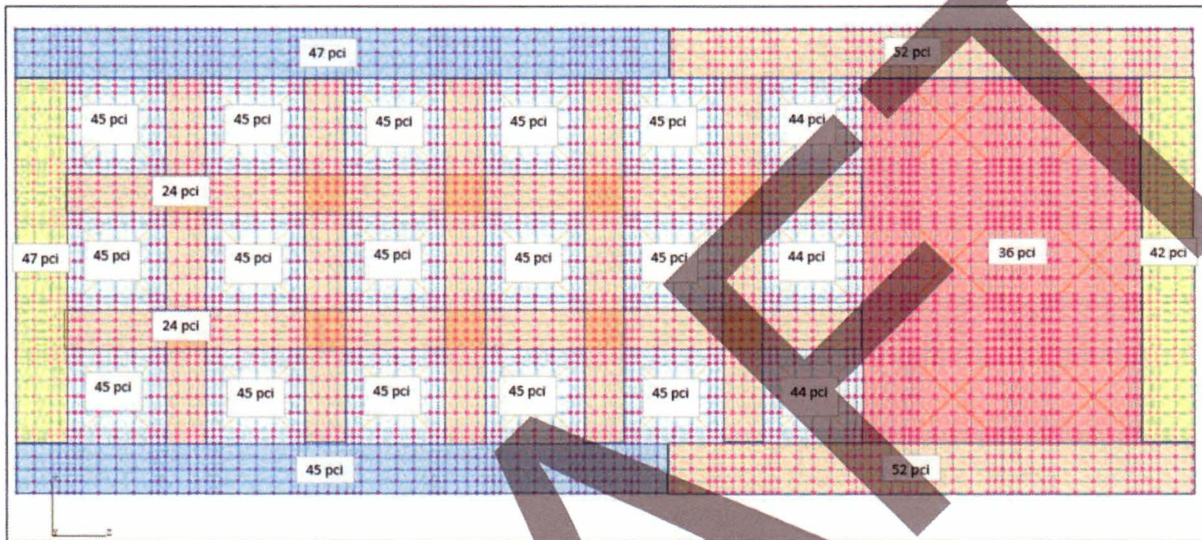

Figure 7-67
Storage Pad Cask Configuration 4 – Modulus of Sub-Grade Reactions for Soil Below the Different Sections of the Pad Used for Design Evaluation

All Indicated Changes are in response to RAI NP-7-3

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:

Background

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." (included in Enclosure X)

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 X)

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 X)

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 (Enclosure X), 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 Sections 7.6.4 and 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, 7.6.4, and 7.6.5, Tables 7-29 through 7-40, and Figures 7-31, and 7-33 through 7-53 have been revised as described in the response.

- *The required strength of OCBF vertical brace connections is determined using the overstrength seismic loads, in accordance with AISC 341-16 Section F1.6a. This requirement is met by designing for $F_{\mu} = 1.0$ seismic demands, in accordance with ASCE 43-05.*
- *All OCBF welded connections are detailed and installed in accordance with the applicable requirements of AWS D1.1 and D1.8 as required.*
- *Column base connections and splices are designed for the required axial, shear, and flexural forces defined in ANSI/AISC 341-16 Sections D2.5 and D2.6.*
- *The available strengths of concrete and reinforcing steel utilized in column base anchorage to the foundation are determined in accordance with ACI 349-13.*

7.5.3.5 Reinforced Concrete Structural Analysis and Design

Analysis and design of the CHB reinforced concrete foundations is performed in accordance with the requirements of ACI 349-13, considering all design load combinations defined in Section 7.5.3.2.3. This is in general accordance with the NUREG-1567 reference to ANSI/ANS 57.9, which in turn references ACI 349-85 for concrete load combinations and design limits. Design of CHB column baseplate anchorage is in accordance with the requirements of ACI 349-13 Appendix D.

Material properties considered in foundation analysis and design, including specified strengths for structural concrete, reinforcing steel, anchor rods, and steel plate (utilized for baseplate shear lugs) are summarized in Table 15-2. Soil properties considered in foundation design are those specified in the project geotechnical report (SAR Attachment E). This includes an allowable bearing pressure of 4000 lb/ft² and a subgrade modulus of 150 lb/in³. As stated in the geotechnical report, the allowable bearing pressure is permitted to be increased to 6000 lb/ft² for limit state loadings. The unit weight of structural fill considered in foundation stability calculations is assumed to be 110 lb/ft³.

Foundation stability is evaluated for the west strip mat foundation, which is considered representative of all three strip mats. The east and west strip mats have a narrower plan dimension in the east-west direction than the center strip mat, while the west strip mat has somewhat less applied dead load with fewer crane columns than the east strip mat. A minimum factor of safety of 1.5 is required for sliding and overturning when evaluated for the stability load combination containing normal wind and crane operating loads in Section 7.5.3.2.3 (load combination #6). For the seismic and tornado uplift load combinations (#7 and #8 in Section 7.5.3.2.3), the minimum factor of safety for sliding and overturning is 1.1. This is in accordance with ASCE 43-05 Section 7.2 for seismic stability.

All Indicated Changes are in response to RAI NP-7-8

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.

Yield Strength (ksi):	50.0
Modulus of Elasticity, E (× 10^6 psi):	*29.3*
Coefficient of Thermal Expansion, α (x 10^{-6} in/in/°F)	*6.5*
Density (lbm/in^3)	*0.284*

15.3.2.2 ASTM A514 - CTS Header Plate [15-4]

Ultimate Strength (ksi):	110.0
Yield Strength (ksi):	100.0

15.3.2.3 ASTM A693/564, Type 630 - Lift Pin [15-3]

Ultimate Strength (ksi):	135.0
Yield Strength (ksi):	105.0
Modulus of Elasticity, E (× 10^6 psi)	28.5
Coefficient of Thermal Expansion, α (x 10^{-6} in/in/°F)	5.9
Density (lbm/in^3)	0.29

15.3.2.4 ASTM A516, Gr 70 - Canister Adapter Plate [15-3]

Ultimate Strength (ksi):	70.0
Yield Strength (ksi):	38.0
Modulus of Elasticity, E (× 10^6 psi):	29.2
Coefficient of Thermal Expansion, α (x 10^{-6} in/in/°F)	6.4
Density (lbm/in^3)	0.284

15.3.2.5 ASTM A574 - Canister Adapter Plate Bolts

The material properties for ASTM A574 are provided in Table 15-3.

15.3.2.6 ASTM A325 – Bolts [15-1]

Ultimate Strength (ksi):	120.0
Yield Strength (ksi):	92.0

15.3.2.7 ASTM A311, Class B – Pins [15-2]

Ultimate Strength (ksi):	170.0
Yield Strength (ksi):	135.0

15.3.2.8 ASTM A572, Grade 50 [*15-3*]

Ultimate Strength (ksi):	65.0
Yield Strength (ksi):	50.0

All Indicated Changes are in response to RAI NP-15-10-S

15.4 References

15-1 ASME NOG-1-2010, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)," The American Society of Mechanical Engineers, 2010.

15-2 ASTM A311/A311M – 04 (Reapproved 2010), "Standard Specification for Cold-Drawn, Stress-Relieved Carbon Steel Bars Subject to Mechanical Property Requirements," ASTM International, West Conshohocken, Pennsylvania.

15-3 "Structural and Thermal Material Properties – MAGNASTOR/MAGNATRAN Cask System," NAC Calculation 71160-2101 Rev. 9, NAC International, Atlanta, Georgia.

