ENERGYSOLUTIONS

February 1, 2023 CD-2023-025

Mr. Doug Hansen, Director
Division of Waste Management and Radiation Control
P.O. Box 144880
Salt Lake City, UT 84114-4880

Re: Responses to Federal Cell Facility Application Request for Information - DRC-2022-023940

Dear Mr. Hansen,

Energy*Solutions* hereby responds to the Utah Division of Waste Management and Radiation Control's December 19, 2022 Request for Information (RFI) on our Federal Cell Facility Application.[1] A response is provided for each request using the Director's assigned reference number. A revised copy of Appendix D, *Geotechnical Seismic Engineering Evaluations of the FCF* and associated references reflecting responses to the Director's request are attached. This revised Appendix is not subject to the Permanent Claim of Business Confidentiality previously asserted.[2]

<u>**Appendix O: Erosion Modeling**</u>

O-2: After downloading SIBERIA from the public website, it did not compile, it may be because it has not been revised for modern architecture. The Division requests that EnergySolutions please provide: (1) Information pertaining to the operating system on which the SIBERIA code was run, (2) Information pertaining to the complier used to compile the SIBERIA source code, (3) SIBERIA compiled version of the code currently being run to support Clive DU PA v2.0, and (4) SIBERIA source code currently being run to support Clive DU PA v2.0. These will greatly expedite our review of the erosion modeling:

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Energy*Solutions* is developing information in response to Request O-4 and will submit it to the director under separate cover.

O-5: NUREG/CR-7200 discusses how a SIBERIA model is calibrated using regressions of beta1, m1, and n1 values. Please describe quantitatively how the SIBERIA model was calibrated to measured data for the Clive DU PA v2.0:

Energy*Solutions* is developing information in response to Request O-5 and will submit it to the director under separate cover.

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Energy*Solutions* is developing information in response to Request O-6 and will submit it to the director under separate cover.

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Energy*Solutions* is developing information in response to Request O-7 and will submit it to the director under separate cover.

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Energy*Solutions* is developing information in response to Request O-8 and will submit it to the director under separate cover.

Appendix D: Geotechnical and Seismic Engineering Evaluations

D-2: Evaluate Uncertainty in Engineering Properties. The geotechnical analyses presented in Appendix D as a basis for the proposed Federal Cell have evaluated expected conditions using engineering properties obtained during past geotechnical explorations at the site and from the literature. Geotechnical properties are inherently spatially variable, and the spatial variability will affect the outcomes of the analyses. Understanding the impact of spatial variability on geotechnical stability is necessary to evaluate the efficacy of the Federal Cell. The Division requests a quantitative evaluation of the sensitivity of each of the geotechnical analyses to uncertainty in the engineering properties by varying the engineering properties used in the analyses two standard deviations above and below the mean.:

To evaluate the uncertainty in engineering properties for geotechnical stability and account for spatial variability in the subsurface, Energy*Solutions* directed Geosyntec to perform a statistical analysis of data collected across the Clive Facility during past geotechnical explorations. The statistical analysis of the various geotechnical material properties for the subsurface units (Unit 1 through 4) relied on in situ measurements and observations and geotechnical laboratory testing results from samples collected during drilling for the following borings:

- B-1 & B-2 (AMEC, 2004);
- SC-1, -7, -8, -10 & SLC-84 (D&M, 1984);
- GW-16, -17, -18, -19A, -19B, -24, -27, -29, -36, -37, -38, -41, -55, DH-33, -48, -51 (Bingham Environmental, 1992); and
- DH-1 (AGRA, 1999).

These borings were selected based on their relative location to the Federal Cell and the availability of meaningful data (i.e., SPT blow counts, laboratory testing). Where robust laboratory testing was limited, the development of material properties for the statistical analysis relied on applicable empirical correlations published in literature.

In response to this request, a statistical evaluation of the engineering properties using mean ±2 standard deviations for sensitivity analyses is developed to consider the potential for underestimating the actual average value of the parameter due to the limited dataset analyzed, assess the potential for lower average values, and evaluate the sensitivity of the geotechnical analyses to these variable properties. A statistical evaluation of data using median and percentile values (or combining median and standard deviation) yields representative values for real physical data with limited number of data points, because median is the 50^{th} percentile data corresponding to an actual data point.

Mean central value estimates using ±2 standard deviations (which statistically captures 95% of the data within the 2.5^{th} and 97.5^{th} percentile range) are highly affected by the presence and number of very large or very small magnitude values in datasets and generally not representative of realistic conditions when conducting sensitivity analyses (i.e., produces negative values, significantly lower than physically reasonable minimum values, or not values uncharacteristic for associated soil types). By contrast, it is common geotechnical engineering practice to consider distributions based on central values ±1 standard deviations (which corresponds to 16^{th} and 84^{th} percentile - applicable to sensitivity analyses) in analysis of extreme conditions.

The use of ±1 standard deviation is more characteristic of the typical range of soil properties and the subsurface conditions across the Clive Facility, while still sufficiently conservative to run produce meaningful sensitivity analyses for the associated geotechnical evaluations (i.e., stability and settlement). Following development of the material property data set, each estimated value is plotted by depth and adjacent the median, ± 1 standard deviation, 33^{rd} percentile (or 66^{th} percentile for compressibility parameters), and the previously selected parameter value for the subsurface unit (Unit 1 through 4). The visual representation of the statistical analysis for each material property is presented on Figures 3 – 10 of the revised Report in Appendix D to the Application (see "GEOTECNICAL ENGINEERING EVALUATIONS FOR FEDERAL CELL AT THE CLIVE FACILITY – CLIVE, UTAH," dated revised on January 18, 2023). Discussion related to the statistical analysis is found in Sections 4.2.1 and 5.8 of Appendix D, with the associated slope stability and settlement sensitivity analyses results summarized in Section 4.8.1, 4.9.1, and 5.8 and Attachment B2 and D2 of the revised Report in Appendix D.

Additional liquefaction triggering analyses is also performed of the sand-like Unit 3 soils during a groundwater rise event to account for spatial variability beneath the proposed Federal Cell by performing the Idriss and Boulanger (2008) method with SPT-blow counts documented in boring logs GW-17A, -18, 19-A, -19B, -25, -26, -27, and -28 (Bingham Environmental, 1992). The previous analysis only included data from logs GW-36, -37, and -38 drilled directly beneath the proposed Federal Cell. The additional logs were selected based on proximity to the Federal Cell and availability of data (i.e., SPT blow counts, rig and sampler information for correction, groundwater elevation, etc.). Results of the extended liquefaction triggering analysis are documented in Section 6.3, Figure 11, and Attachment E1 of the revised Report in Appendix D. In addition to the extended liquefaction triggering analysis, the liquefied residual strength of Unit 3 was also analyzed for a post-earthquake slope stability analysis, documented in Section 4.12 and Attachment B of the revised Report in Appendix D.

Additional seismic slope stability or deformation analyses with lower bound sensitivity parameters do not inform understanding of the sensitivity for decision making. As presented in Section 4.2 of the revised Report in Appendix D, the shear strength parameters are conservative for stability and seismic analyses because the undrained shear strength of fine-grained soils increases as the waste is placed and the fine-grained soils consolidate. For example, the minimum effective stress on top of Unit 4 and Unit 2, fine-grained soils, will be approximately 6,300 and 7,900 pounds per square foot (psf) at final build-out and assuming only 90% consolidation takes place, which is anticipated to occur within 1 year of waste placement, prior to the design earthquake the preconsolidation pressures on top of these units would be 5,670 and 7,110 psf. Using SHANSEP's formulation for estimating shear strength of fine-grained soils, the undrained shear strength on top of these layers is estimated as 1,475 and 1,850 psf, respectively. These values are significantly greater than the undrained shear strength values, 1,000 and 1,500 psf, as summarized in Table 2-1 in the revised Report in Appendix D. Therefore, additional sensitivity analyses of seismic slope stability are unnecessary.

D-3: <u>Evaluate Static and Seismic Stability of Internal Slopes</u>. The geotechnical analyses in Appendix D have been conducted in the context of global stability using the build out geometry. Case histories have shown, however, that stability failures in waste containment systems often occur within internal slopes during operations (e.g., during filling). The potential for internal slope failures needs to be evaluated, and any vulnerable internal slope geometries identified. Please evaluate quantitatively the static stability of a range of likely scenarios for internal slopes. Identify critical internal slopes geometries, if any, that are prone to stability failure:

Based on planned waste placement activities and configuration of the proposed Federal Cell, the critical geometry for interim stability is the excavation into native soils prior to waste placement. Interim slope stability analyses for short-term (undrained strengths for clay-like soils) were performed to address item D-3. The analysis is summarized in Section 4.8.2 with supporting results provided in Attachment B3 of the revised Report in Appendix D. Since this analysis evaluates a temporary slope, seismic deformation is not evaluated. If a seismic event occurs during a temporary slope condition, deformation and resulting deficiencies will be corrected by Energy*Solutions* prior to continued construction of the Federal Cell.

D-4: <u>Evaluate Blow Counts Using Appropriate Hammer Correction Factor and Re-evaluate Geotechnical Analyses</u>. The standard penetration testing (SPT) hammer correction factor used to adjust the blow count data may not have been appropriate for the hammer used for the geotechnical exploration activities. Determine the type of hammer (specifically that of a rope and cathead or one using an automatic system) used for standard penetration testing in the past geotechnical exploration activities and the appropriate hammer correction factor to be used to adjust the blow counts for the hammer that was employed. If necessary, re-compute the blow counts used in the analyses and re-conduct the geotechnical analyses using blow counts updated with a revised hammer correction factor. In addition, if geotechnical parameters were developed from empirical relationships using SPT blow counts, confirm the appropriate SPT blow counts were utilized in developing those geotechnical parameters.:

As discussed in Section 4.2 of the revised Report in Appendix D, the material properties used in the analyses are based on review of available geotechnical lab data, boring logs, and previous parameterization of the adjacent Class A West. Therefore, those parameters are not strictly based on Standard Proctor Test (SPTD) blow counts. As part of the statistical analysis completed for Request Item D-2, SPT blow count data were collected for nearby borings:

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The SPT blow counts provided from these borings are used to estimate material properties, including friction angle, undrained shear strength, and effective cohesion using published empirical correlations with N-value, N_{60}, or $(N1)_{60}$. To do this, the appropriate information from the boring logs is used to correct SPT blow counts with the characteristic correction factors (i.e., hammer efficiency, borehole diameter, rod length, etc.). This data and the selected value of the analyses are provided in Figure 3 through 10 of the revised Report in Appendix D. It is noted that the selected values in the analyses typically fall below the median value for each of the parameters, therefore, Geosyntec did not identify a need to re-conduct the geotechnical analyses. To further support a conclusion that the sensitivity analyses are conservative when using ±1 standard deviation property values for slope stability and settlement, additional liquefaction triggering analyses for the sand-like Unit 3 soils, and post-earthquake stability analyses with residual strengths for Unit 4, Unit 3, and Unit 2 soils capture the potential for uncertainty and variability in the native soils' material parameterization.

Additional references reflected in these responses and the revisions to Appendix D are also attached.

If you have further questions regarding the responses to the director's requests of DRC-2022-023940 and revision of Appendix D to the Federal Cell Facility Radioactive Material License Application, please contact me at (801) 649-2000.

Sincerely,

Vern C. Rogers

Vern C. Rogers
Director, Regulatory Affairs

enclosure

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

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If you have further questions regarding the responses to the director's requests of DRC-2022-023940 and revision of Appendix D to the Federal Cell Facility Radioactive Material License Application, please contact me at (801) 649-2000.

Sincerely,

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I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

APPENDIX D

FEDERAL CELL FACILITY

GEOTECHNICAL AND SEISMIC ENGINEERING EVALUATIONS

Energy*Solutions'* Federal Cell Facility design is primarily an above-grade landfill embankment. The Federal Cell Facility will be constructed using materials native to the site or found near the site. Synthetic materials are also used in the construction of the mixed waste embankment. Engineered features of the embankments are designed based upon State of Utah regulations, NRC guidance, Environmental Protection Agency guidance, and Energy*Solutions'* experience at this location. UAC R313-25-23 requires principal design features to be selected for the Federal Cell Facility that promote long-term stability. The geotechnical stability of the Federal Cell Facility has been evaluated by Geosyntec (report presented in this Appendix).

Geosyntec consultants

215 South State Street, Suite 500
Salt Lake City, UT 84111
(801) 618-0483
www.geosyntec.com

Mr. Vern Rogers
Director of Regulatory Affairs
EnergySolutions, LLC
299 South Main Street, Suite 1700
Salt Lake City, UT 84111

Subject: **Response to DWMRC RFI (DRC-2002-024035) dated 19 December 2022**
Federal Cell at Clive Facility
Clive, Utah

Dear Vern,

Geosyntec Consultants (Geosyntec) has prepared this transmittal letter in response to the Request for Information (RFI) from the Division of Waste Management and Radiation Control (DWMRC) dated 19 December 2022 regarding the Federal Cell Facility Application dated 4 August 2022. The following sections of this letter provide Geosyntec's response to requests for Appendix D of the application. The requests for Appendix D are numbered as D-2, D-3, and D-4 in the RFI. Geosyntec provides each request and our response to each request below. We refer the reader to the appropriate section of the revised Appendix D, "Geotechnical Engineering Evaluations for Federal Cell at the Clive Facility," (Geosyntec, 2022) calculation package, where additional analyses are provided. The revised calculation package is appended to this letter.

GEOSYNTEC'S RESPONSE TO REQUEST FOR INFORMATION

DWMRC Request Item D-2:

"*Evaluate Uncertainty in Engineering Properties. The geotechnical analyses presented in Appendix D as a basis for the proposed Federal Cell have evaluated expected conditions using engineering properties obtained during past geotechnical explorations at the site and from the literature. Geotechnical properties are inherently spatially variable, and the spatial variability will affect the outcomes of the analyses. Understanding the impact of spatial variability on geotechnical stability is necessary to evaluate the efficacy of the Federal Cell. The Division requests a quantitative evaluation of the sensitivity of each of the geotechnical analyses to uncertainty in the engineering properties by varying the engineering properties used in the analyses two standard deviations above and below the mean.*"

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Geosyntec Response to Item D-2:

To evaluate the uncertainty in engineering properties for geotechnical stability and account for spatial variability in the subsurface, Geosyntec performed a statistical analysis of the existing data collected across the Clive Facility during past geotechnical explorations. The statistical analysis of the various geotechnical material properties for the subsurface units (Unit 1 through 4) relied on in situ measurements and observations and geotechnical laboratory testing results from samples collected during drilling for the following borings:

- B-1 & B-2 (AMEC, 2004);
- SC-1, -7, -8, -10 & SLC-84 (D&M, 1984);
- GW-16, -17, -18, -19A, -19B, -24, -27, -29, -36, -37, -38, -41, -55, DH-33, -48, -51 (Bingham Environmental, 1992); and
- DH-1 (AGRA, 1999).

These borings were selected based on their relative location to the Federal Cell and the availability of meaningful data (i.e., SPT blow counts, laboratory testing). Where robust laboratory testing was limited, the development of material properties for the statistical analysis relied on applicable empirical correlations published in literature.

RFI Item D-2 requests a statistical evaluation of the engineering properties using mean ±2 standard deviations for sensitivity analyses. The purpose of statistically evaluating the engineering properties used for geotechnical evaluations is to consider the potential for underestimating the actual average value of the parameter due to the limited dataset analyzed, assess the potential for lower average values, and evaluate the sensitivity of the geotechnical analyses to these variable properties. The statistical evaluation of data can be done by using mean and standard deviation terms. However, statistical analyses using median and percentile values (or combining median and standard deviation) generally yield more realistic values for real physical data with limited number of data points because median is the 50th percentile data corresponding to an actual data point whereas mean is affected by the presence and number of very large or very small magnitude values in the dataset that may not be realistic. It is common in geotechnical engineering practice to consider a 33rd percentile data point as the lower bound or conservative estimate for the average value of the parameter. It is also common to consider mean (or median) ±1 standard deviation which corresponds to 16th and 84th percentile for extreme condition analyses which can be considered applicable to a sensitivity analysis. The use of a range corresponding to ±2 standard deviations statistically captures 95% of the data within the 2.5th and 97.5th percentile range. Considering mean -2 standard deviation for estimating the lower bound average value of a parameter for a sensitivity analysis is not realistic in our opinion. Geosyntec checked the +2 standard deviations over median for several of the parameters. Due to the large value of the standard deviation, ±2 standard deviations did not represent meaningful parameter values for the subsequent engineering evaluations and was not relevant to the data set (i.e., the value was negative in value, significantly lower than the minimum value, or not characteristic of the soil type).

The use of ±1 standard deviation was more characteristic of the typical range of soil property values and our understanding of the subsurface conditions across the site, while still conservative enough to run meaningful sensitivity analyses for the associated geotechnical evaluations (i.e., stability and settlement). Following development of the material property data set, each estimated value was plotted by depth and adjacent the median, ± 1 standard deviation, 33rd percentile (or 66th percentile for compressibility parameters), and the previously selected parameter value for the subsurface unit (Unit 1 through 4). The visual representation of the statistical analysis for each material property is presented on **Figures 3 – 10** of the revised calculation package appended to this letter. Discussion related to the statistical analysis can be found in **Sections 4.2.1 and 5.8**, with the associated slope stability and settlement sensitivity analyses results summarized in **Section 4.8.1, 4.9.1, and 5.8** and **Attachment B2 and D2** of the revised package.

Geosyntec performed additional liquefaction triggering analyses of the sand-like Unit 3 soils during a groundwater rise event to account for spatial variability beneath the proposed cell by performing the Idriss and Boulanger (2008) method with SPT-blow counts documented in boring logs GW-17A, -18, 19-A, -19B, -25, -26, -27, and -28 (Bingham Environmental, 1992). The previous analysis only included data from logs GW-36, -37, and -38 drilled directly beneath the proposed Federal Cell. The additional logs were selected based on proximity to the Federal Cell and availability of data (i.e., SPT blow counts, rig and sampler information for correction, groundwater elevation, etc.). Results of the extended liquefaction triggering analysis are documented in **Section 6.3, Figure 11, and Attachment E1** of the revised package. In addition to the extended liquefaction triggering analysis, Geosyntec estimated the liquefied residual strength of Unit 3 for a post-earthquake slope stability analysis, documented in **Section 4.12 and Attachment B** of the revised package.

Geosyntec did not identify the need to conduct additional seismic slope stability or deformation analyses with lower bound sensitivity parameters resulting from the data statistics. As discussed in Section 4.2 of our report, the shear strength parameters used are considered conservative because the undrained shear strength of fine-grained soils will increase as the waste is placed and the fine-grained soils consolidate. These parameters are especially conservative for a long-term seismic analysis. For example, the minimum effective stress on top of Unit 4 and Unit 2, fine-grained soils, will be approximately 6300 and 7900 psf at final build-out and assuming only 90% consolidation takes place, which is anticipated to occur within 1 year of waste placement, prior to the design earthquake the preconsolidation pressures on top of these units would be 5,670 and 7,110 psf. Using SHANSEP's formulation for estimating shear strength of fine-grained soils, the undrained shear strength on top of these layers is estimated as 1,475 and 1,850 psf, respectively. These values are significantly greater than the undrained shear strength values, 1,000 and 1,500 psf, used in our analyses as summarized in Table 2-1 in our report. Therefore, additional sensitivity analyses of seismic slope stability are not considered necessary

<u>DWMRC Request Item D-3:</u>

"*Evaluate Static and Seismic Stability of Internal Slopes***.** *The geotechnical analyses in Appendix D have been conducted in the context of global stability using the build out geometry. Case histories have shown, however, that stability failures in waste containment systems often occur within internal slopes during operations (e.g., during filling). The potential for internal slope failures needs to be evaluated, and any vulnerable internal slope geometries identified. Please evaluate quantitatively the static stability of a range of likely scenarios for internal slopes. Identify critical internal slopes geometries, if any, that are prone to stability failure.*"

<u>Geosyntec Response to Item D-3:</u>

Based on conversations with EnergySolutions regarding their waste placement activities and configuration of the proposed Federal Cell, the critical geometry for interim stability was identified as the excavation into native soils prior to waste placement. Interim slope stability analyses for short-term (undrained strengths for clay-like soils) were performed to address this RFI item. The analysis is summarized in **Section 4.8.2** with supporting results provided in **Attachment B3** of the revised calculation package. Since this is a temporary slope condition, seismic deformation is not typically evaluated. In the event that a seismic event occurs during the temporary slope condition, deformation and resulting deficiencies shall be corrected prior to continued construction of the cell.

<u>DWMRC Request Item D-4:</u>

"*Evaluate Blow Counts Using Appropriate Hammer Correction Factor and Re-evaluate Geotechnical Analyses. The standard penetration testing (SPT) hammer correction factor used to adjust the blow count data may not have been appropriate for the hammer used for the geotechnical exploration activities. Determine the type of hammer (specifically that of a rope and cathead or one using an automatic system) used for standard penetration testing in the past geotechnical exploration activities and the appropriate hammer correction factor to be used to* *adjust the blow counts for the hammer that was employed. If necessary, re-compute the blow counts used in the analyses and re-conduct the geotechnical analyses using blow counts updated with a revised hammer correction factor. In addition, if geotechnical parameters were developed from empirical relationships using SPT blow counts, confirm the appropriate SPT blow counts were utilized in developing those geotechnical parameters.*"

Geosyntec Response to Item D-4:

As discussed in Section 4.2 of our report, the material properties used in our analyses were based on our review of available geotechnical lab data, boring logs, and previous parameterization of the adjacent CAW performed. Therefore, those parameters were not strictly based on SPT blow counts. As part of the statistical analysis completed for RFI Item D-2, Geosyntec gathered all SPT blow count data from the following nearby borings:

- B-1 & B-2 (AMEC, 2004); and
- GW-16, -17, -18, -19A, -19B, -24, -27, -29, -36, -37, and -38 (Bingham Environmental, 1992).

The SPT blow counts provided from these borings were used to estimate material properties, including friction angle, undrained shear strength, and effective cohesion through the use of published empirical correlations with N-value, N_{60}, or $(N_1)_{60}$. To do this, Geosyntec used appropriate information from the boring logs to correct SPT blow counts with the characteristic correction factors (i.e., hammer efficiency, borehole diameter, rod length, etc.). This data and the selected value of our analyses are provided in Figure 3 through 10 of the revised report. We noted that the selected values in our analyses typically fall below the median value for each of the parameter, therefore, Geosyntec did not identify a need to re-conduct the geotechnical analyses. To further bolster this conclusion, the sensitivity analyses with conservative ±1 standard deviation property values for slope stability and settlement, additional liquefaction triggering analyses for the sand-like Unit 3 soils, and post-earthquake stability analyses with residual strengths for Unit 4, Unit 3, and Unit 2 soils capture the potential for uncertainty and variability in the native soils' material parameterization.

CLOSING

If you have any questions or require additional information regarding this submittal, please contact Madeline Downing at (650) 868-7913 or Keaton Botelho of Geosyntec at (858) 674-6559.

Madeline Downing
Engineer

Bora Baturay, Ph.D., P.E., G.E.
Principal

Keaton Botelho, P.E.
Principal

ATTACHMENTS:

Geotechnical Engineering Evaluations for the Federal Cell at the Clive Facility – Revision 2 (Geosyntec, 2023)

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **Energy Solutions** Project: **Federal Cell at Clive Facility** Project No.: **SLC1025**

Title of Computations **GEOTECHNICAL ENGINEERING EVALUATIONS**

Computations by:	Signature	Madeline Downing	3/11/2021 Date
	Printed Name	Madeline Downing	
	Title	Engineer	
Assumptions and Procedures Checked by: (peer reviewer)	Signature	Bora Baturay	3/17/2021 Date
	Printed Name	Bora Baturay, PhD, G.E.	
	Title	Principal	
Computations Checked by:	Signature	Bora Baturay	3/17/2021 Date
	Printed Name	Bora Baturay, PhD, G.E.	
	Title	Principal	
Computations backchecked by: (originator)	Signature	Madeline Downing	3/18/2021 Date
	Printed Name	Madeline Downing	
	Title	Engineer	
Approved by: (pm or designate)	Signature	KB	3/18/2021 Date
	Printed Name	Keaton Botelho, P.E.	
	Title	Senior Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval
1	ALL	10/7/22	MD	MD	MD
2	ALL	1/18/23	MD	BB	KB

Written by:	M. Downing	Date:	3/11/2021	Reviewed by:	B. Baturay	Date:	3/17/21
Client:	ES	Project:	Federal Cell	Project/ Proposal No.:	SLC1025	Task No.:	01

GEOTECHNICAL ENGINEERING EVALUATIONS FOR FEDERAL CELL AT THE CLIVE FACILITY CLIVE, UTAH

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1. OBJECTIVE

The objective of this analysis is to evaluate the geotechnical engineering mechanisms related to the performance of the proposed Federal Cell at the EnergySolutions, LLC (EnergySolutions) Clive Facility in Clive, Utah. The geotechnical analyses performed for the Federal Cell include static and seismic stability, foundational soil settlement, and liquefaction triggering for the proposed embankment. The evaluations presented herein have been based on conservative approaches to evaluate this facility and are designed to capture the potential long-term changes over the design life. The analyses were performed in accordance with our proposal dated February 17, 2021.

A Request for Information (RFI) from the Division of Waste Management and Radiation Control (DWMRC) regarding the Federal Cell Facility Application dated 4 August 2022 was submitted to EnergySolutions on 19 December 2022. Geosyntec has prepared this revised report (Revision 2) to address the requests for Appendix D (Item D2 through D4) of the application.

2. BACKGROUND

Based on our understanding of the Federal Cell design, the intended waste to be placed in the containment cell includes depleted uranium (DU) stored in cylinders and drums and controlled low strength material (CLSM); a flowable fill which will be placed in between and around the cylinders and drums. According to the Radioactive Waste Inventory for Clive DU PA Model v1.4 (Neptune, 2015b), approximately 690,000 metric tons of the DU filled drums and cylinders are intended to be placed in the proposed cell. Existing grades at the proposed cell location range between 4,268 and 4,270 feet above mean sea level (amsl). The Design Drawings (EnergySolutions, 2020) suggest the average subgrade elevation of the proposed cell is approximately 4,261 feet amsl, which would be achieved by excavating approximately 7 to 9 feet below ground surface (bgs).

To support the design of the proposed Federal Cell, EnergySolutions and Neptune and Company, Inc. (Neptune) developed the Final Report for the Clive Depleted Uranium Performance Assessment (DU PA) and the DU PA Model v1.4 in 2015 and submitted it to the Utah Division of Waste Management and Radiation Control (DWMRC) for review. The DWMRC provided a review of the DU PA and documented their feedback in their Technical Report dated January 28, 2021 (DWMRC, 2021). EnergySolutions requested that Geosyntec provide assistance to respond to DWRMC's feedback and demonstrate compliance with the performance objectives of the Utah Administrative Code (UAC) R313-25-19 through 23 and 10 Code of Federal Regulations (CFR)

61.41 through 44, specifically the geotechnical stability evaluations. Geosyntec performed a review of the referenced Technical Report and has subsequently completed the following engineering evaluations to help address the technical issues identified by the DWMRC:

- Global static slope stability of the proposed Federal Cell: Short- and long-term stability including analysis of the various groundwater elevation conditions (current and potential groundwater level rise);
- Seismic slope stability of the proposed Federal Cell: Pseudostatic stability and deformation analysis of the most critical stability section;
- Settlement of the proposed Federal Cell foundational soils: Immediate and long-term settlement analysis including evaluation of embankment response to foundation settlement over the design life; and
- Liquefaction: Liquefaction triggering analysis caused by potential rise in groundwater elevation.

3. SITE CHARACTERIZATION

The subsurface conditions and proposed Federal Cell liner and cover system components were characterized based on our review of existing explorations, previous parameterizations performed for adjacent existing waste cells, and available data provided for our review. The following sections summarize the documents reviewed, subsurface stratigraphy characterization, groundwater conditions, and seismic design parameters used to perform our engineering evaluations presented in this calculation package.

3.1 Document Review

Extensive subsurface explorations have taken place at the Clive Facility dating back to 1984 and extending through 2020 (**Figure 1** presents a site layout of the explorations used in this evaluation). The following reports provided to us for review were utilized to characterize the subsurface stratigraphy beneath the proposed Federal Cell, define the groundwater levels critical for the engineering evaluations, and define the seismic hazard parameters at the facility:

- Hydrogeologic Report for the Clive Facility prepared by Bingham Environmental (Bingham) dated 1992 (including Addendum 1 and 2);

- Combined Embankment Study for Class A Waste Embankment (CAW) (just North of the proposed Federal Cell) prepared by AMEC Earth & Environmental (AMEC) dated December 2005;
- Geotechnical Update Report for CAW prepared by AMEC dated February 2011;
- Seismic Hazard Evaluation/Seismic Stability Analysis Update for CAW prepared by AMEC dated April 2012; and
- Phase 1 Basal Depth Aquifer Study for Clive Facility prepared by Stantec Consulting Services, Inc. (Stantec) dated September 2020.

3.2 Subsurface Stratigraphy

Based on our review of the referenced Hydrogeologic Report (Bingham, 1992), three exploratory drill holes were excavated beneath the proposed Federal Cell in 1991 by Overland Drilling under the direction of Bingham personnel. Drill hole logs for GW-36 through GW-38 (**Attachment A**) were reviewed to develop a generalized subsurface stratigraphy beneath the proposed Federal Cell (Bingham, 1992). In general, the geologic units include the following from top to bottom:

- Unit 4 Silty Clay – silty clays, classifying as CL in accordance with Unified Soil Classification System (USCS), containing some fine silt layers and is generally dry near surface with increasing moisture with depth, and medium stiff to stiff consistency.
- Unit 3 Silty Sand – dense to medium dense silty sands and silts containing few thin clay layers.
- Unit 2 Silty Clay – interbedded clay and silt layers with a few isolated sand layers up to 2-feet thick, generally stiff, and saturated clays.
- Unit 1 Silty Sand with interbedded clay/silt lens – generally dense to very dense sands.