15-4 ASTM A514/A514M – 05 (Reapproved 2009) "Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding," West Conshohocken, PA, 2009.

15-5 ANSI N14.6-1993 American National Standard for Radioactive Materials – "Special Lifting Devices for Shipping Containers Weighing 10,000 Pounds (4500 kg) or More," 1993.

15-6 *ASME Boiler and Pressure Vessel Code, Section II, Material Specifications, Part D – Properties, 2001 Edition.*

All Indicated Changes are in response to RAI NP-15-10-S

Table **15-3**
Material Properties for ASTM A574

Temp (°F)	*Yield Strength[1] (ksi)*	*Ultimate Strength[2] (ksi)*
100	*135*	*170*
300	*126*	*170*

Notes:

1. *The yield strength is taken from Reference [15-6] Table Y-1, pages 536-538.*
2. *The ultimate strength is taken from Reference [15-6] Table U, pages 428-429.*

All Indicated Changes are in response to RAI NP-15-10-S

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. <u>The use of surrogate inspections identified in SAR Sections C.13.3.1 and C.13.4.1</u>: 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.

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 31 through 49
Withheld Pursuant to 10 CFR 2.390

A.13.1 Aging Management Review

The Aging Management Review (AMR) of the MP187 Cask system contained in the application for the renewal of the Rancho Seco Independent Spent Fuel Storage Installation (ISFSI), License SNM-2510 [A.13-1] provides an assessment of aging effects that could adversely affect the ability of in-scope structures, systems, and components (SSC) to perform their intended functions beyond the initial 20 year storage period. Aging effects, and the mechanisms that cause them, were evaluated for the combinations of materials and environments identified for the subcomponent of the in-scope SSCs based on a review of relevant technical literature, available industry operating experience (OE), and AREVA Inc. OE. Aging effects that could adversely affect the ability of the in-scope SSC to perform their safety function(s) require additional aging management activity to address potential degradation that may occur during the period of extended operation. The TLAAs and AMPs that are credited with managing aging effects during the period of extended operation are discussed in Sections A.13.2 and A.13.3, respectively.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

A.13.2 Time Limited Aging Analyses

The AMR of the MP187 Cask system in Reference [A.13-1] identified the following two TLAAs associated with the management of aging effect:

A. Fatigue Evaluation of the DSCs

This TLAA evaluated the DSC pressure boundary subcomponents for pressure and temperature fluctuations in accordance with the provisions of NB 3222.4(d) of the ASME B&PV Code, Section III, Division 1, 1992 Edition, with Addenda through 1993. As provided by NB 3222.4(d) of the ASME B&PV Code, fatigue effects need not be specifically evaluated provided the six criteria in NB 3222.4(d) are met. Reference [A.13-1] describes an evaluation performed considering a 60-year service life using maximum bounding initial DSC pressures and temperatures (at the beginning of storage). The evaluation showed that the six criteria of NB 3222.4(d) are met. The fatigue evaluation was revisited in Section C.13.2 to address the actions and loadings associated with the transportation of the DSC from the original ISFSI to the ISP/WCS CISF facility.

B. Boron Depletion

Reference [A.13-1] describes an analysis performed to determine the amount of boron depletion in the FO and FC DSC poison plates for a total of 100 years of storage. Over a period of 100 years, the evaluation considers a bounding neutron irradiation rate and indicates that the amount of B-10 depleted is negligible.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

A.13.3 Aging Management Programs

The aging management programs (AMPs) described in Appendix C, Section C.13.3 are applicable to the SSCs of the MP187 Cask system.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

A.13.4 References

A.13-1 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

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

B.13.1 Aging Management Review

The Aging Management Review (AMR) of the Standardize Advanced NUHOM® System contained in the application for the renewal of the CoC 1029 [B.13-1] provides an assessment of aging effects that could adversely affect the ability of in-scope structures, systems, and components (SSC) to perform their intended functions beyond the initial 20 year storage period. Aging effects, and the mechanisms that cause them, were evaluated for the combinations of materials and environments identified for the subcomponent of the in-scope SSCs based on a review of Managing Aging Process in Storage (MAPS) Report [B.13-2]. Aging effects that could adversely affect the ability of the in-scope SSC to perform their safety function(s) require additional aging management activity to address potential degradation that may occur during the period of extended operation. The TLAAs and AMPs that are credited with managing aging effects during the period of extended operation are discussed in Sections B.13.2 and B.13.3, respectively.

All Indicated Changes are in response to RAI NP-15-13-S

B.13.2 Time Limited Aging Analyses

The renewal submittal for CoC 1029 [B.13-1] describes the comprehensive review performed to identify the TLAAs for the in-scope SSCs of the Standardized Advanced NUHOMS® System to determine the analyses that could be credited with managing aging effects over the extended storage period. That review identified the following TLAAs associated with the 24PT1 DSC or AHSM:

- *Boron depletion in the BORAL® plates in the 24PT1 dry shielded canisters (DSCs)*
- *Fatigue analyses for the 24PT1 DSC shells*
- *Irradiation embrittlement of metals in the 24PT1 DSC*
- *Irradiation effects on the concrete in the Advanced Horizontal Storage Modules (AHSM)*
- *Establishment of cladding temperature limits for fuel stored in the 24PT1 DSC*

The identified TLAAs were dispositioned by demonstrating that the pre-renewal analysis remains valid for the period of extended operation or the analysis was updated. Of the above identified TLAAs it was determined that only the following TLAAs did not bound the period of extended operation and this were updated:

- *Fatigue analyses for the 24PT1 DSC shells*
- *Irradiation embrittlement of metals in the 24PT1 DSC*
- *Irradiation effects on the concrete in the Advanced Horizontal Storage Modules (AHSM)*

A. Fatigue Evaluation of the DSCs

This TLAA evaluated the DSC shell for pressure and temperature fluctuations in accordance with the provisions of NB 3222.4(d) of the ASME B&PV Code, Section III, Division 1, 1992 Edition, with Addenda through 1994. As provided by NB 3222.4(d) of the ASME B&PV Code, fatigue effects need not be specifically evaluated provided the six criteria in NB 3222.4(d) are met. Reference [B.13-1] describes an evaluation performed considering a 100-year service life using maximum bounding initial DSC pressures and temperatures (at the beginning of storage). The evaluation showed that the six criteria of NB 3222.4(d) are met. The fatigue evaluation was revisited in Section C.13.2 to address the actions and loadings associated with the transportation of the DSC from the original ISFSI to the ISP/WCS CISF facility.

B. Irradiation embrittlement of metals in 24PT1 and Irradiation effects on the concrete in the AHSM

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

Reference [B.13-1] describes an analysis performed that demonstrates that the irradiation embrittlement of metals in the 24PT1 DSCs and the irradiation effects on the concrete in the AHSM will not lead to a loss of intended safety functions for a 100-year service life.

B.13.3 Aging Management Programs

The aging management programs (AMPs) described in Appendix C, Section C.13.3 are applicable to the SSCs of the Standardized Advanced NUHOMS® system.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

B.13.4 References

B.13-1 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.

B.13-2 NRC NUREG-2214, "Managing Aging Process in Storage (MAPS) Report" (draft report for comment, October 2017) (ML17289A237).