As mentioned previously, existing grades beneath the cell range between 4,268 to 4,270 feet above mean sea level (amsl). The Design Drawings (EnergySolutions, 2020) suggests the average subgrade elevation of the proposed cell is approximately 4,261 feet amsl. This will result in excavations ranging between 7 to 9 feet into native Unit 4. Minimal portions of the Unit 4 will therefore be left in the subgrade. We assume that soft spots of these silty clays will be reworked and compacted prior to construction of the Federal Cell clay liner. Conservatively we have assumed approximately 2 feet of Unit 4 silty clay with medium stiff consistency remains beneath the Federal Cell for the engineering evaluations presented herein. For the purposes of this calculation package, the subsurface geology and Federal Cell is idealized as shown in **Figure 2** below.

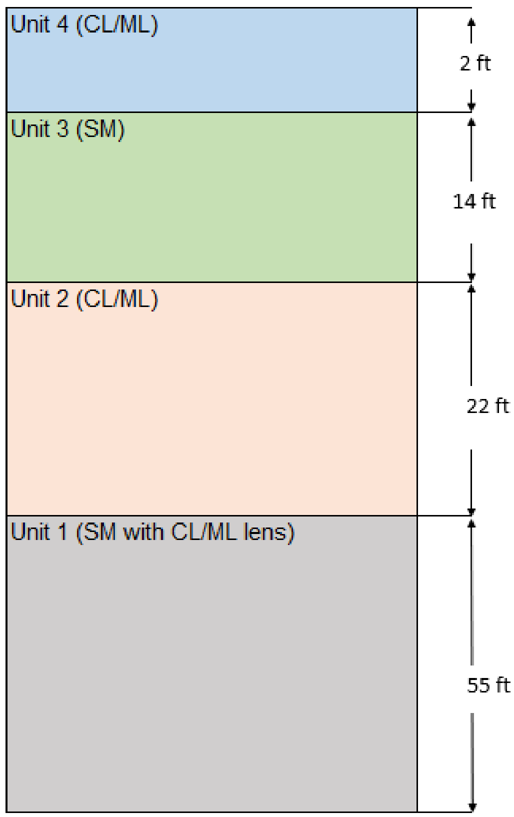


Figure 2 Subsurface Stratigraphy

The subsurface conditions beneath the Federal Cell and CAW embankment are generally consistent, with the exception of Unit 2 extending on average only 45 feet bgs as opposed to the approximated 64 feet bgs for the CAW. Conditions documented from various explorations are in general agreement with the hydrogeologic cross sections across the Clive Facility (**Attachment A**). The same geologic unit numbers used in the hydrogeologic characterization (Bingham, 1992) are used herein for consistency. The importance of this finding is the subsurface conditions are sufficiently uniform and therefore a single idealized profile is appropriate for the Federal Cell.

3.3 Groundwater

The latest static groundwater levels were collected during the referenced Aquifer Study (Stantec, 2020). Depth to water in wells I-1-30, I-1-50, I-1-100, and I-1-700 ranged between 28 to 31 feet. Groundwater depth reported on well logs GW-36 through GW-38 (used for subsurface stratigraphy characterization beneath the Federal Cell) was encountered at approximately 20 feet bgs. Groundwater records for these wells report a depth of approximately 20 feet between 2016 and 2020. A depth of 20 feet was therefore used to represent the existing conditions in our stability and settlement analyses.

Based on available historical records, no significant groundwater elevation rises have occurred at the Facility. However, DWMRC has requested that the proposed Federal Cell be evaluated for potential geotechnical instabilities over the design life caused by future hypothetical groundwater rise events. Therefore, we also evaluated a design groundwater level elevation synonymous with the ground surface elevation as a bounding scenario as requested by DWMRC. The extreme-case groundwater rise condition was used to evaluate liquefaction triggering and long-term stability of the proposed Federal Cell.

3.4 Seismic Hazard Evaluation

DMWRC accepted an updated assessment of the seismic hazard for the Clive Facility consistent with the requirements of the Utah Code of Regulations R313-25-8(5) to justify a 2012 licensing action (AMEC, 2012). The previously accepted seismic hazard analysis for the site was therefore used in this analysis. The seismic hazard assessment was based on deterministic assessment of the 84th percentile peak ground acceleration (PGA) associated with the Maximum Credible Earthquake (MCE) for known active and potentially active faults in the site region and the PGA obtained from a probabilistic seismic hazard analysis (PSHA) considering a 5,000-year return period to assess the seismic hazard for earthquakes that may occur on unknown faults in the area surrounding the site. The largest PGA from the assessment was 0.24g which was same for both deterministic and probabilistic methods. The maximum magnitude (Mw) identified was 7.3. Based on our review of the seismicmap.org tool created by Structural Engineers Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD) and a review of Unified Hazard Tool (UHT) by the US Geologic Survey (USGS), the PGA obtained using current fault and ground motion estimation models is 0.22g. Therefore, the seismic parameters previously accepted by DMWRC are considered reasonable estimates of the seismic hazard for the site and were utilized in Geosyntec's seismic hazard analyses documented in this package.

4. SLOPE STABILITY

The evaluation of global slope stability of the Federal Cell waste embankment was identified as an unresolved requirement in the referenced Technical Report (DWMRC, 2021). Analyses presented herein for global stability consider the geotechnical response of the site for the 10,000-year design life (or compliance period). Deep-seated global slope stability analyses were performed for both static and seismic conditions. In addition, the stability analyses include groundwater modeling at current conditions and at the existing ground surface that represents extreme case bounding future scenario in terms of pore pressures for stability. The following sections summarize the methods and analyses performed to demonstrate global static and seismic stability of the proposed Federal Cell. The graphical output files for the analyses are presented in **Attachment B, B2, and B3.**

4.1 Federal Waste Cell Geometry

Based on our review of the Design Drawings for the Federal Cell dated February 2021 (EnergySolutions, 2021), the proposed cell will retain the waste previously described in Section 2 with maximum side slopes of 20 percent (%). For slope stability analyses, the cell geometry has been summarized in Table 1 below.

Table 1: Summary of Federal Cell Design Dimensions

Description	Dimension and Unit
Length	1,920 feet
Width	1,225 feet
Height	52 ½ feet, maximum at crest
Base Elevation	4,262 to 4,263 feet
Crest Elevation	4,314.5 feet
Shoulder Side Slopes	20%
Shoulder Side Slope Width	175 feet
Shoulder Side Slope Height	32.5 feet

Description	Dimension and Unit
Cover Top Gradient	2.4%

4.2 Subsurface Material Properties

The material properties of the subsurface soils used to evaluate slope stability reflect our review of available geotechnical lab data, boring logs, and previous parameterization of the adjacent CAW performed and compiled for DWMRC's 2012 Class A West licensing decision (AMEC 2005 & 2011). The subsurface units are generally consistent beneath the CAW and the proposed Federal Cell, therefore, Geosyntec considers previous material property assignment of the units to be generally applicable for the analyses presented herein. Based on review of the geotechnical lab data summarized in 2005 (AMEC, 2005) and the DWMRC's 2012 licensing action, and the boring logs available within the Federal Cell footprint, Geosyntec made more conservative assumptions for the undrained shear strength of clay units. The undrained shear strengths test results reflect the in-situ conditions during the previous explorations. These selections are considered potentially conservative as consolidation of the underlying clay units are expected to occur during construction of the cell, resulting in strength gain overtime with pore pressure dissipation. The material properties for use in slope stability analyses are summarized in Table 2-1 below.

Table 2-1: Summary of Subsurface Material Properties for Slope Stability

Unit	Material Classification	Depth	Total Unit Weight, γ	Undrained	Drained	
				Undrained Shear Strength, Su	Friction Angle, ϕ'	Effective Cohesion, c'
		(ft-bgs)	(pcf)	(psf)	(deg)	(psf)
4	CL/ML	0 - 9	118	1,000	29	0
3	SM	9 - 23	120	-	34	0
2	CL-ML	23 - 45	121	1,500	29	1,000
1	SM with Interbedded thin lifts of CL-ML	45 - 100	120	-	29	0

4.2.1 Subsurface Material Properties – Statistical Analysis

A statistical analysis of the native soil material properties was performed in response to the DWMRC's Request for Information (RFI) dated 19 December 2022 Item D-2. To account for the inherent spatial variability of geotechnical properties, a more focused review of the available exploration data collected across the Clive Facility was performed to develop reasonable sensitivity ranges for each slope stability parameter based on data statistics. The statistical analysis relied on in situ measurements and observations and geotechnical laboratory testing results from samples collected during drilling for the following borings:

- B-1 & B-2 (AMEC, 2005);
- SC-1, -7, -8, -10 & SLC-84 (D&M, 1984);
- GW-16, -17, -18, -19A, -19B, -24, -27, -29, -36, -37, -38, -41, -55, DH-33, -48, -51 (Bingham Environmental, 1992); and
- DH-1 (AGRA, 1999).

These boring logs were selected based on proximity to the Federal Cell and the availability of meaningful data (i.e., SPT blow counts, drill rig information, laboratory testing). The logs and laboratory testing summary are provided in **Attachment A.** In the occurrence where robust laboratory testing was limited, the development of material properties for the statistical analysis relied on applicable empirical correlations published in literature.

The DWMRC RFI Item D-2 requests a statistical evaluation of the parameters and estimation of the parameters for mean ± standard deviations for sensitivity analyses. The objective of a standard statistical evaluation of data in geotechnical evaluations is to consider the potential for underestimating the actual average value of a parameter because of a limited dataset analyzed as part the project and to assess potential for presence of lower average strength zones and perform a sensitivity analysis. The statistical evaluation of data can be done by using mean and standard deviation terms. However, often, the statistical analysis using median and percentile values (or combining median and standard deviation) would yield more realistic values for real physical data with limited number of data points because median is the 50th percentile data corresponding to an actual data point, whereas mean is affected by the presence and number of very large or very small magnitude values in the dataset and may not be realistic. It is common in engineering practice to consider 33rd percentile data point as the lower bound or conservative estimate for the average value of the parameter. It is also common to consider mean (or median) ±1 standard deviation which corresponds to 16th and 84th percentile for extreme condition analyses which can be considered applicable to a sensitivity analysis. The use of a range corresponding to ± 2 standard deviations statistically captures 95% of the data within the range, 2.5th and 97.5th percentile. Considering mean minus two standard deviation for estimating the lower bound average value for a sensitivity analysis is not realistic in our opinion. Geosyntec checked the two-standard deviation above/below median for several of the parameters. Due to the large value of the standard deviation, ±2 standard deviations did not represent meaningful parameter values for the subsequent engineering evaluations and was not relevant to the data set (i.e., the value was negative in value, significantly lower than the minimum value, or not characteristic of the soil type).

The use of ±1 standard deviation was more characteristic of the typical range of soil property values and our understanding of the subsurface conditions across the site, while still conservative enough to run meaningful sensitivity analyses for the associated geotechnical evaluations (i.e., stability and settlement). Following development of each material property data set, each estimated value was plotted by subsurface elevation and adjacent the median, ± 1 standard deviation, 33rd percentile, and the previously selected parameter value for the subsurface unit (Unit 1 through 4). Results of the statistical analysis for material properties related to slope stability are shown on **Figure 3 through Figure 5.** The minus 1 standard deviation value was selected as the lower bound sensitivity value for slope stability; intended to capture the potential for spatial variability beneath the proposed Federal Cell that could impact its stable condition. One exception was made for undrained shear strength of Unit 4, as the -1 standard deviation value resulted in a negative value due to the large standard deviation value of the data set, thus the minimum value was selected for the sensitivity analysis. The material properties for use in the sensitivity analysis of slope stability are summarized in Table 2-2 below.

Table 2 - 2: Summary of Lower Bound Sensitivity Strength Properties for Slope Stability

Unit	Material Classification	Depth	Undrained	Drained	
			Undrained Shear Strength, Su	Friction Angle, ϕ'	Effective Cohesion, c'
		(ft-bgs)	(psf)	(deg)	(psf)
4	CL/ML	0 - 9	500	27	0
3	SM	9 - 23	-	31	0
2	CL-ML	23 - 45	750	29	80

Unit	Material Classification	Depth	Undrained	Drained	
			Undrained Shear Strength, Su	Friction Angle, ϕ'	Effective Cohesion, c'
		(ft-bgs)	(psf)	(deg)	(psf)
1	SM with Interbedded thin lifts of CL-ML	45 - 100	-	29	0

The following sections briefly summarizes the development of each material property data set for statistical analysis and subsequent sensitivity parameter selection for slope stability.

4.2.1.1 Friction Angle

Sand-like Soils in Unit 3 & 1

The effective stress friction angle (ϕ') for the sand-like soils in Unit 3 and 1 was estimated by selecting the minimum correlated value from the following four published empirical correlations with SPT blow counts:

- Hatanaka and Uchida (1996) in the Federal Highway Administration (FHWA, 2002)

$$\phi' = \sqrt{15.4 * (N_1)_{60}} + 20$$

- Schmertmann (1975)

$$\phi' = \tan^{-1}(N_{60}/(12.2 + 20.3 * \frac{\sigma_V'}{2116}))^{0.34}$$

- Peck (1953)

$$\phi' = 0.3 * N + 27$$

- Peck et. al. (1974)

$$\phi' = 27.1 + 0.3 * N_{60} + 0.00054({N_{60}}^2)$$

The Peck (1953) correlation resulted in the minimum friction angle value for all blow counts representing the Unit 3 and Unit 1 soils. **Figure 3** presents the estimated friction angle values plotted by subsurface elevation used to complete the statistical analysis and select lower bound -1 standard deviation sensitivity values.

Clay-like Soils in Unit 4 & 2

The effective stress friction angle for clay-like soils in Unit 4 and 2 was estimated by the following empirical correlation with plasticity index (PI) presented by Sorensen (2013):

$$\phi' = 45 - 14\log(PI)$$

Plasticity index testing results used to develop the friction angle data set for Unit 4 and 2 was based on laboratory testing data provided in **Attachment A**. **Figure 3** presents the estimated friction angle values plotted by subsurface elevation used to complete the statistical analysis and select lower bound -1 standard deviation sensitivity values.

4.2.1.2 Effective Cohesion

The effective cohesion (or drained cohesion, c') for the clay-like soils in Unit 4 and 2 was estimated by the following empirical correlation with undrained shear strength (Su) presented by Sorensen (2013):

$$c' = 0.2\,Su$$

Figure 4 presents the estimated effective cohesion values for Unit 4 and 2 clay-like soils plotted by subsurface elevation used to complete the statistical analysis and select lower bound -1 standard deviation sensitivity values.

4.2.1.3 Undrained Shear Strength

Due to the lack of direct laboratory testing of the undrained shear strength for the clay-like soils in Unit 4 and Unit 2, the undrained shear strength for the clay-like soils relied on three main bases summarized as follows:

- Limited vane shear testing performed on Unit 2 clay-like soils by AGRA (1999).
- SHANSEP equation used by AMEC (2005):

$$\frac{Su}{\sigma_v'} = m\ OCR^n$$

Where, the overconsolidation ratio (OCR) was based on limited consolidation data collected by D&M (1984), Bingham Environmental (1992), AGRA (1999), and AMEC (2004) and m & n based on lab testing of Bonneville Clay from various projects in the Salt Lake Valley.