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

C.13.1 Aging Management Review

The Aging Management Review (AMR) of the 61BT system contained in the application for the renewal of CoC 1004 [C.13-29] provides an assessment of aging effects that could adversely affect the ability of in-scope structures, systems and components (SSC) to perform their intended functions beyond the initial 20 year storage period. Aging effects, and the mechanisms that cause them, were evaluated for the combinations of materials and environments identified for the subcomponent of the in-scope SSCs based on a review of relevant technical literature, available industry operating experience (OE), and AREVA Inc. OE. Aging effects that could adversely affect the ability of the in-scope SSC to perform their safety function(s) require additional aging management activity to address potential degradation that may occur during the period of extended operation. The TLAAs and AMPs that are credited with managing aging effects during the period of extended operation are discussed in Sections C.13.2 and C.13.3, respectively.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

C.13.2 Time Limited Aging Analyses

The AMR of the 61BT system in Reference [C.13-29] identified the following TLAAs associated with the management of aging effect. Note that Reference [C.13-29] also contains supplemental evaluations in support of the aging management reviews and to provide defense-in-depth analyses. Since these supplemental evaluations do not meet the definition of a TLAA, they are not summarized below.

A. Boron Depletion

This TLAA demonstrates that the ability of the poison plates to maintain sub-criticality remains unaffected over the period of extended operation. The TLAA determines the amount of B-10 depleted in the poison plates due to neutron irradiation during 100 years of storage. The evaluation considers a bounding neutron irradiation rate with the least amount of B-10 content available in the poison plates and computes the reaction rate density. The evaluation shows that the depleted amount of B-10 is not more than 0.001% of the initial concentration of B-10.

B. Evaluation of Neutron Fluence and Gamma Radiation on Storage System Structural Materials

This TLAA evaluates the effect of neutron and gamma radiation on DSC and HSM structural materials for a storage period of 100 years.

Bounding sources and bounding MCNP models are used in order to envelop all of the HSM and DSC configurations. The evaluation takes credit for source strength decay over a total service (storage) life of 100 years and the energy deposition is integrated over the same period. The calculated neutron fluence on the DSC shell assembly is 4.12×10^{15} *neutrons/cm*^{2}*. The maximum neutron fluence in the basket assembly center fuel compartments is* 1.22×10^{16} *neutrons/cm*^{2}*. These levels are well below the level of concern for embrittlement of steels of* 10^{18} *neutrons/cm*^{2}*.*

The neutron fluence and gamma exposure are below the threshold levels of concern for the DSC and HSM materials. Therefore, no degradation due to radiation effects is expected and radiation is not an applicable aging mechanism.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

C. Fatigue Evaluation of the DSCs

This TLAA evaluated the DSC pressure boundary subcomponents for pressure and temperature fluctuations in accordance with the provisions of NB 3222.4(d) of the ASME B&PV Code, Section III, Division 1. As provided by NB 3222.4(d) of the ASME B&PV Code, fatigue effects need not be specifically evaluated provided the six criteria in NB 3222.4(d) are met. Reference [C.13-29] describes an evaluation performed considering a 100-year service life using maximum bounding initial DSC pressures and temperatures (at the beginning of storage). The evaluation showed that the six criteria of NB 3222.4(d) are met. The fatigue evaluation was revised to address the actions and loadings associated with the transportation of the DSC from the original ISFSI to the ISP/WCS CISF facility. The revised fatigue evaluation is summarized below and bounds operation and transport of the FO, FC, FF, 24PT1, 61BT, and 61BTH Type 1 DSCs.

Criterion #1 – Atmospheric to Service Pressure Cycle

The specified number of times (including startup and shutdown) that the pressure will be cycled from atmospheric pressure to service pressure and back to atmospheric pressure during normal service does not exceed the number of cycles on the applicable fatigue curve corresponding to an S_a value of three times the S_m value for the material at service temperature.

The reduction in the service pressure, or initial pressure condition, back to an ambient pressure, occurs very slowly over the life of the canister due to eventual cooling of the DSC. The reduction in the service pressure, or initial pressure condition, back to an ambient pressure, occurs very slowly over the life of the canister due to eventual cooling of the DSC. This reduction represents one full cycle of pressure change. The transfer/transportation loads do not cause any additional DSC pressure fluctuations between atmospheric and service pressure. It is apparent that one cycle does not exceed the number of cycles on the design fatigue curve for austenitic steels corresponding to the value of $S_a = (3)(20) = 60$ ksi, which is 1.37×10^4, where the value of $S_m = 20$ ksi at 70 °F is used conservatively. Therefore, the first criterion is satisfied.

Criterion #2 – Normal Service Pressure Fluctuation

The specified full range of pressure fluctuations during normal service does not exceed the quantity 1/3 × Design Pressure × (S_a/S_m), where S_a is the value obtained from the applicable design fatigue curve for the total specified number of significant pressure fluctuations and S_m is the allowable stress intensity for the material at service temperature. Significant pressure fluctuations are those for which the total excursion exceeds the quantity: Design Pressure × 1/3 × (S/S_m), where S is the value of S_a obtained from the applicable design fatigue curve for 10^6 cycles.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

Significant pressure fluctuations in the DSC may occur due to the seasonal ambient temperature changes. Based on a bounding temperature fluctuation from -40 °F to 125 °F, there are no significant pressure fluctuations due to the seasonal ambient temperature changes. Transfer/transportation loads do not cause additional seasonal ambient temperature changes. Therefore, the second criterion is satisfied.

Criterion #3 – Temperature Difference – Startup and Shutdown

The temperature difference between any two adjacent points of the component during normal service does not exceed $S_a/2E\alpha$, where S_a is the value obtained from the applicable design fatigue curves for the specified number of startup-shutdown cycles, α is the value of the instantaneous coefficient of thermal expansion, and E is the mean value of the temperatures at the two points.

For an operational cycle of the DSC, thermal gradients occur gradually during fuel loading and transfer to the HSM. The startup-shutdown thermal cycle begins to reverse after the DSC is loaded into the HSM and the DSC gradually begins to cool over the life of the DSC. This normal operational cycle occurs only once for the service life. The transfer/transportation loads cause one additional cycle due to removal from the existing HSM and loading into the HSM at the ISP/WCS CISF facility. Even for 10 cycles, the value of S_a is 708 ksi. Using conservative values to give an upper bound on the product of E and α (at 500 °F), the temperature difference limit ΔT_a is calculated as:

$\Delta T_a = S_a/2E\alpha = 708/[(2)(25{,}900)(10.5\times10^{-6})] = 1301.7$ °F

This value is much greater than the maximum temperature of the DSCs and consequently much greater than the difference between any two adjacent points on the DSCs. Therefore, the third criterion is satisfied.

Criterion #4 – Temperature Difference – Normal Service

The temperature difference between any two adjacent points does not change during normal service by more than the quantity $S_a/2E\alpha$, where S_a is the value obtained from the applicable design fatigue curve for the total specified number of significant temperature difference fluctuations. A temperature difference fluctuation shall be considered to be significant if its total algebraic range exceeds the quantity $S/2E\alpha$, where S is the value of S_a obtained from the applicable design fatigue curve for 10^6 cycles; i.e., 28.3 ksi.

If conservative values of E and α (at 500 °F) are used to give an upper bound on the product, a significant temperature difference fluctuation exceeds the range of:

$\Delta T_s = S/2E\alpha = 28.3/[(2)(25{,}900)(10.5\times10^{-6})] = 52.0$ °F

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

Small fluctuations in the DSC thermal gradients during normal storage in the HSM occur as a result of seasonal ambient temperature changes. Ambient temperature cycles significant enough to cause a measurable thermal gradient fluctuation are considered to occur five times per year, although this cycle can really occur only once a year going from winter to summer conditions. Therefore, considering five cycles per year is conservative, which results in 100 years × 5 cycles/year = 500 cycles, where 100 years is the bounding service life.