- Correlations with corrected blow counts (N_{60}) presented in the MDT Geotechnical Manual (2008).

Figure 5 presents the resulting estimated undrained shear strength values plotted by subsurface elevation used to complete the statistical analysis and select lower bound -1 standard deviation sensitivity values. One exception was made for undrained shear strength of Unit 4, as the -1 standard deviation value resulted in a negative value due to the large standard deviation value of the data set, thus the minimum value was selected for the sensitivity analysis.

4.3 Federal Cell Cover and Base Liner System Material Properties

The material properties for the cover and base liner system components of the Federal Cell were selected based on review of embankment cell designs, gradations and specifications presented on the design drawings, a review of estimated properties from literature, and our previous experience with similar type materials. The material properties for the liner and cover system components for use in slope stability analyses are presented in Table 3 below.

Table 3: Summary of Liner and Cover System Material Properties for Slope Stability

System Component	Material Classification	Thickness	Total Unit Weight, γ	Friction Angle, ϕ'	Apparent Cohesion, c'	Undrained Shear Strength
		(inches)	(pcf)	(deg)	(psf)	(psf)
Side Rock	Rip Rap	18	135	40	-	-
Top Slope Cover	Silty Clay from Native Unit 4 amended with 15% gravel	12	120	30	200	-

System Component	Material Classification	Thickness	Total Unit Weight, γ	Friction Angle, ϕ'	Apparent Cohesion, c'	Undrained Shear Strength
		(inches)	(pcf)	(deg)	(psf)	(psf)
Filter Zone	Mix of Gravel/Sand/Fines (GM-GC)	12	130	34	0	-
Frost Protection	Cobble/Gravel/Soil Mixture (GM-GC)	18	130	38	0	-
Radon	Clay	24	123	0	1,000	-
Evaporative Zone	Silty Clay from Native Unit 4	12	120	29	300	-
Clay Liner	Clay	24	123	28	0	1,000[1]
Liner Protective Cover	Silty Sand	12	118	38	250	-

Notes:

1. Undrained strength properties assigned to Clay Liner only. All other materials expected to exhibit drained strength under the analyzed loading conditions.

4.4 Federal Cell Waste Material Properties for Stability

The Federal Cell waste fill material properties for stability are based on our understanding of the planned waste placement methods and a review of readily available literature on the shear strength of CLSM. The stability analyses presented herein assume that the proposed Federal Cell will be filled with DU in the form of LLRW cylinders and drums surrounded by flowable fill (CLSM) at a ratio of approximately 1.9 CY of CLSM per CY of DU placed below grade and beneath the embankment top slope. While the compressive strength is typically used to define specifications for CLSM (150 psi specified for the neighboring LARW embankment), a long-term degraded condition over the 10,000-year compliance period is better represented by the residual shear strength resulting from shear zone failures between the waste cylinders and drums and solidified CLSM. Alternative characterizations for the waste were considered, however the residual strength approach is considered to be an appropriate representation. According to a study titled "Flowable Backfill Materials from Bottom Ash for Underground Pipeline," UU triaxial testing of CLSM suggests that residual strength of CLSM may exhibit strength properties of 36 to 46 degrees for effective friction angle and an effective cohesion of 49 to 140 kPa (Lee, K-J, Kim, S-K and Lee, K-H, 2014). Conservatively, the Federal Cell waste for stability was assigned a **friction angle of**

30 degrees and **unit weight of 120 pcf** (consistent with unit weight selected for the LARW) with **no effective cohesion.** This characterization is conservative and represent the potential long-term degradation of the CLSM and DU fill over the compliance period.

4.5 Analysis Methodology

Slope stability analyses for Federal Cell was performed using the two-dimensional computer program SLOPE/W version 10.2.0.19483 (GEO-STUDIO International, Ltd, 2019). GEOSTUDIO programs are a widely used for geotechnical and geo-environmental modeling and has been in employed by industry geotechnical engineers since 1977 and used in over 100 countries. SLOPE/W is the leading slope stability software for soil and rock slopes. GEOSTUDIO, maker of SLOPE/W, reports that several US Federal clients using their software include USACE, Federal Energy Regulatory Commission (FERC), United States Department of Agriculture Natural Resources Conservation Service (USDA NRCS), Federal Bureau of Reclamation, and Environmental Protection Agency (EPA). The SLOPE/W program can effectively analyze a variety of slope surface shapes, pore-water pressure conditions, soil properties, and loading conditions. The selected SLOPE/W analyses were based on the Morgenstern-Price method of slices, which satisfies both moment and force equilibrium stability on circular sliding surfaces. The method of slices analysis is consistent with guidelines presented by the US Army Corps of Engineers (USACE) Engineering and Design Slope Stability Engineering Manual No. 1110-2-1902 (USACE, 2003). The results of the slope stability analyses are typically presented in terms of a factor of safety (FS) defined as the ratio of the total stabilizing forces/moments along an assumed sliding plane divided by the total sum of internal and external driving forces/moments acting on the sliding mass. SLOPE/W stability analysis graphical results include the assumed critical sliding surface and corresponding rotation center and resulting sliding mass divided into slices for computational purposes, and material properties.

4.6 Design Criteria

The design criteria for global static and seismic slope stability evaluations presented herein were adopted from the DWMRC's CAW licensing action. The accepted criteria are commonly used for evaluating embankment and dam stability and are consistent with Geosyntec's experience with similar projects. The criteria and associated literature references are summarized in Table 4 below.

Table 4: Geotechnical Design Criteria Summary

Analysis	Criteria	Reference
Static Stability	FS>1.5	USACE (2003)
Seismic Stability	Seismic coefficient (k_h) = ½ PGA	Hynes-Griffin, Mary E. and Franklin, Arley G. (1984) and USACE (2003).
	Pseudostatic, FS > 1.2	Hynes-Griffin, Mary E. and Franklin, Arley G. (1984)[1]
	Pseudostatic FS = 1, Post-earthquake cover deformations 150 – 300 mm allowable	Makdisi, F.I., and H.B. Seed (1978)

1. FS of 1.2 was conservatively adapted in previous analyses in 2011 accepted by DWMRC for CAW licensing action based on a review of Hynes-Griffin, Mary E. and Franklin, Arley G. (1984).

4.7 Analyses Scenarios

The following conditions were analyzed to evaluate global static slope stability of the Federal Cell. Upon review of the North-South and East-West geometries and adjacent features of the Federal Cell and existing groundwater levels, two cross-sections were found to be representative of the cell embankment for stability analyses: one section adjacent the proposed ditch and inspection road and one section adjacent an existing waste cell [11(e) or CAW] as shown on the referenced drawings (EnergySolutions, 2020):

- **Short-term** with **existing** groundwater, undrained strength of clay-like soils.
- **Long-term** with **existing groundwater,** drained strength.
- **Long-term** with **groundwater rise,** drained strength.

Each scenario was also analyzed utilizing lower bound sensitivity properties presented in Table 2-2 to account for the impacts of spatial variability and inherent uncertainty in geotechnical engineering properties.

4.8 Short-Term Stability

Short-term loading conditions represent temporary construction conditions where pore water pressures generated by the loads associated with waste embankment construction have not dissipated in the clay-like soils and soil behavior can be characterized as undrained.

The various modes of failure (i.e., circular failures, block failures, deep-seated, and shallow) commonly seen in embankments of similar design and geology were evaluated to identify the critical case for each scenario analyzed. The most critical failure surface is herein reported for each section and loading condition. The results of short-term stability analyses are presented in terms of FS as presented in **Attachment B** and summarized in Table below. The FS for both sections exceed the design criteria of 1.5 for static conditions. The proposed cell geometry is therefore considered stable under short-term conditions.

Table 5-1: Federal Cell Slope Stability Results for Short-Term Conditions

Section	Groundwater	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Existing Conditions at 20 feet bgs	2.7	Block Failure Through Undrained Unit 2 Native	1.5	B-1
Adjacent Cell 11(e)	Existing Conditions at 20 feet bgs	2.6	Block Failure Through Undrained Unit 2 Native	1.5	B-2

4.8.1 Short-Term Stability Analysis – Sensitivity Analysis

The various modes of failure (i.e., circular failures, block failures, deep-seated, and shallow) commonly seen in embankments of similar design and geology were evaluated to identify the critical case for each scenario analyzed using sensitivity properties summarized in **Table 2 - 2**. The most critical failure surface is herein reported for each section and loading condition. The results of short-term stability analyses using sensitivity properties are presented in terms of FS as presented in **Attachment B2** and summarized in **Table 5-2**. The FS for both sections exceed the design criteria of 1.5 for static conditions. The proposed cell geometry is therefore considered stable under short-term conditions even with lower bound sensitivity strengths.

Table 5-2: Federal Cell Slope Stability Results for Short-Term Conditions with Lower Bound Sensitivity Properties

Section	Groundwater	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Existing Conditions at 20 feet bgs	1.8	Block Failure Through Undrained Unit 2 Native	1.5	B2-1
Adjacent Cell 11(e)	Existing Conditions at 20 feet bgs	1.7	Block Failure Through Undrained Unit 2 Native	1.5	B2-2

4.8.2 Short-Term Stability Analysis – Interim Grading

Based on input provided by EnergySolutions regarding their waste placement and cell configuration for the proposed Federal Cell, the critical geometry for interim stability was identified as the excavation into native soils prior to waste placement. The base of the cell is expected to sit approximately 7 feet below current grade with native side slopes excavated at 2H:1V serving as the subgrade for the overlying liner system. The critical scenario for this interim grading condition is short-term loading scenario (undrained strength of clay-like soils) with existing groundwater conditions (20 feet bgs). The result of the interim stability analysis is presented in terms of FS presented in **Attachment B3**. The FS exceeds the recommended value of 1.5. Therefore, the proposed excavation is considered stable.

4.9 Long-Term Stability Analysis

Long-term slope stability was evaluated considering the two design groundwater levels, existing conditions (20 feet bgs) and the extreme-case groundwater rise conditions (base elevation), and drained soil material properties. The drained shear strength of the foundation soils, liner, and cover materials were selected for a Mohr-Coulomb SLOPE/W material model. Materials are expected to exhibit drained strength properties in the long-term condition where pore pressures have dissipated over time, following construction completion of the cell.

The various modes of failure (i.e., circular failures, block failures, deep-seated, and shallow) commonly seen in embankments of similar design and geology were evaluated to identify the critical case for each scenario analyzed. The most critical failure surface is herein reported for each section and loading condition. The results of the long-term stability analysis are presented in terms of FS summarized in Table below and presented in **Attachment B**. The FS for all scenarios analyzed exceed the recommended value. Therefore, the proposed Federal Cell design is considered stable under long-term conditions.

Table 6-1: Federal Cell Slope Stability Results for Long -Term Conditions

Section	Groundwater	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Groundwater Level at Existing 20 feet bgs	3.4	Block Failure Through Clay Liner	1.5	B-3
	Groundwater Level during Future Rise Event (modeled at base elevation)	3.4	Block Failure Through Unit 4 Native	1.5	B-4
Adjacent Cell 11(e)	Groundwater Level at Existing 20 feet bgs	3.3	Block Failure Through Clay Liner	1.5	B-5
	Groundwater Level during Future Rise Event (modeled at base elevation)	3.3	Block Failure Through Unit 4 Native	1.5	B-6

4.9.1 Long-Term Stability Analysis – Sensitivity Analysis

The various modes of failure (i.e., circular failures, block failures, deep-seated, and shallow) commonly seen in embankments of similar design and geology were evaluated to identify the critical case for each scenario analyzed using sensitivity properties of native soils summarized in **Table 2 - 2**.

The most critical failure surface is herein reported for each section and loading condition. The results of long-term stability analyses using sensitivity properties of the native soils are presented in terms of FS as presented in **Attachment B2** and summarized in **Table 6-2**. The FS for both sections exceed the design criteria of 1.5 for static conditions. The proposed cell geometry is therefore considered stable under long-term conditions even with lower bound sensitivity strengths.

Table 6-2: Federal Cell Slope Stability Results for Long -Term Conditions with Lower Bound Sensitivity Properties

Section	Groundwater	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Groundwater Level during Future Rise Event (modeled at base elevation)	3.3	Block Failure Through Unit 4 Native	1.5	B2-3
Adjacent Cell 11(e)	Groundwater Level during Future Rise Event (modeled at base elevation)	3.1	Block Failure Through Unit 4 Native	1.5	B2-4

4.10 Pseudostatic Stability

Pseudostatic slope stability procedures are commonly used to evaluate the likely seismic performance of embankment and dam slopes. The pseudostatic analysis presented in this section is based on the previously accepted analyses by DWMRC and guidelines presented in the Hynes-Griffin and Franklin method (Hynes-Griffin, Mary E. and Franklin, Arley G, 1984). In pseudostatic analyses, the effects of an earthquake are evaluated by applying a static horizontal inertial force to the potential sliding mass. This horizontal inertial force is expressed as the product of the seismic coefficient (k) and the weight of the potential sliding mass. If resulting forces including the inertial forces are greater than the resisting forces, then seismic deformations will take place. In accordance with the design criteria adopted from adjacent cell designs based on Hynes-Griffin and Franklin method (Hynes-Griffin, Mary E. and Franklin, Arley G, 1984), a seismic coefficient equal to 50% of the PGA was used for the pseudostatic analysis and a FS of 1.2 was adapted as a limiting factor of safety for large deformations. The analysis also used groundwater conditions that represent the extreme-case groundwater rise event and undrained material properties for the clay liner and foundational units.

Various modes of failure are evaluated to identify the critical case for each scenario analyzed. The most critical failure surface has been reported herein for each section and loading condition. The results of the pseudostatic stability analysis are presented in terms of FS summarized in Table below and presented in Attachment B. The FS for the scenarios analyzed meet the design criteria. Therefore, the proposed Federal Cell design is not expected to experience large deformations during seismic loading. Simplified seismic deformation analyses for the range of anticipated deformations are presented in Section 4.13.

Table 7: Federal Cell Slope Stability Results for Pseudostatic

Section	Loading Condition	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	k = 0.12 g Groundwater Level during Future Rise Event (modeled at base elevation)	1.3	Block failure through Unit 4 Native	1.2	B-7
Adjacent Cell 11(e)	k = 0.12 g Groundwater Level during Future Rise Event (modeled at base elevation)	1.3	Block failure through Unit 4 Native	1.2	B-8

4.11 Post-Earthquake Stability

To demonstrate the potential effects of cyclic softening in native soils discussed further in Section 6, the proposed Federal Cell was analyzed in SLOPE/W with the potential strength degradation of the clay-like soils following an earthquake event. To model this in SLOPE/W, the foundational clay-like soils (Units 2 and 4) and clay liner were modeled with reduced undrained strength properties. An undrained shear strength degradation of 50% was used to model this phenomenon. This strength reduction is a lower bound estimate to the strength reduction, if any cyclic softening were to happen. Justification for this conservative assumption is provided in Section 6. **A minimum FS for stable static conditions of 1.5** was considered acceptable per design criteria and criteria found in published literature summarized in Section 4.6 above.