The only additional significant fluctuation in temperature difference is caused during transfer and transportation, which shows that the maximum fluctuation in the maximum DSC temperature is 65 °F in the case of the 61BTH Type 1 DSC. The fluctuation in the temperature difference between adjacent points will be smaller. The fluctuation in the temperature difference under storage conditions is also less than 65 °F. Therefore, when the temperature difference fluctuations due to seasonal ambient temperature changes and transfer/transportation conditions are combined, it is conservative to consider 1000 cycles and a temperature difference fluctuation of 65 °F. At 1000 cycles, the S_a value on the design fatigue curve is 119 ksi and the resulting allowable change in temperature difference between adjacent points is:

$$\Delta T_a = S_a/2E\alpha = 119/[(2)(25{,}900)(10.5\times10^{-6})] = 219\ °F$$

This value is much greater than 65 °F. Therefore, the fourth criterion is satisfied.

Criterion #5 – Temperature Difference – Dissimilar Materials

For components fabricated from materials of differing moduli of elasticity or coefficients of thermal expansion, the total algebraic range of temperature fluctuation experienced by the component during normal service does not exceed the magnitude $S_a/2(E1\alpha1 - E2\alpha2)$, *where* S_a *is the value obtained from the applicable design fatigue curve for the total specified number of significant temperature fluctuations, E1 and E2 are the moduli of elasticity, and* $\alpha1$ *and* $\alpha2$ *are the values of the instantaneous coefficients of thermal expansion at the mean temperature value involved for the two materials of construction.*

All integrally attached components of the DSCs are made of stainless steel material with the same mechanical and structural properties. Therefore, the fifth criterion is satisfied by the DSCs.

Criterion #6 – Mechanical Loads

The specified full range of mechanical loads, excluding pressure, does not result in load stresses whose range exceeds the S_a value obtained from the applicable design fatigue curve for the total specified number of significant load fluctuations. A load fluctuation shall be considered to be significant if the total excursion of load stress exceeds the quantity S, where S is the value of S_a obtained from the applicable design fatigue curve for 10^6 cycles if the total specified number of service cycles is 10^6 cycles or less. If the total specified number of service cycles exceeds 10^6 cycles, S is the value of S_a obtained from the applicable design fatigue curve for the maximum number of cycles defined on the curve.

Storage Loads:

The only significant mechanical loads for the DSC for the storage conditions are those associated with handling operations. The primary plus secondary stress intensity range is required to be less than $3S_m$ for Service Levels A and B. This evaluation conservatively considers 2000 cycles of handling loads for the bounding service life of 100 years.

Transportation Loads:

The 2000 cycles of handling loads considered for storage conditions is conservative and is considered to include the additional handling loads associated with the transfer/transportation loads. The only other significant normal mechanical loads for the DSC caused by the transfer/transportation loads are the loads during transportation, which include vibration and shock loads. The DSC stresses resulting from these mechanical loads are relatively small since the structural capacity of the DSC is designed for an extreme accident load such as a postulated cask drop. In this evaluation, the stresses due to the vibration and shock loads are estimated using the stress analysis results for the NCT (Normal Conditions of Transport) loading condition, for which 25g side drop (vertical acceleration) and 30g end drop (longitudinal acceleration) are considered. Since a side drop causes the DSC to be in contact with the shipping cask rails, the resulting stresses would be greater than the stresses due to the transverse acceleration. Therefore, the results for the NCT condition are applicable for the accelerations in all three directions, vertical, longitudinal, and transverse.

The primary plus secondary stress intensity range is required to be less than $3S_m$ for the NCT conditions (Service Level A). The allowable stresses are conservatively calculated to be 3(20) = 60 ksi, using the value of S_m at 70 °F.

In addition, the analyses for the NCT cases are based on linear elastic material properties of the steel components. Therefore, the results from the NCT analyses can be used to estimate stresses due to a different level of inertial loading.

Vibration Loads:

According to ANSI N14.23 [C.13-31], the peak vibration load for truck transport is 0.3g longitudinal, 0.3g transverse, and 0.6g vertical. NUREG 766510 [C.13-32] specifies a peak vibration load of 0.19g longitudinal, 0.19g transverse, and 0.37g vertical for rail car transport. Consequently, the inertial loading for truck transport is bounding. This inertial loading represents a fraction of the NCT loading condition where 25g side drop (vertical) and 30g end drop (longitudinal) accelerations are considered. Since the primary plus secondary stress intensity range is required to be less than $3S_m$ = 60 ksi for NCT, the maximum stress due to vibration is estimated as (0.6g/25g) (60 ksi) = 1.44 ksi. This stress is considerably lower than the significant stress S of 28.3 ksi, by an order of magnitude. Therefore, vibration load during transportation does not cause any significant load fluctuations.

Shock Loads:

According to ANSI N14.23 [C.13-32], the peak shock load for truck transport is 2.3g longitudinal, 1.6g lateral, 3.5g vertical up, and 2.0g vertical down. NUREG 766510 [C.13-32] specifies a peak shock loading of 4.7g in all directions for rail car transport. Consequently, the inertial loading caused by a railcar shock is bounding. This inertial loading represents a fraction of the NCT loading condition where 25g side drop (vertical) and 30g end drop (longitudinal) accelerations are considered. Since the primary plus secondary stress intensity range is required to be less than $3S_m$ = 60 ksi for NCT, the maximum stress due to shock is estimated as (4.7g/25g) (60 ksi) = 11.3 ksi. This stress is lower than the significant stress S of 28.3 ksi, but the difference is relatively small. Therefore, the severity of the cycling loading is examined below.

NUREG 766510 [C.13-32] reports that there are roughly 9 shock cycles per 100 miles of rail car transport. If the maximum trip distance for transportation is conservatively taken as 5,000 miles, the number of cycles is:

5,000 miles × 0.09 shocks / mile = 450 cycles

Combining Storage and Transportation Loads:

Based on computations in Storage Loads and Transportation loads the total number of significant load fluctuations is 2000 + 450 = 2450. From the applicable fatigue design curve, the S_a value corresponding to 2500 cycles is S_a = 91.4 ksi. This value is greater than the $3S_m$ value of 60 ksi. Therefore, the sixth criterion is satisfied.

Conclusion:

The above evaluation shows that all six criteria in NB 3222.4(d) are satisfied for the DSCs. Therefore, as provided by NB 3222.4(d), fatigue effects need not be specifically evaluated.

All Indicated Changes are in response to RAI NP-15-13-S

C.13.3 Aging Management Programs

C.13.3.1 Purpose

This chapter describes the aging management programs (AMPs) credited for managing each of the identified aging effects for the in-scope structures, systems, and components (SSCs) of the NUHOMS® related dry storage systems at the Waste Control Specialists (WCS) Consolidated Interim Storage Facility (CISF). The purpose of the AMPs is to ensure that aging effects do not result in a loss of intended function of the SSCs. The AMPs are based on the results of the aging management reviews (AMR) for the dry shielded canisters (DSCs), horizontal storage modules (HSMs), and concrete basemat presented in [C.13-29].

The AMPs developed to manage aging effects are:

- DSC External Surfaces Aging Management Program (applicable to DSC)
- DSC Aging Management Program for the Effects of Chloride-Induced Stress Corrosion Cracking (applicable to DSC)
- Horizontal Storage Module Aging Management Program for External and Internal Surfaces (applicable to HSM and DSC support structure)

In this chapter, the terms, DSC and HSM are used in a generic sense, and are intended to apply to the various types of DSCs, and HSMs used in the NUHOMS® related dry storage systems.