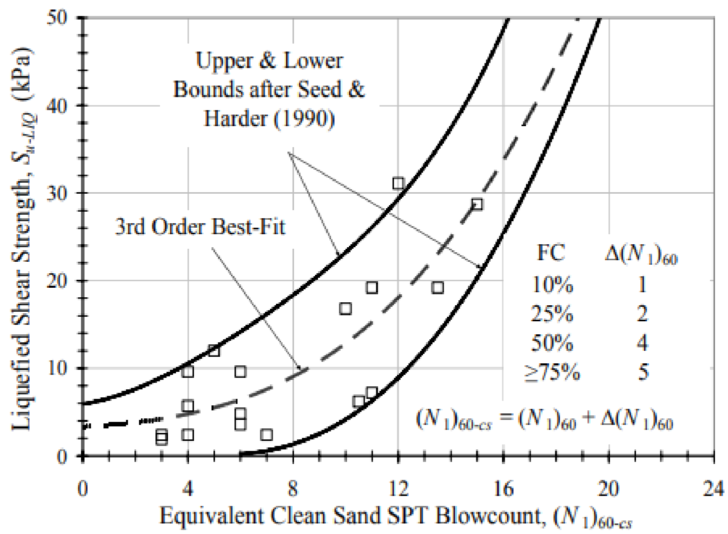
Various modes of failure (i.e. failures through deeper clay Unit 2, clay liner, and shallower clay Unit 4) are evaluated to identify the critical case for each section analyzed. The most critical failure surface has been reported here for each section and loading condition. The results of the post-earthquake stability analysis are presented in terms of FS summarized in the Table below and presented in Attachment B. **The minimum FS of 1.5 was achieved for the sections analyzed** and is therefore considered stable in a post-earthquake scenario where clay-like soils have undergone significant shear strength degradation. A discussion on cyclic softening of clay-like soils is provided in Section 6 of this package.

Table 8-1: Federal Cell Slope Stability Results for Post-Earthquake Cyclic Softening

Section	Loading Condition	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Groundwater Level during Future Rise Event (modeled at base elevation)	1.8	Block Failure Through Unit 4 Native	1.5	B-9
Adjacent Cell 11(e)	Groundwater Level during Future Rise Event (modeled at base elevation)	1.6	Block Failure Through Unit 4 Native	1.5	B-10

4.12 Post-Earthquake Stability – Unit 3 Liquefied Residual Strength

To demonstrate the potential effects of liquefaction of the sand-like soils in Unit 3 discussed further in Section 6, the proposed Federal Cell was analyzed in SLOPE/W with the potential residual strength of the soils following an earthquake event in the event that groundwater rises in the future. To model this in SLOPE/W, the foundational sand-like soils in Unit 3 were modeled with residual strength properties. As discussed further in Section 6, there is a potential for liquefaction of localized medium dense silty sand pockets in Unit 3, assuming a groundwater rise condition. Results of the liquefaction triggering analysis discussed in Section 6 were used to inform the selection residual strength for Unit 3 by estimating a liquefied undrained shear strength through correlation with the minimum $(N_1)_{60\text{-}CS}$ from the liquefaction analysis results (**Attachment E2)** and use of an empirical relationship presented by Seed and Harder (1990) shown in the figure below. The resulting minimum $(N_1)_{60\text{-}CS}$ for Unit 3 sand-like soils has a value of 20, correlating to a liquefied shear strength of at least 50 kPa (or ~1000 psf).



Various modes of failure (i.e. failures through deeper clay Unit 2 and shallower Unit 4 and 3) were evaluated to identify the critical case for each section analyzed. The most critical failure surface has been reported here for each section and loading condition. The results of the post-earthquake stability analysis with liquefied residual strengths are presented in terms of FS summarized in the Table below and presented in Attachment B3. **The minimum FS of 1.5 was achieved for the sections analyzed** and is therefore considered stable in a post-earthquake scenario where sand-like soils have liquefied, and clay-like soils have undergone significant shear strength degradation. A discussion on liquefaction of the sand-like soils is provided in Section 6 of this package.

Table 8-2: Federal Cell Slope Stability Results for Post-Earthquake Liquefaction and Cyclic Softening

Section	Loading Condition	Factor of Safety	Critical Failure Mode	Minimum Required Factor of Safety	Figure
Adjacent Road/Ditch	Groundwater Level during Future Rise Event (modeled at base elevation)	2.0	Block Failure Through Unit 3 Native	1.5	B-11
Adjacent Cell 11(e)	Groundwater Level during Future Rise Event (modeled at base elevation)	1.9	Block Failure Through Unit 3 Native	1.5	B-12

4.13 Seismic Deformation

The seismic deformation analysis for the Federal Cell was performed using the Makdisi and Seed (1978) simplified method for estimating seismically induced deformations for earthen embankments and geosynthetics. The site-specific seismic design parameters such as PGA and Mw required for estimating seismically induced slope deformations were based on the referenced seismic hazard analysis that justified DWMRC's 2012 license action and as discussed in Section 3.4, are as follows:

- **PGA = 0.24g**
- **Mw = 7.3**

The seismic deformation analysis includes performing a pseudostatic stability analysis and determining the yield coefficient, k_y, resulting in an FS equal to 1. The k_y is next compared with the maximum estimated inertial force, k_{max}, to empirically estimate the anticipated embankment deformations based on the earthquake magnitude. In accordance with the current state of practice and previous analyses for the adjacent cells, seismically induced deformations of 150 to 300 mm are considered acceptable. The seismic deformation analysis results are summarized in Table 9 and presented in Attachment C.

Table 9: Federal Cell Seismic Deformation Results

Case/Description	k_y	$\ddot{u}_{max}$	y (ft)	H (ft)	y/H	$k_{max}/\ddot{u}_{max}$	k_{max}	k_y/k_{max}	Estimated Deformation (mm)
Critical Section Failure Through Unit 4 Native, Entire Slope Face (y/H=1), Adjacent Cell 11(e)	0.18	0.58	52	52	1	0.34	0.2	0.91	4

Notes:
1. y is depth of sliding mass under evaluations
2. H is average height of the potential sliding mass

Results of the permanent deformation analyses (using undrained strengths and groundwater rise elevation), estimate seismically induced deformations to be negligible. Therefore, the performance of the Federal Cell under the provided earthquake ground motions, is considered to be acceptable in terms of seismically induced deformations.

5. SETTLEMENT ANALYSIS

The DWMRC raised concerns for the uncertainty in the parameters used for geotechnical analysis of the proposed Federal Cell foundation settlement and subsequent embankment response in the referenced Technical Report (DWMRC, 2021). The following sections describe the method of analysis and results of estimated elastic, primary consolidation, and secondary compression settlement of the Federal Cell foundational soils and the consequences of these estimates. Settlement calculations presented herein are considered conservative as the condition modeled assumes a "wished into place" scenario. In reality, construction of the proposed cell is likely to be slow enough (on the order of ±10 years) to allow for dissipation of pore pressures in the underlying fine-grained soils, resulting in near completion of primary consolidation settlement by the end of waste placement and start of cover construction. Conservatively we assumed primary consolidation settlements would go on another year following final placement of waste. This is considered conservative due to the presence of consistent interbedded sandy layers observed in the subsurface. Sandy soils act as drainage layers that allow for pore pressures to dissipate and expedite consolidation of the fine-grained soils. Over the course of construction, these fine-grained soils are expected to experience this consolidation and be nearly complete by end of waste placement. This phenomenon has been modeled and predicted for the other adjacent cells (AMEC, 2005). Based on the analysis, Geosyntec's opinion is that predicted settlement of the cell would not have an adverse impact on the stable slope conditions as magnitude of settlement is expected be limited and would cause only limited flattening of the top slopes. The flattening slopes and potential differential settlements could reduce the drainage slopes over the cover locally and affect infiltration. This is something that should be considered during design and construction.

5.1 Previous Analyses

While other adjacent cells varied in geometry and waste fill types, findings of previous settlement analyses and models for other cells were reviewed for comparison and consistency. The load and geometry may vary, but the subsurface conditions beneath the adjacent cells are generally consistent with that of the Federal Cell. Settlements of the foundational soils due to embankment loading are projected to be on the order of 12 to 16-inches with secondary settlements calculated over 500-year compliance period on the order of 8-inches. The analysis justifying DWMRC's license action for the CAW predicted and modeled these settlements for an embankment height of approximately 100 feet for various waste types including compressible debris, incompressible debris, and CLSM. The proposed waste and cover materials for the Federal Cell may have a greater average unit weight than the CAW (120 pcf versus 100 pcf), but the proposed embankment is almost half the height of the CAW. Therefore, Geosyntec predicts that the expected foundation settlement for the Federal Cell will likely be less than the CAW models.

5.2 Compressibility Properties of Foundation Soils

The compressibility properties of the subsurface soils used to evaluate the foundation settlement were estimated from laboratory testing results for the fine-grained soils and derived from typical values for the coarse-grained soils at specified in-situ confining pressures. Correlations from published literature were also used to supplement the laboratory data.

2005 interpretation of various explorations across the Clive Facility (Bingham 1992, AGRA 1999, and AMEC 2004) has been provided in Attachment A. In these previous studies, consolidation tests were performed on fine-grained soil units (Units 2 and 4) that have been consistently encountered in the subsurface across the Clive Facility. Geosyntec used the interpreted results provided to evaluate consolidation properties (Cc, Cr, OCR) of these soils that also underlie the proposed Federal Cell.

Initial void ratios (e_o) from the consolidation tests were not provided in the aforementioned lab summary data table (Attachment A), therefore Geosyntec used in-situ water content (w) laboratory test results for the underlying soils to estimate the initial void ratio of the fine-grained soils through the use of published empirical correlations. The e_o of the materials was estimated using the following relationship between water content and the specific gravity for saturated soils:

$$e_o = Gs\ (w/100)$$

Where Gs is the specific gravity of the soils; assumed to equal 2.65.

The modified secondary compression index ($C\alpha\varepsilon$) is typically calculated through interpretation of the consolidation test results and defined as the slope of the compressive strain plotted against logarithm of time observed post primary consolidation during the test. A correlation was used that relates $C\alpha\varepsilon$ to the estimated in-situ moisture content. $C\alpha\varepsilon$ of the materials was estimated using the following relationship between water content:

$$C\alpha\varepsilon = 0.0001w$$

Elastic settlement of the coarse-grained materials (Units 1 and 3) are typically estimated through use of the constrained modulus (M_s) of the soil. The sandy subsurface materials in Unit 3 are assumed to have an elastic modulus of approximately 1,800 psi and a Poisson's ratio of 0.25. The subsurface materials in the Lower Sand Unit 1 are assumed to have an elastic modulus of approximately 2,300 psi and a Poisson's ratio of 0.38. The elastic modulus and Poisson's ratios were selected based properties of similar soils types are equivalent confining pressures (Qian et al. 2002). The M_s was calculated with equation presented above.

$$M_s = \frac{E_s \times (1 - v_s)}{(1 + v_s)(1 - 2 \times v_s)}$$

where:

v_s = Poisson's ratio of soil, ft; and

E_s = elastic modulus of soil, lb/ft^2.

The unit weights of geologic units are consistent with the assignments used in the slope stability analyses discussed earlier.

A summary of the resulting settlement material properties used in our settlement analysis is provided in Table 10.

Table 10: Summary of Properties for Foundation Settlement Analysis

Unit	Thickness	Unit Weight γ	Constrained Modulus Ms	Primary Compression Index C_c	Recompression Index C_r	Modified Secondary Compression Index $C_{\alpha\varepsilon}$	OCR	Water content (%)	Initial Void Ratio e_o
	(ft)	(pcf)	(psf)				(psf)		
4	2	118	-	0.25	0.02	0.004	5	40	1.06
3	14	120	311,040	-	-	-	-	-	-
2	22	121	-	0.20	0.025	0.0045	1.5	45	1.2
1	55	120	531,560	-	-	-	-	-	-

5.3 Federal Cell Loading and Geometry

For this calculation package, the settlement evaluation is based on the geometry presented in Table 1. For simplification the load was calculated as the maximum height (52.5 feet) of fill with an average unit weight of 120 pcf. The loading shape was approximated with a rectangular loading shape for the purposes of settlement analysis. This is considered representative of the average unit weight of CLSM, the waste, and the various cover and liner materials. This results in a load over the foundational soils of **approximately 6,300 psf applied at the base of the Federal Cell.**

A stress distribution model was developed to assess elastic and consolidation settlement. The change in stress ($\Delta\sigma$) is due to the Federal Cell height above the ground surface approximated to be 6,300 psf. The change in stress in the underlying soils is calculated as the difference between the existing overburden stress and the overburden pressure due to the Federal Cell. The distribution of the total stress with depth assumes that the Federal Cell is an infinite embankment. The increase in stress at depth ($\Delta\sigma_{(z)}$) is equal to the change in stress at the surface ($\Delta\sigma$) distributed over an effective base area that increases with each depth interval below the surface, this is determined with the following equations:

$$\Delta\sigma_{(z)} = (\Delta\sigma * Area_{base})/Area_{effective}$$

$Area_{effective} = (B + z)*(L+z)$ and

B = Base width of the cell (ft)

L = Base length of the cell (ft)

z = interval depth below ground surface (ft)

The change in stress within the geologic units was evaluated for each 1-foot interval bgs. The stress distribution calculations are presented in the settlement analysis calculations presented in **Attachment D.**

The magnitude of loading estimated here are the average loading beneath the top deck portion of the embankment where the maximum embankment height is experienced and expected to decrease linearly over the top slopes to essentially to no loading at the toe of the embankment.

5.4 Elastic Settlement (Immediate) of the Sand-Like Units (1 and 3)

Because of the coarse-grained nature of sand-like units (Units 1 and 3), the settlement of these layers is anticipated to be primarily the result of elastic or immediate settlement. To evaluate the potential effects of elastic settlement of the sand units, the units are assumed to behave as an elastic and homogeneous medium. The foundation settlement is calculated using the Elastic Settlement Equation, which is:

$$Z_e = \frac{\Delta\sigma}{M_s} \times H_o$$

where:

Z_e = elastic settlement of soil layer, ft;
H_o = initial thickness of soil layer, ft;
$\Delta\sigma$ = change in stress, psf (discussed in Section 5.3); and
M_s = constrained modulus of soil, lb/ft^2 (provided in Table 9, discussed in Section 5.3).

The change in stress at each 1-foot interval in Units 1 and 3 and the corresponding constrained modulus were then used to calculate the elastic settlement with the equation presented above for each layer interval. The results of each interval where then summated to a cumulative estimate for elastic settlement of Units 1 and 3. The elastic settlement for each unit is summarized in the Table below and presented in **Attachment D**. The elastic settlements are expected to occur during construction of the Federal Cell and be complete prior to cover construction. The elastic settlement reported herein is therefore not expected to adversely impact the long-term stability of the cover and will likely not need to be considered or accounted for during cover construction.