C.13.3.2 Methodology

The AMPs are based on the AMPs approved for the renewal of CoC 1004 [C.13-29 and C.13-30]. The structure of the AMPs is consistent with the 10 program elements described in NUREG-1927 [C.13-1], as follows:

1. Scope of the program: The scope of the program includes the specific SSCs and subcomponents subject to the AMP and the intended safety functions to be maintained. In addition, the element states the specific materials, environments, and aging mechanisms and effects to be managed.

2. Preventive actions: Preventive actions used to prevent aging or mitigate the rates of aging for SSCs.

3. Parameters monitored or inspected: This element identifies the specific parameters that will be monitored or inspected and describes how those parameters will be capable of identifying degradation or potential degradation before there is a loss of intended safety function.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

Environments

DSC shell assembly components subject to AMR are exposed to the following environments:

- Sheltered

Aging Effects Requiring Management

The following aging effects associated with the DSC shell assembly components require management:

- Loss of material due to crevice and pitting corrosion for stainless steel components
- Loss of material due to galvanic corrosion for the DSC shell contacting graphite lubricant at the sliding rail surface
- Loss of material due to radiation-induced crevice corrosion, pitting corrosion, and stress corrosion cracking (SCC) for stainless steel DSC shell
- Cracking due to SCC for stainless steel components when exposed to moisture and aggressive chemicals in the environment.

Preventive Actions

The program is a condition-monitoring program that does not include preventive actions.

Parameters Monitored or Inspected

The DSC External Surfaces AMP consists of visual inspections to monitor for material degradation of the DSC shell assembly.

- DSC surfaces, welds and HAZs, and crevice locations near the DSC support rails are inspected for discontinuities and imperfections.

Parameters Monitored or Inspected for Identified Aging Effects

Aging Effect	Aging Mechanism	Parameter(s) Monitored
Loss of Material	Crevice Corrosion	Surface Condition
Loss of Material	Pitting Corrosion	Surface Condition
Loss of Material	Galvanic Corrosion	Surface Condition
Cracking	Stress Corrosion Cracking	Surface Condition, Cracks

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

- Cracking due to CISCC for stainless steel components when exposed to moisture and aggressive chemicals in a coastal location, near salted roads, or in the path of effluent downwind from the cooling tower(s),
- Loss of material due to radiation-induced crevice corrosion, pitting corrosion, and SCC for stainless steel DSC shell.

Preventive Actions

The program is a condition-monitoring program that does not include preventive actions.

Parameters Monitored or Inspected

The surface monitoring portion of the AMP consists of collection and measurements of chloride salts on the surfaces of selected DSC shell(s) on different positions of the DSC shell surface to get a representation of the spatial variation of the chloride concentration, if any. The surface chloride concentration data are monitored, correlated, and trended, and compared to NDE results to monitor for CISCC initiation threshold.

The visual inspection portion of the AMP consists of visual inspections to monitor for material degradation of the DSC shell assembly.

- DSC surfaces, welds and HAZs, and crevice locations near the DSC support rails are visually inspected for discontinuities and imperfections.

Parameters Monitored or Inspected for Identified Aging Effects

Aging Effect	Aging Mechanism	Parameter(s) Monitored
Loss of Material	Crevice Corrosion	Surface Condition
Loss of Material	Pitting Corrosion	Surface Condition
Loss of Material	Galvanic Corrosion	Surface Condition
Cracking	Stress Corrosion Cracking	Surface Condition, Cracks

Detection of Aging Effects

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 the selection of a bounding DSC(s):

1. Time in service: Storage duration (time in service at originating ISFSI and WCS CISF) is related to surface temperature and deposition of contaminants. The DSC(s) selected for inspection is from the pool of DSCs with longest time in service.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

C.13-29 Letter E-46190 from Jayant Bondre (AREVA Inc.) to Document Control Desk (NRC), "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).

C.13-30 Letter from Meraj Rahimi (NRC) to Jayant Bondre (TN Americas LLC), "Renewal of Initial Certificate and Amendments Nos. 1 through 11and 13 Revision 1 and Amendment No. 14 of Certificate of Compliance No. 1004 for the Standardized NUHOMS® Horizontal Modular Storage System," December 4, 2017, (ADAMS Accession Number ML17338A092).

C.13-31 ANSI N14.23, "Design Basis for Resistance to Shock and Vibration of Radioactive Material Packages Greater than One Ton in Truck Transport," 1980.

C.13-32 NUREG 766510, "Shock and Vibration Environments for Large Shipping Containers on Rail Cars and Trucks."

DRAFT

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

D.13.1 Aging Management Review

The Aging Management Review (AMR) of the 61BTH System contained in the application for the renewal of the CoC 1004 [D.13-1] provides an assessment of aging effects that could adversely affect the ability of in-scope structures, systems, and components (SSC) to perform their intended functions beyond the initial 20 year storage period. Aging effects, and the mechanisms that cause them, were evaluated for the combinations of materials and environments identified for the subcomponent of the in-scope SSCs based on a review of relevant technical literature, available industry operating experience (OE), and AREVA Inc. OE. Aging effects that could adversely affect the ability of the in-scope SSC to perform their safety function(s) require additional aging management activity to address potential degradation that may occur during the period of extended operation. The TLAAs and AMPs that are credited with managing aging effects during the period of extended operation are discussed in Sections D.13.2 and D.13.3, respectively.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

D.13.2 Time Limited Aging Analyses

The renewal submittal for CoC 1004 [D.13-1] describes the comprehensive review performed to identify the TLAAs for the in-scope SSCs of the 61BTH System to determine the analyses that could be credited with managing aging effects over the extended storage period. The TLAAs described in Appendix C, Section C.13.2 are applicable to the 61BTH system.

D.13.3 Aging Management Programs

The aging management programs (AMPs) described in Appendix C, Section C.13.3 are applicable to the SSCs of the 61BTH system.

This section is newly added.

All Indicated Changes are in response to RAI NP-15-13-S

D.13.4 References

D.13-1 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).

All Indicated Changes are in response to RAI NP-15-13-S

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.

E.3.1.1.2 Water Level (Flood) Design

The NAC-MPC may be exposed to a flood during storage on an unsheltered concrete storage pad at an ISFSI site. The source and magnitude of the probably maximum flood depend on several variables. The NAC-MPC is evaluated for a maximum flood water depth of 50 feet above the base of the storage cask. The flood water velocity is considered to be 15 feet per section.

As documented in Sections 2.4.2.2 and 3.2.2, the WCS CISF is not in a floodplain and is above the Probable Maximum Flood elevation and, therefore, will remain dry in the event of a flood.

E.3.1.1.3 Seismic Design

The NAC-MPC may be exposed to a seismic event (earthquake) during storage on *the storage pad.* The only significant effect of a seismic event on an NAC-MPC would be a possible tip-over; however, tip-over does not occur in the evaluated design basis earthquake.

Section 7.6.3 demonstrates that the MAGNASTOR system, which bounds the NAC-MPC system, experiences minimal sliding (maximum 1.20 inches) and does not tip-over in the design basis earthquake.

E.3.1.1.4 Snow and Ice Loadings

The criteria for determining design snow loads is based on ANSI/ASCE 7-93, Section 7.0. The NAC-MPC is assumed to have a site location typical for siting Category C, which is defined to be "locations in which snow removal by wind cannot be relied on to reduce roof loads because of terrain, higher structures, or several trees near by." Ground snow loads for the contiguous United States are given in Figures, 5, 6 and 7 of ANSI/ASCE 7-93. A worst-case value of 100 pounds per square foot was assumed. Section 2.2.4 of Reference E.3-1 demonstrates the snow load is bounded by the weight of the loaded transfer cask. The snow load is also considered in the load combinations described in Section 3.4.4.2.2 of Reference E.3-1. Therefore, no further site-specific evaluations are required.

E.3.1.1.5 Combined Load Criteria

Each normal, off-normal and accident condition has a combination of load cases that defines the total combined loading for that condition. The individual load cases considered include thermal, seismic, external and internal pressure, missile impacts, drops, snow and ice loads, and/or flood water forces. The load conditions to be evaluated for storage casks are identified in 10 CFR 72 and in the "Design Criteria for an Independent Spent Fuel Storage Installation (Dry Storage Type)" (ANSI/ANS 57.9 – 1992).

All Indicated Changes are in response to RAI NP-E-1

E.3.2.1 Design Criteria for Environmental Conditions and Natural Phenomena

The design criteria defined in this section identifies the site environmental conditions and natural phenomena to which the storage system could reasonably be exposed during the period of storage. Analyses to demonstrate that the NAC-MPC design meets these design criteria are presented in the relevant chapters of Reference E.3-1.

E.3.2.1.1 Tornado and Wind Loadings

The tornado and wind loadings design criteria that are defined in Section 2.2 of Reference E.3-1 for the NAC-MPC apply to the MPC-LACBWR system in their entirety. These design criteria are described in WCS CISF SAR Appendix E, Section E.3.1.1.1. Therefore, no further site-specific evaluations are required.

E.3.2.1.2 Water Level (Flood) Design

The water level (flood) design criteria that are defined in Section 2.2 of Reference E.3-1 for the NAC-MPC apply to the MPC-LACBWR system in their entirety. These design criteria are described in WCS CISF SAR Appendix E, Section E.3.1.1.2. As documented in Sections 2.4.2.2 and 3.2.2, the WCS CISF is not in a floodplain and is above the Probable Maximum Flood elevation and, therefore, will remain dry in the event of a flood.

E.3.2.1.3 Seismic Design

The MPC-LACBWR may be exposed to a seismic event (earthquake) during storage on *the storage* pad. The only significant effect of a seismic event on an MPC-LACBWR would be a possible tip-over; however, tip-over does not occur in the evaluated design basis earthquake.

Section 7.6.3 demonstrates that the MAGNASTOR system, which bounds the NAC-LACBWR system, experiences minimal sliding (maximum 1.20 inches) and does not tip-over in the design basis earthquake.

E.3.2.1.4 Snow and Ice Loadings

The snow and ice loadings design criteria that are defined in Section 2.2 of Reference E.3-1 for the NAC-MPC apply to the MPC-LACBWR system in their entirety. These design criteria are described in WCS CISF SAR Appendix E, Section E.3.1.1.4. Therefore, no further site-specific evaluations are required.

E.3.2.1.5 Combined Load Criteria

The combined load design criteria that are defined in Section 2.2 of Reference E.3-1 for the NAC-MPC apply to the MPC-LACBWR system in their entirety. These design criteria are described in WCS CISF SAR Appendix E, Section E.3.1.1.5. Therefore, no further site-specific evaluations are required.

All Indicated Changes are in response to RAI NP-E-1

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.

Table E.3-1
Summary of WCS CISF Principal Design Criteria
(5 pages)

Design Parameter	WCS CISF Design Criteria	Condition	NAC-MPC Design Criteria
Seismic (Ground Motion)	Site-specific ground-surface uniform hazard response spectra (UHRS) with 1E-4 annual frequency of exceedance (AFE) having peak ground acceleration (PGA) of 0.250 g horizontal and 0.175 g vertical. (Table 1-5 and Figure 1-5)	Accident (Bounded)	***The NAC-MPC (Yankee-MPC, CY-MPC, and MPC-LACBWR) system is bounded by the MAGNASTOR seismic evaluation. See Evaluations in Section 7.6.1***
Vent Blockage	For MPC Systems: Inlet and outlet vents blocked 24 hrs	Accident (Same)	**Yankee-MPC, NAC-MPC FSAR Section 11.2.8.4** **CY-MPC, NAC-MPC FSAR Section 11.2.8.4** **MPC-LACBWR, NAC-MPC FSAR Section 11.2.8.4** Inlet and outlet vents blocked: 24 hrs
Fire/Explosion	For MPC Systems: Equivalent fire 50 gallons of diesel fuel	Accident (Same)	**NAC-MPC FSAR Section 11.2.5** Equivalent fire 50 gallons of diesel fuel
Cask Drop	For MPC Systems: Drop height 6 inches	Accident (Same)	**NAC-MPC FSAR Section 11.2.11.2** **(MPC-LACBWR) NAC-MPC FSAR Section 11.A.2.11.2** Drop height 6 inches
Ambient Temperatures	Yearly average temperature 67.1°F	Normal (Bounded)	**NAC-MPC FSAR Section 2.2.6** Average Annual Ambient Temperature 75°F
Off-Normal Temperature	Minimum 3 day avg. temperature 27.9°F Maximum 3 day avg. temperature 89.4°F	Off-Normal (Bounded)	**NAC-MPC FSAR Section 2.2.6** Minimum 3 day avg. temperature -40°F Maximum 3 day avg. temperature 100°F
Extreme Temperature	Maximum temperature 113°F	Accident (Bounded)	**NAC-MPC FSAR Section 2.2.6** Maximum temperature 125°F
Solar Load (Insolation)	Horizontal flat surface insolation 2949.4 BTU/day-ft^2 Curved surface solar insolation 1474.7 BTU/day-ft^2	Normal (Same)	**Yankee-MPC, NAC-MPC FSAR Section 4.4.1.1.2** **CY-MPC, NAC-MPC FSAR Section 4.5.1.1** **MPC-LACBWR, NAC-MPC FSAR Section 4.A.3.1.1** Curved Surface: 1475 Btu/ft^2 for a 24-hour period. Flat Horizontal Surface: 2950 Btu/ft^2 for a 24-hour period.

All Indicated Changes are in response to RAI NP-F-1

Table F.3-1
Summary of WCS CISF Principal Design Criteria
(4 pages)

Design Parameter	WCS CISF Design Criteria	Condition	NAC-UMS® Design Criteria
Seismic (Ground Motion)	Site-specific ground-surface uniform hazard response spectra (UHRS) with 1E-4 annual frequency of exceedance (AFE) having peak ground acceleration (PGA) of 0.250 g horizontal and 0.175 g vertical. (Table 1-5 and Figure 1-5)	Accident (Bounded)	*The NAC-UMS system is bounded by the MAGNASTOR seismic evaluation. See Evaluations in Section 7.6.1.*
Vent Blockage	For UMS Systems: Inlet and outlet vents blocked 24 hrs	Accident (Same)	**NAC-UMS FSAR Section 11.2.13.3** Inlet and outlet vents blocked: 24 hrs
Fire/Explosion	For UMS Systems: Equivalent fire 50 gallons of diesel fuel	Accident (Same)	**NAC-UMS FSAR Section 11.2.6.1** Equivalent fire 50 gallons of flammable fluid
Cask Drop	For UMS Systems: VCC's Drop height 24 inches	Accident (Same)	**NAC-UMS FSAR Section 11.2.4** VCCs for UMS Systems: Drop height 24 inches
Ambient Temperatures	Yearly average temperature 67.1°F	Normal (Bounded)	**NAC-UMS FSAR Section 2.2.6** Average Annual Ambient Temperature 76°F
Off-Normal Temperature	Minimum 3 day avg. temperature 27.9°F Maximum 3 day avg. temperature 89.4°F	Off-Normal (Bounded)	**NAC-UMS FSAR Section 2.2.6** Minimum 3 day avg. temperature -40°F Maximum 3 day avg. temperature 106°F
Extreme Temperature	Maximum temperature 113°F	Accident (Bounded)	**NAC-UMS FSAR Section 2.2.6** Maximum temperature 133°F