Table 11: Foundation Soil Elastic Settlement

Unit	Material Description	Estimated Elastic Settlement (inches)
3	Upper Silty Sand	3
1	Deeper Silty Sand with CL/ML lens	8

5.5 Primary Consolidation

Because of the fine-grained nature of Units 2 & 4, the settlement of these layers is anticipated to be a result of consolidation. The subsurface stratigraphy is discussed in Section 3.2 above with the material properties summarized in Table 10. To calculate the consolidation settlement (S_c), Units 2 and 4 were broken into 1-foot-thick intervals. The total consolidation settlement within each unit was the summation of the consolidation settlement in the individual 1-foot-thick layers. Based on the consolidation lab data discussed in Section 5.2, the soils are likely overconsolidated.

The overconsolidation ratio (OCR) for Units 2 and 4 are presented in Table 10. The equation for consolidation settlement for overconsolidated soil is as follows:

$$S_c = \frac{C_r}{(1+e_o)} \times H \times log\left[\frac{(\sigma'_p)}{\sigma'_o}\right] + \frac{C_c}{(1+e_o)} \times H \times log\left[\frac{(\sigma'_{vo}+\Delta\sigma)}{\sigma'_p}\right]$$

where,

e_o = See Table 10	initial void ratio
H = 1	thickness of the compressible layer interval (ft)
C_c = See Table 10	compression index
Cr = See Table 10	recompression index
OCR = See Table 10	overconsolidation ratio
σ'_p = OCR *σ'_{vo}	maximum past pressure (psf)
σ'_{vo} = varies	initial vertical effective stress (psf). Groundwater was assumed at a depth of 25 feet below the ground surface (existing level)
$\Delta\sigma$ = varies	change in stress due to overburden loading (psf) (See Section 5.3 for discussion and Attachment C for stress distribution calculations)

Calculation of primary consolidation settlement of Units 2 and 4 is provided in **Attachment D** and summarized in Table 12 below.

5.6 Secondary Compression

Secondary compression is typically observed after primary settlement is substantially complete. For the purpose of calculations, this is often assumed as the time at which the material reaches 95% degree of consolidation. As discussed earlier, because the waste embankment placement takes place relatively slowly, the primary consolidation is expected to be substantially complete as the filling is complete and by the time cover materials are placed. With this assumption and using the secondary compression parameter presented in Table 10, **secondary compression during the compliance period of 10,000 years was estimated through the following relationship**:

$$Ss = C\alpha\varepsilon * H_{100}(\frac{t_2}{t_1})$$

Where

Ss	time dependent secondary settlement occurring between t_1 and t_2
$C\alpha\varepsilon$ = See Table 9	modified secondary compression index
H_{100} = varies	total thickness of compressible layers at the end of primary consolidation (for each 1-foot interval in Units 2 and 4)
t_1 = 1-year	time between the placement of last significant waste in the cell and cover construction (assumed to be 1 year based on review of previous analyses and conservative assumptions regarding the pace of construction)
t_2 = 10,000 years	time for which secondary settlements are estimated for (compliance period of 10,000 years)

Summation of the secondary compression of each 1-foot interval of Units 2 and 4 was performed to estimate the cumulative secondary compression of each unit. The calculations for secondary compression are presented in **Attachment D** and summarized in Table 12 below.

Table 12: Foundation Soil Consolidation and Secondary Compression Settlement

Unit	Material Description	Estimated Primary Consolidation Settlement (inches)	Estimated Secondary Compression Settlement (inches)
4	Upper CL-ML	3	<1
2	Deeper CL-ML	9	5

5.7 Consequences of Settlement

Based on our understanding of the subsurface stratigraphy beneath the proposed Federal Cell and review of other adjacent cell studies (AMEC, 2005 & 2011), there are two principal geologic units (Units 2 and 4) which may be subject to long-term settlements. These long-term settlements estimated in this calculation package are principally a result of consolidation settlements of fine-grained soils. **The upper sand unit (Unit 3) and lowermost sequence of sands with thin lifts of clays and silts (Unit 1) are not anticipated to impact long-term settlements**. The elastic settlements of those layers were reported in this package to provide a complete picture of the total estimated settlement in the foundational soils of the proposed Federal Cell. It is the primary consolidation and secondary compression settlements, however, that should be considered during design and construction of the cell cover. **Based on the results presented in Table 12, 12 inches of primary consolidation settlement and 6 inches of secondary compression settlement may result from construction of the Federal Cell.** Considering the loading rate, the primary consolidation settlement will likely occur simultaneously during waste placement and will be substantially complete by the time the waste reaches its final elevation. We assumed 1 year after completion of waste placement for completion of primary consolidation, as a conservative estimate discussed previously. Secondary compression settlements which are relatively small in magnitude, however, should be considered in cover design to ensure proper drainage is achieved because these settlements will occur after the cover construction. The analyses assumed a secondary compression time period of 10,000 years per compliance period requirements. **A conservative assumption of zero secondary compression at the edge of the cell and the maximum magnitude of 6 inches at the center would result in an average settlement gradient of 6 inches over approximately 600 feet as 0.1 %.** Therefore, the current design gradient of 2.4% maybe reduced to 2.3% in an average sense which is considered negligible.

The magnitude of settlements estimated here are for the top deck portion of the embankment where the maximum embankment height is experienced and expected to decrease linearly over the top slopes to essentially no settlement at the toe of the embankment. Therefore, settlement of the foundational soils as a result of construction of the Federal Cell are not expected to adversely impact the adjacent cells.

Settlement plate instrumentation may be used during cell construction to monitor consolidation settlements, project substantial completion of consolidation settlements, and confirm design assumptions prior to construction of the cover. These results may be useful for future waste cell designs and construction. Overbuilding the cover and performing inspections and routine maintenance over the monitoring period may help to mitigate the effects of long-term settlement.

5.8 Consequences of Spatial Variability for Settlement

In response to DWMRC's RFI dated 19 December 2022 Item D-2 requesting sensitivity analyses for the geotechnical engineering evaluations to account for spatial variability and inherent uncertainty of the subsurface conditions, a statistical analysis was performed on the available laboratory testing data available from the following explorations:

- B-1 & B-2 (AMEC, 2005);
- SC-1, -7, -8, -10 & SLC-84 (D&M, 1984);
- GW-16, -17, -18, -19A, -19B, (Bingham Environmental, 1992); and
- DH-1 (AGRA, 1999).

The statistical analysis was focused on the compressibility parameters of the clay-like soils, since the nature of how these soils may consolidate over a long time period compared to immediate settlement of sand-like soils impact the design and construction of the Federal Cell. As mentioned previously in Section 5.4, Unit 3 and 1 sand-like soils are expected to undergo elastic settlements that will likely occur during construction of the Federal Cell and be complete prior to cover construction. Therefore, these elastic settlements are not expected to adversely impact the long-term stability of the cover and thus a sensitivity analysis of the compressibility parameters for Unit 3 and 1 soils was not performed.

The laboratory testing summary table is provided in **Attachment A.** Following assembly of the compressibility data set for Unit 4 and Unit 2, each value (i.e., Cc, Cr, eo) was plotted by subsurface elevation and adjacent the median, ±1 standard deviation, 33rd or 66th percentile, and the previously selected parameter value for the subsurface unit (Unit 4 and Unit 2). Results of the statistical analysis for compressibility properties related to consolidation settlement are shown on **Figure 7 through Figure 10.**

The driving factor for considering impacts of long-term settlement on a stable condition for the proposed Federal Cell is the potential for final cover slope reversal that could adversely impact the drainage design and lead to unwanted ponding. Thus, the key consideration for spatial variability under the proposed cell is the potential for differential settlement. To quantitatively assess the potential for differential settlement, the statistical analysis results (**Figures 7 - 10**) for were used to evaluate primary and secondary compression of Unit 4 and 2 soil layers by using +1 (maximum settlement) and -1 (minimum settlement) standard deviation compressibility values. The result of this calculation is provided in **Attachment D2** and summarized in the Table below.

Table 13: Minimum and Maximum Estimated Settlement

Unit	Material Description	Estimated Minimum Primary Consolidation Settlement (inches)	Estimated Minimum Secondary Compression Settlement (inches)	Estimated Maximum Primary Consolidation Settlement (inches)	Estimated Maximum Secondary Compression Settlement (inches)
4	Upper CL-ML	1	<1	6	<1
2	Deeper CL-ML	3	1	22	5

As mentioned previously, secondary compression settlements should be considered in cover design to ensure proper drainage is achieved, because these settlements will occur after the cover construction. Results in Section 5.7 indicated a maximum differential settlement of 6 inches may occur in response to secondary compression. Results of the sensitivity analysis, using minimum and maximum secondary compression estimates in Table 13 above, indicate similar results (~6 inches of differential settlement), thus conclusions in Section 5.7 are unchanged.

6. LIQUEFACTION

Based on our understanding of the Technical Report (DWMRC, 2021), we understand the 10,000-year compliance period for the proposed Federal Cell presents a need for conservative approaches to analyzing the geotechnical stability mechanisms. The following sections summarize the liquefaction analyses performed for the proposed Federal Cell that support this need. The analyses presented are based on an extreme groundwater level rise resulting in a groundwater elevation equal to the current existing ground surface (a 25 feet groundwater rise event).

6.1 Previous Analyses

A groundwater level of 26 feet bgs was used in previous liquefaction analyses for the Clive Facility (AMEC 2005, 2011, and 2012). Therefore, the upper sand Unit 3 was not considered during their liquefaction triggering analysis. Previous calculations indicated that liquefaction of the saturated soil layers below the site (Units 1 and 2 at the time) was not a design issue for the adjacent waste cells. For the seismic design event analyzed, majority of the soils in the upper 30 to 60 feet of the subsurface, Unit 2, consist of cohesive deposits, which have a low probability of liquefaction due to their high clay content. It was also found that the interbedded cohesionless silt and silty sand deposits in Unit 1 would be also unlikely to liquefy due to their relatively high density. **Geosyntec**

generally agrees with this prediction for Unit 1 and considers it applicable to the Federal Cell Unit 1 soils, however consideration for the upper sand Unit 3 was included in the current analysis to reflect the groundwater level rise condition that would saturate the cohesionless soils.

6.2 Seismic Design Parameters

The site-specific seismic design parameters such as PGA and Mw required for estimating liquefaction triggering were based on the referenced seismic hazard analysis that justified DWMRC's 2012 license action and as discussed in Section 3.4, and are as follows:

- **PGA = 0.24g**
- **Mw = 7.3**

6.3 Liquefaction of Sand-Like Soils

The liquefaction triggering analysis was performed following the procedures outlined in Idriss and Boulanger (2008) for the sand-like soils in Unit 3. Sand-like soils are referred to soils which primarily consist of coarse-grained particles more than 50 percent by weight or very low plasticity fine-grained soils (i.e., low plasticity silts). The soils classified as clay were not considered susceptible to liquefaction and their evaluation is discussed in following section.

Boring logs for GW-36 through GW-38 (Bingham, 1992) which were excavated with a hollow-stem auger (HSA) and extended to depths of 30 feet bgs into proposed Federal Cell area limits were used to complete the analysis (logs are provided in **Attachment A**). Due to the limitations of HSA drilling methods in keeping the drilled hole stable for drilling at or below groundwater level, SPT blow counts recorded at or below groundwater do not provide a meaningful representation of the subsurface soil density. Therefore, the liquefaction triggering analysis herein only presents results for soils with SPT blow-counts above the groundwater readings; approximately 18 to 20 feet bgs. Fines content results were not available for Unit 3 samples collected from GW-36 through GW-38. The fines content was therefore assumed to represent a silty sand with the lower bound fines content of 15%.

Detailed calculations for the liquefaction triggering analysis are presented in Attachment E. Results indicate that sand-like soils within the upper 20 feet below ground surface are not anticipated to liquefy under the design seismic loading with the exception of a thin layer between 14 and 16 feet bgs encountered in GW-38 that resulted in a FS greater than 1.0 but less than 1.1, which indicates there is potential for localized liquefaction to occur in this layer. The potential for seismic settlement in this layer is less than ½ an inch and localized to the location of GW-38 (**Figure 1**). Considering the dense nature of the sands in Unit 3, localized liquefaction will likely induce a dilative behavior and not adversely impact the strength of the sands. **Therefore, these affects are not anticipated to undermine the stable conditions of the proposed Federal Cell**.

6.3.1 Additional Liquefaction Analyses for Unit 3

In response to DWMRC's RFI dated 19 December 2022 Item D-2 requesting sensitivity analyses for the geotechnical engineering evaluations to account for spatial variability and inherent uncertainty of the subsurface conditions, additional boring logs (GW-16, -17, -18, -19A, -19B, -24, -27, -29, -36, -37, -38, **included in Attachment A**) were used to perform a focused liquefaction triggering assessment of the Unit 3 sand-like soils. The additional boring logs were selected based on proximity to the proposed Federal Cell and availability of meaningful data (i.e., groundwater, rig information, borehole diameter, etc.). Adding more SPT blow count data to the liquefaction triggering assessment is intended to capture the probable variability of the Unit 3 sand-like soils and reduce uncertainty in our liquefaction triggering results. Detailed calculations are presented in **Attachment E2** with results presented on **Figure 11.** Results indicate that sand-like soils in the upper 26 feet are not anticipated to liquefy under the design seismic loading, with the exception of 4 out of 56 blow count data points (**Figure 11**) around 14 to 16 feet and 18 to 20 feet bgs suggesting the potential for localized liquefaction with resulting FS calculated as less than 1.0. The potential for seismic settlement in these layers is less than ½ an inch cumulatively. **These effects are not anticipated to undermine the stable conditions of the proposed Federal Cell.** As an additional conversative measure, the minimum $(N_1)_{60\text{-}CS}$ value from the liquefaction triggering analysis for Unit 3 sand-like soils was used to estimate a residual liquefied strength for a post-earthquake slope stability analysis discussed in Section 4.12. Results indicated that residual liquefied strengths will still yield a stable condition post-earthquake.

6.4 Cyclic Softening of Clay-Like Soils

Cyclic softening is a phenomenon where fine-grained soils do not undergo liquefaction, but experience reduction in strength and stiffness caused by cyclic deformations due to increase in pore pressures during seismic shaking. Previous analysis concluded that cyclic softening is highly unlikely, presenting a very low related risk of cyclic softening (of Units 2 and 4 clay-like soils) (AMEC, 2012). Considering that most clays in upper Unit 4 will be removed as part of construction of the proposed Federal Cell and given the stiff nature of Unit 2 clays, Geosyntec generally agrees with this conclusion from the DWMRC's prior licensing decisions. Geosyntec has evaluated the global stability of the Federal Cell for a post-earthquake event that results in 50% strength reduction of all clay-like soils, clay-liner included representing a conservative and less likely strength reduction scenario. The results of this stability condition are discussed in Section 4.11. Results indicated that even a strength reduction of 50% in the clay-like soils and liner will still yield a stable condition post-earthquake.

7. CONCLUSIONS

7.1 Global Static, Seismic Slope Stability and Deformation

Based on the results of Geosyntec's slope stability analyses, the design of the proposed Federal Cell will remain stable for global static short-term (including interim), long-term, seismic, and post-earthquake conditions presented in this package. Results are presented in **Attachment B, B2, and B3.** Based on the results of the seismic deformation analysis, the design of the proposed Federal Cell slopes and cover will not experience significant seismic induced deformations (<5 mm). Results are presented in **Attachment C**.