All Indicated Changes are in response to RAI NP-F-1

Table G.3-1
Summary of WCS CISF Principal Design Criteria
(3 pages)

Design Parameter	WCS CISF Design Criteria	Condition	MAGNASTOR® Design Criteria
Type of fuel	Commercial, light water reactor spent fuel	Normal (Bounded)	MAGNASTOR FSAR Section 2.2
Storage Systems	Transportable canisters and storage overpacks docketed by the NRC	Normal (Bounded)	72-1031 71-9356 (Pending)
Fuel Characteristics	Criteria as specified in previously approved licenses for included systems	Normal (Bounded)	MAGNASTOR FSAR Section 2.2
Tornado (Wind Load)	Max translational speed: 40 mph Max rotational speed: 160 mph Max tornado wind speed: 200 mph Radius of max rotational speed: 150 ft Tornado pressure drop: 0.9 psi Rate of pressure drop: 0.4 psi/sec	Accident (Bounded)	**MAGNASTOR FSAR Section 2.3.1.1** Max translational speed: 70 mph Max rotational speed: 290 mph Max tornado wind speed: 360 mph Radius of max rotational speed: 150 ft Tornado pressure drop: 3.0 psi Rate of pressure drop: 2.0 psi/sec
Tornado (Missile)	Automobile: 4000 lb, 112 ft/s (76.4 mph) Schedule 40 Pipe: 287 lb, 112 ft/s (76.4 mph) Solid Steel Sphere: 0.147 lb, 23 ft/s (15.7 mph)	Accident (Bounded)	**MAGNASTOR FSAR Section 2.3.1.3** Massive Missile: 4000 lb, 126 mph Rigid hardened steel: 280 lb, 126 mph Solid Steel Sphere: 0.15 lb, 126 mph
Floods	The WCS CISF is not in a floodplain and is above the Probable Maximum Flood elevation and will remain dry in the event of a flood.	Accident (Bounded)	**MAGNASTOR FSAR Section 2.3.2.1** Flood height: 50 ft Water velocity: 15 ft/s
Seismic (Ground Motion)	Site-specific ground-surface uniform hazard response spectra (UHRS) with 1E-4 annual frequency of exceedance (AFE) having peak ground acceleration (PGA) of 0.250 g horizontal and 0.175 g vertical. (Table 1-5 and Figure 1-5)	Accident (Bounded)	*See Evaluations in Section 7.6.1.*

All Indicated Changes are in response to RAI NP-F-1

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.

F.3.1.1.2 Water Level (Flood) Design

The NAC-UMS may be exposed to a flood during storage on an unsheltered concrete storage pad at an ISFSI site. The source and magnitude of the probable maximum flood depend on several variables. The NAC-UMS is evaluated for a maximum flood water depth of 50 feet above the base of the storage cask. The flood water velocity is considered to be 15 feet per second.

As documented in Sections 2.4.2.2 and 3.2.2, the WCS CISF is not in a floodplain and is above the Probable Maximum Flood elevation and therefore will remain dry in the event of a flood.

F.3.1.1.3 Seismic Design

The NAC-UMS may be subject to a seismic event (earthquake) during storage on *the storage pad.* The only significant effect of a seismic event on a NAC-UMS would be a possible tip-over of the cask or a collision of two casks due to sliding; however, neither tip-over nor sliding occurs in the evaluated design basis earthquake.

Section 7.6.3 demonstrates that the MAGNASTOR system, which bounds the NAC-UMS system, experiences minimal sliding (maximum 1.20 inches) and does not tip-over in the design basis earthquake.

F.3.1.1.4 Snow and Ice Loadings

The criteria for determining design snow loads is based on ANSI/ASCE 7-93, Section 7.0. The NAC-UMS is assumed to have a site location typical for siting Category C, which is defined to be "locations in which snow removal by wind cannot be relied on to reduce roof loads because of terrain, higher structures, or several trees nearby." Ground snow loads for the contiguous United States are given in Figures, 5, 6 and 7 of ANSI/ASCE 7-93. A worst case value of 100 pounds per square foot was assumed. Section 2.2.4 of Reference F.3-1 demonstrates that the snow load is bounded by the weight of the loaded transfer cask.

The snow load is also considered in the load combinations described in Section 3.4.4.2.2 of Reference F.3-1. Therefore, no further site-specific evaluations are required.

All Indicated Changes are in response to RAI NP-F-2

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.

G.3.1.1.1 Tornado Missiles and Wind Loadings

The concrete casks are typically placed outdoors on an unsheltered reinforced concrete storage pad at an ISFSI site. This storage condition exposes the casks to tornado and wind loading. The design basis tornado and wind loading is defined based on Regulatory Guide 1.76 Region 1 and NUREG-0800. The design basis tornado missile impacts are defined in Paragraph 4, Subsection III, Section 3.5.1.4 of NUREG 0800. Analyses presented in Reference G.3-1, Section 3.7.3.2 and discussed in Reference G.3-1, Section 12.2.11 demonstrates that the MAGNASTOR design meets these criteria. Therefore, no further site-specific evaluations are required.

G.3.1.1.2 Water Level (Flood) Design

The loaded concrete cask may be exposed to a flood during storage on an unsheltered concrete storage pad at an ISFSI site. The source and magnitude of the probable maximum flood depend on specific site characteristics. The MAGNASTOR concrete cask design basis is a maximum floodwater depth of 50 feet above the base of the cask and a floodwater velocity of 15 ft per second.

As documented in Sections 2.4.2.2 and 3.2.2, the WCS CISF is not in a floodplain and is above the Probable Maximum Flood elevation and, therefore, will remain dry in the event of a flood.

G.3.1.1.3 Seismic Design

The WCC CISF may be subject to seismic events (earthquakes) during its lifetime. The possible significant effect of a beyond-design-basis seismic event on the concrete cask would be a tip-over; however, the loaded concrete cask does not tip over during the design-basis seismic event. Although it is a nonmechanistic event, the loaded concrete cask design basis includes consideration of the consequences of a hypothetical cask tip-over event.

Section 7.6.3 demonstrates that the MAGNASTOR system, experiences minimal sliding (maximum 1.20 inches) and does not tip-over in the design basis earthquake.

Additionally, to evaluate concrete cask stress the evaluation in Section 3.7.3.4 of Reference G.3-1 conservatively applies seismic loads of 0.5g in the horizontal direction and 0.5g in the vertical direction. These accelerations reflect a more rigorous seismic loading and, therefore, bound the design basis earthquake event. These compressive stresses are used in the load combinations for the concrete cask discussed in Section G.3.1.1.5, and the combined stress results meet stress criteria for the accident events.

All Indicated Changes are in response to RAI NP-G-1

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.

<u>Methodology:</u>

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-2.6-3-1 displays the locations of the historical borings provided.

<u>Stratigraphy Development:</u>

- 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]).

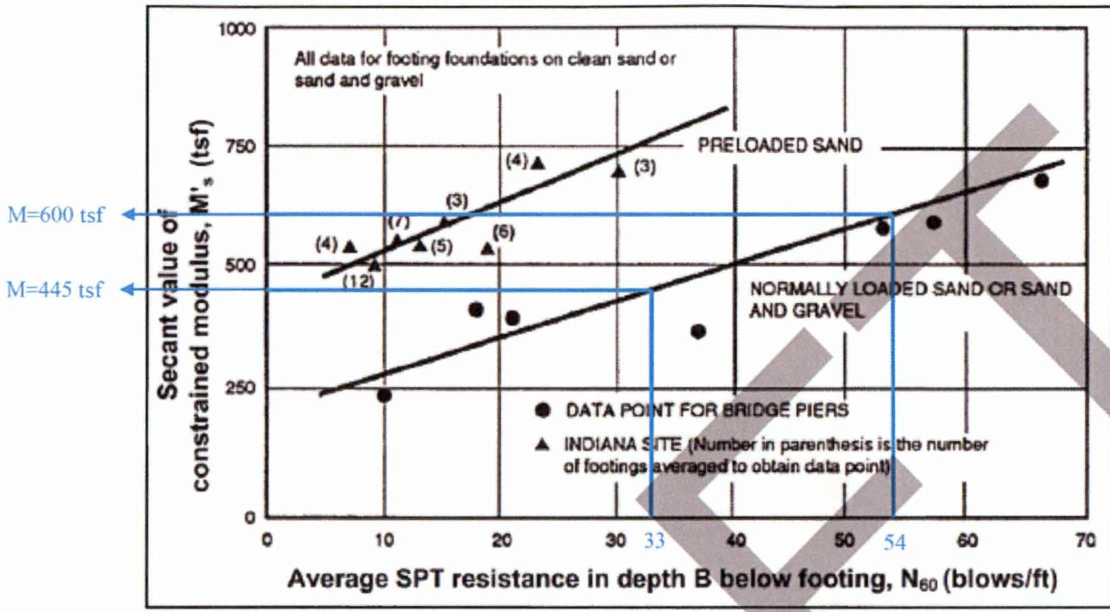

Figure P-2.6-3-1
Graphical Representation of Constrained Modulus to NPT N-Values from Reference [1]

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.

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.

Results:

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.

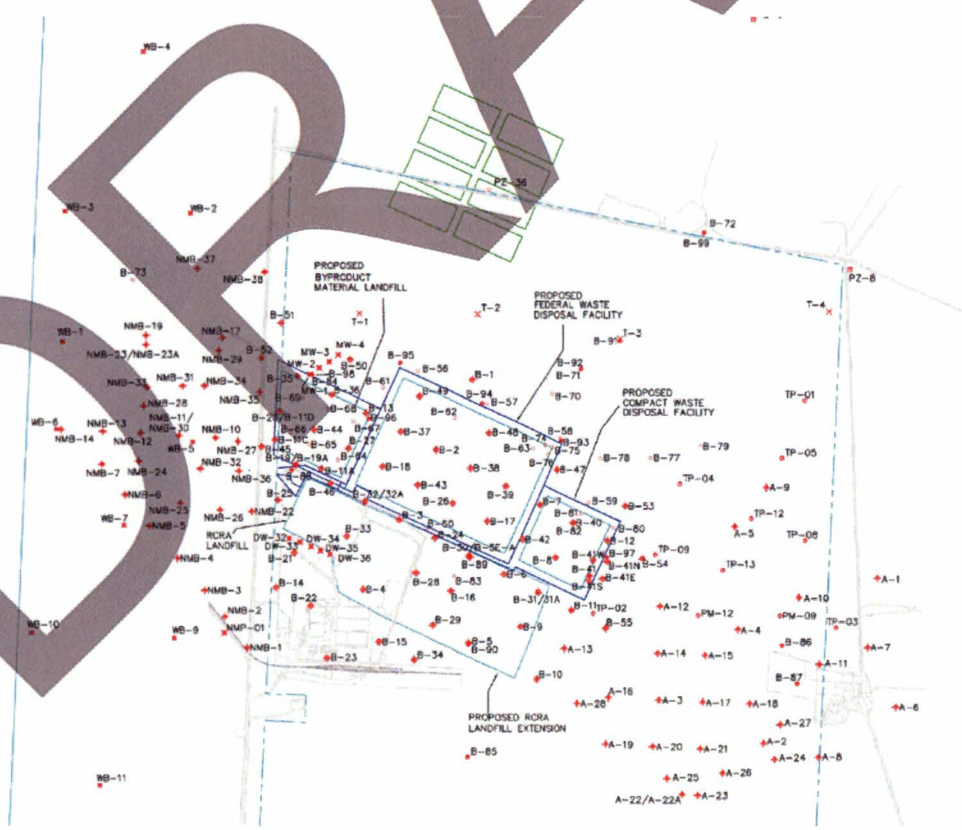

Figure P-2.6-3-1
Historical Borings at WCS Site

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.

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

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.

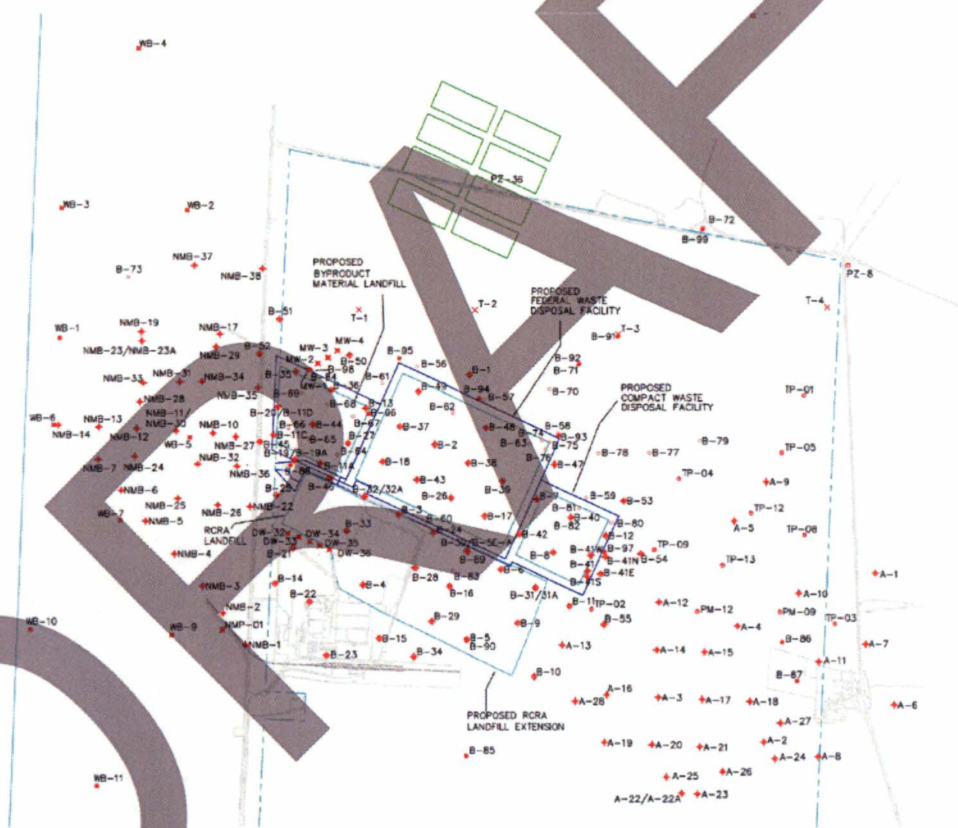

Figure NP-2.6-3-1
Historical Borings at WCS Site

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.