7.2 Settlement

Based on the results of the settlement analyses, the current load of the proposed Federal Cell may result in up to 11-inches of elastic settlement of sand-like soils, 12-inches of primary consolidation of clay-like soils, and 6-inches of secondary compression settlement of clay-like soils. Elastic settlement and primary consolidation settlement presented in this package should be complete within one year after the embankment waste placement (within the required settlement monitoring period) and is not interfere with the post-construction performance of the cover. The 6-inches of secondary compression settlement of clay-like foundation soils should occur over a compliance period of 10,000 years and are not projected to impact the long-term performance of the cover and embankment. The magnitude of settlements estimated here are for the top deck portion of the embankment where the maximum embankment height is experienced and expected to decrease linearly over the top slopes to essentially no settlement at the toe of the embankment. Therefore, settlement of the foundational soils as a result of construction of the Federal Cell are not expected to adversely impact the adjacent cells. Results are presented in **Attachment D & D2.**

7.3 Liquefaction and Cyclic Softening

Based on the results of liquefaction triggering analyses and seismically induced cyclic softening, these hazards are not projected to undermine the stable condition of the proposed Federal Cell.

Seismically-induced settlements of the sand-like soils are negligible (<1 inch.) In the event that sand-like soils liquefy, liquefied residual strengths would still yield a stable slope condition post earthquake. Cyclic softening of the clay-like soils is highly unlikely to occur as a result of the design seismic event (0.24g PGA and 7.3 Mw), nevertheless a 50% strength degradation of the clay-like soils would also still yield a stable slope condition post-earthquake. Results of the sand-like soils liquefaction analysis are presented in **Attachment E & E2** and the post-earthquake softened clay stability analyses are provided in **Attachment B.**

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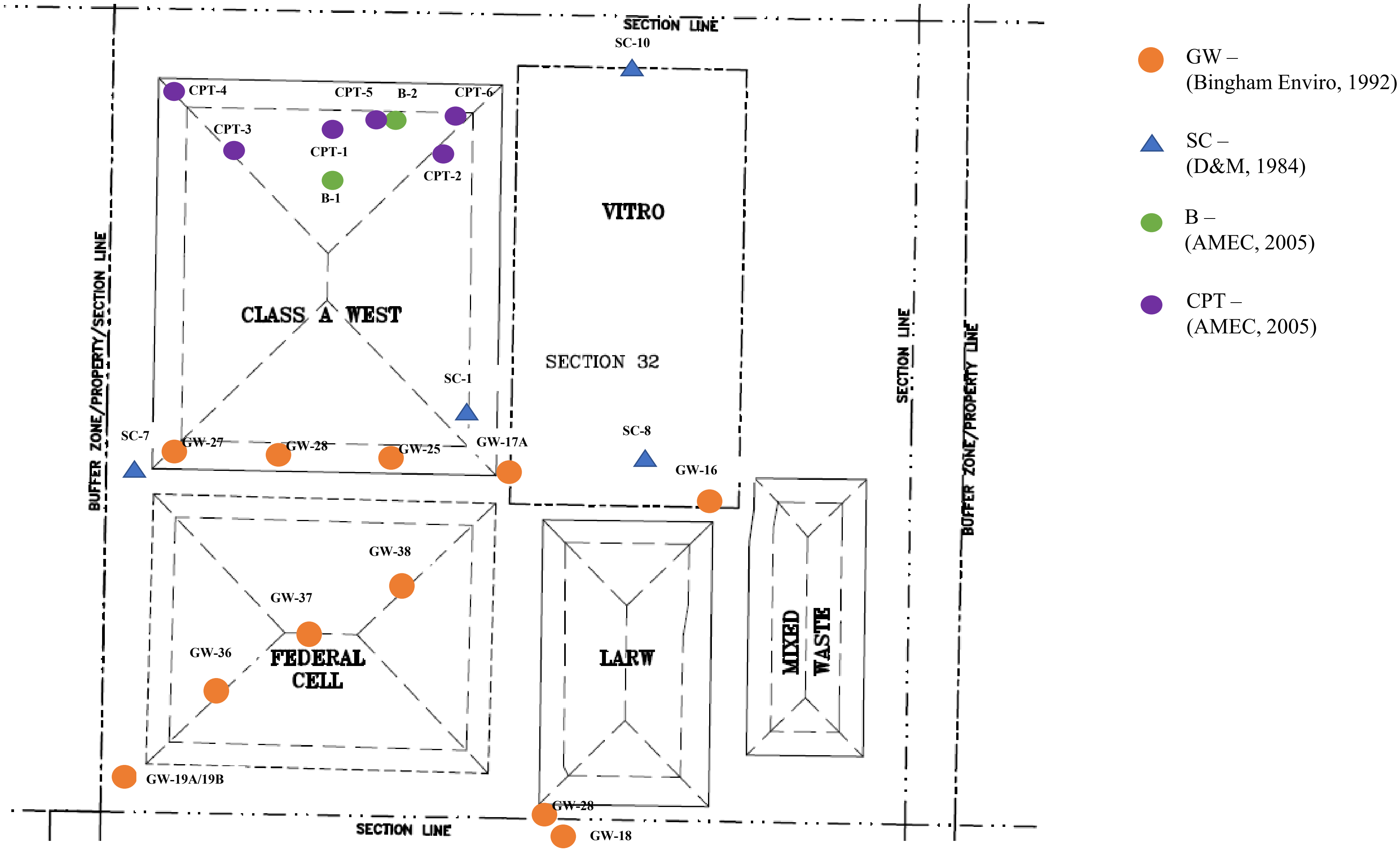
FIGURES



Notes:
1. Base image from the Hydrogeologic Report (Bingham, 1992)
2. Other explorations are known to exist across Section 32 of the Clive Facility. Explorations shown here were used for the Federal Cell geotechnical engineering evaluations.

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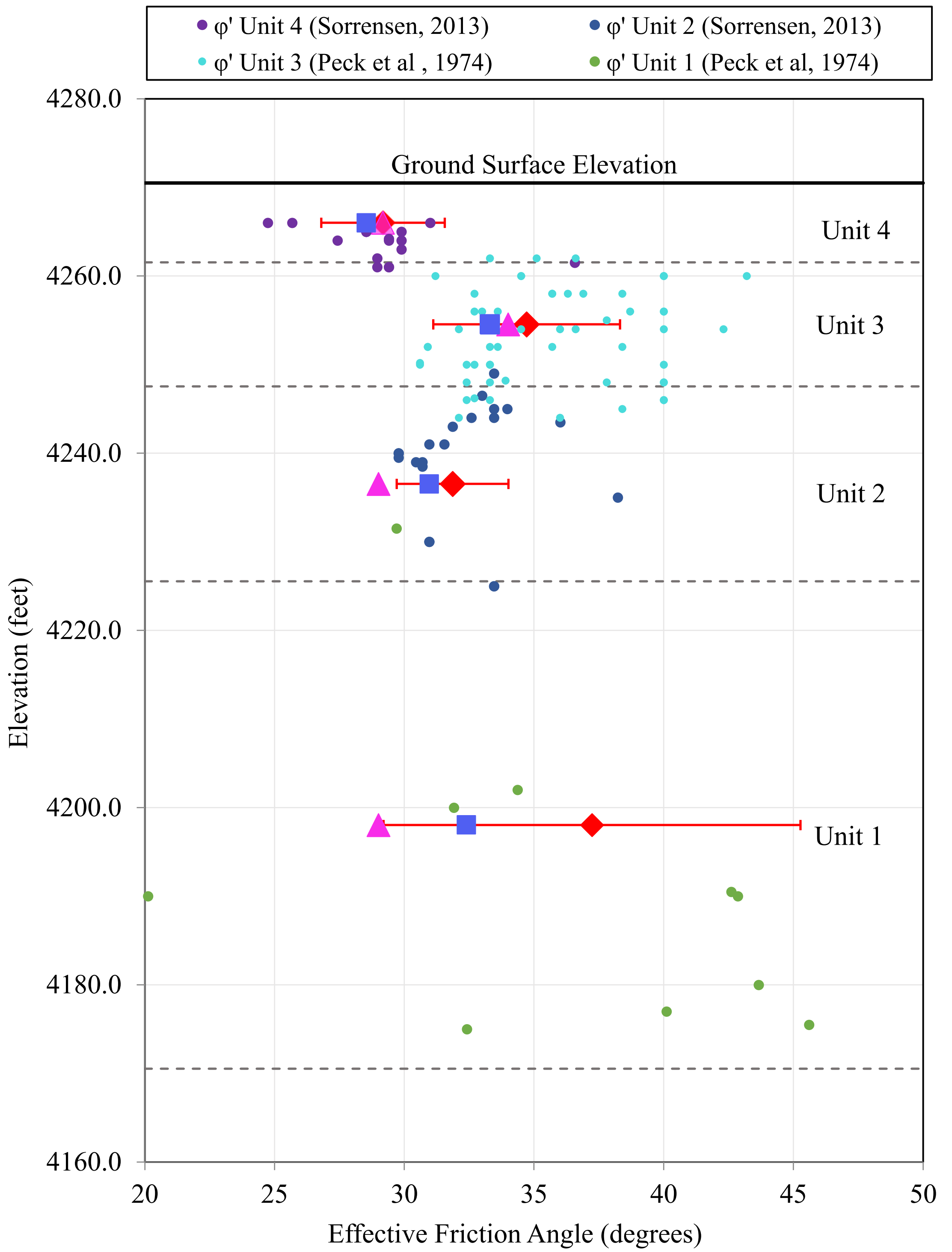
SITE LAYOUT AND EXISTING EXPLORATIONS
FEDERAL CELL AT CLIVE FACILITY
CLIVE, UTAH

FIGURE NO.	1
PROJECT NO.	SLC1025
DATE:	MARCH 2021



LEGEND:

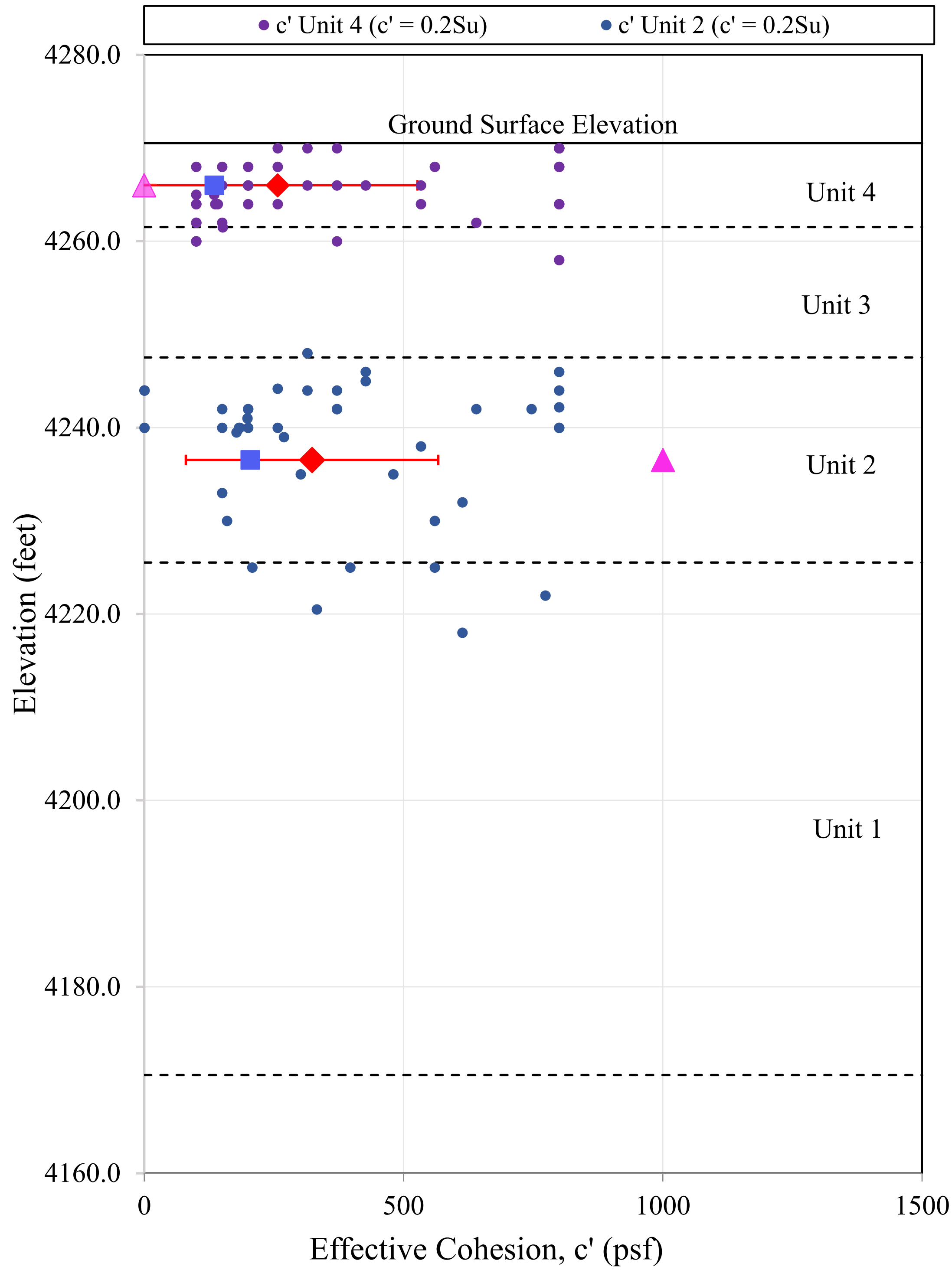
- MEDIAN
- SELECTED VALUE
- 33RD PERCENTILE
- +- 1 STANDARD DEVIATION

FRICTION ANGLE STATISTICAL ANALYSIS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

Geosyntec consultants

Figure 3

Project No: SLC1025 | JANUARY 2023



LEGEND:

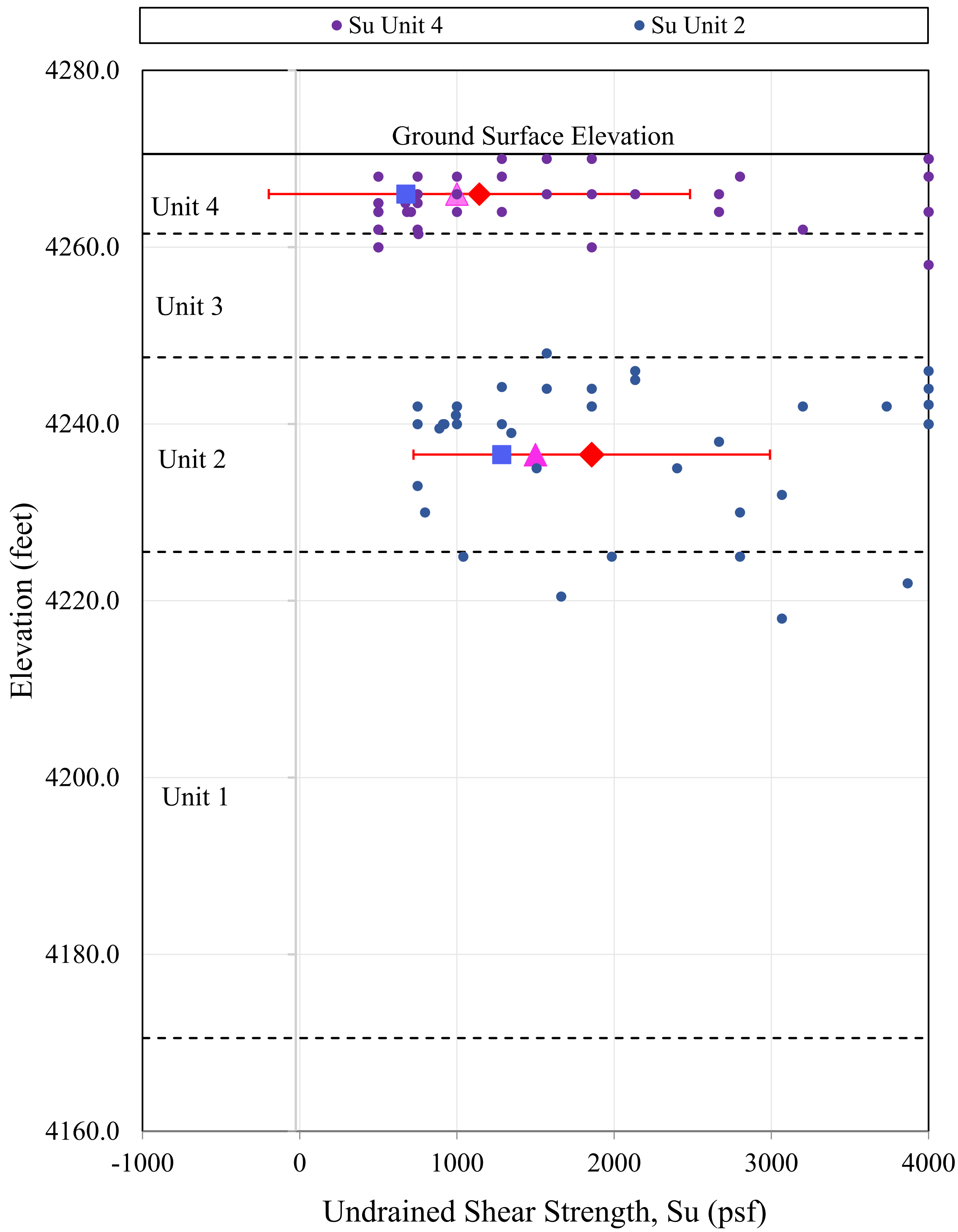
- MEDIAN
- SELECTED VALUE
- 33RD PERCENTILE
- +- 1 STANDARD DEVIATION

EFFECTIVE COHESION STATISTICAL ANALYSIS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

Geosyntec consultants

Figure **4**

Project No: SLC1025 | JANUARY 2023



LEGEND:

- MEDIAN
- SELECTED VALUE
- 33RD PERCENTILE
- +- 1 STANDARD DEVIATION

UNDRAINED SHEAR STRENGTH STATISTICAL ANALYSIS

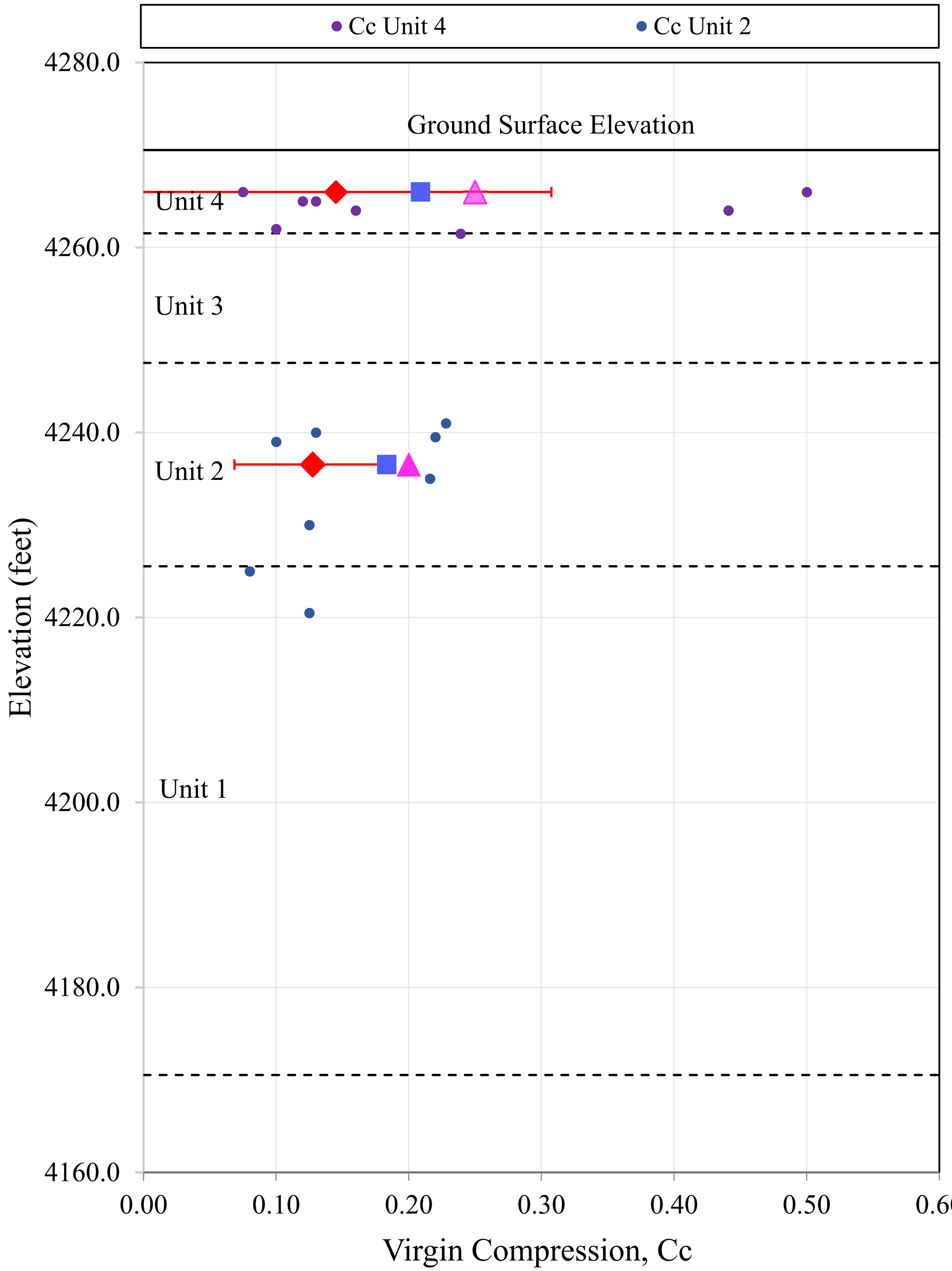
CLIVE FACILITY FEDERAL CELL

CLIVE, UTAH

Geosyntec consultants

Figure **5**

Project No: SLC1025	JANUARY 2023



LEGEND:

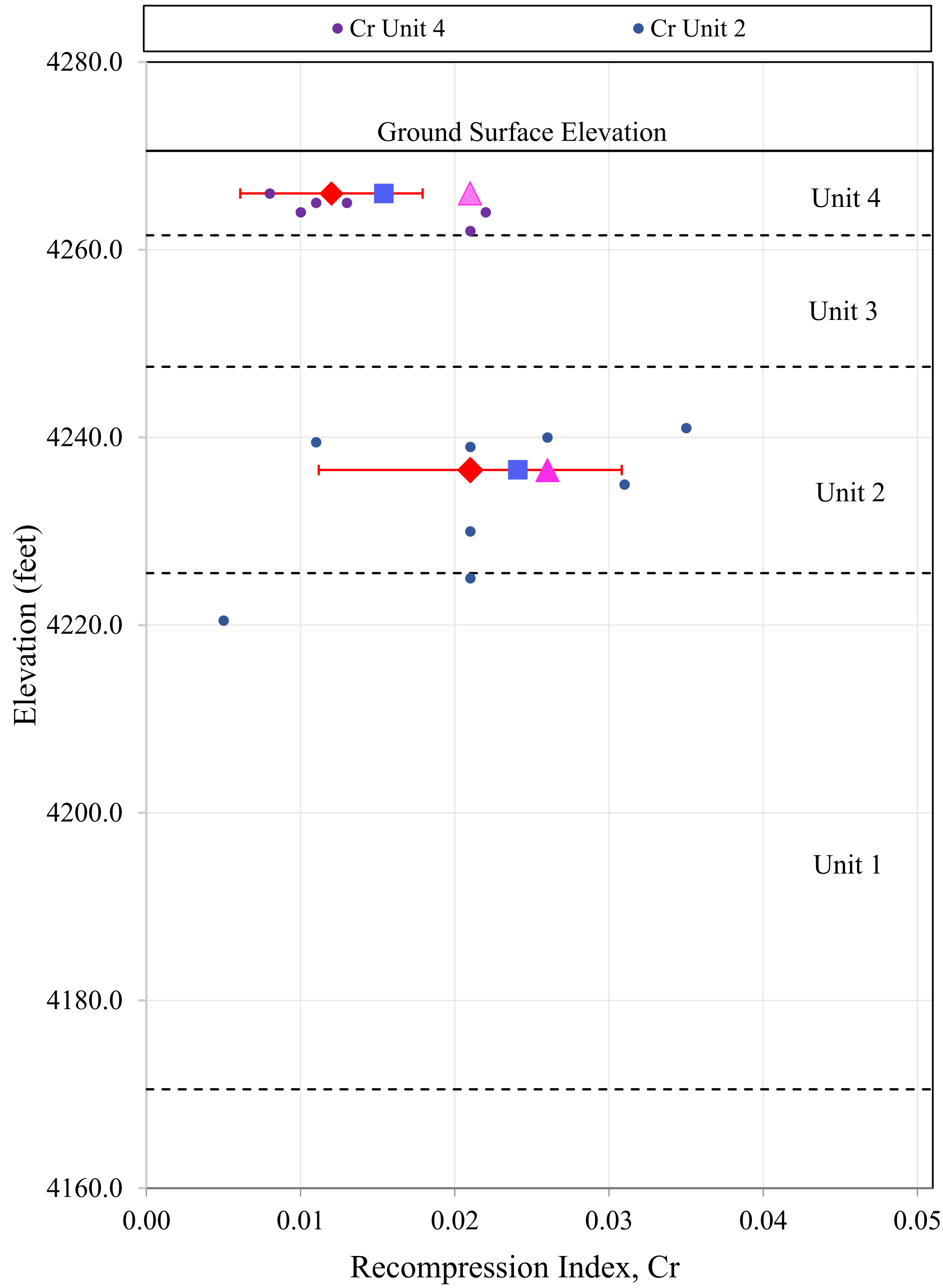
- MEDIAN
- SELECTED VALUE
- 66th PERCENTILE
- +- 1 STANDARD DEVIATION

COMPRESSION INDEX STATISTICAL ANALYSIS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

Geosyntec consultants

Figure **6**

Project No: SLC1025 | JANUARY 2023



LEGEND:

- MEDIAN
- SELECTED VALUE
- 66th PERCENTILE
- +- 1 STANDARD DEVIATION

RECOMPRESSION INDEX STATISTICAL ANALYSIS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

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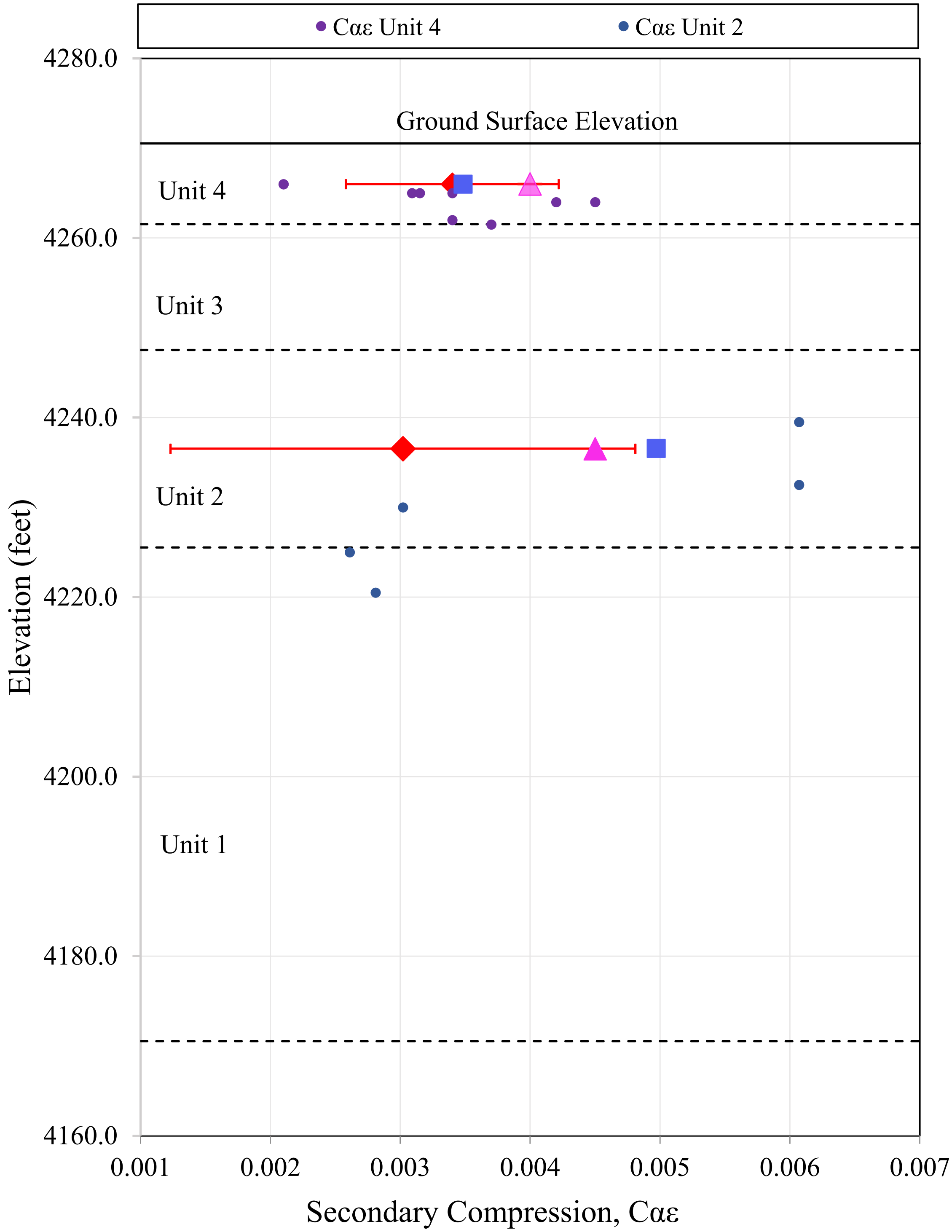
Figure

7

Project No: SLC1025

JANUARY 2023



LEGEND:

- MEDIAN
- SELECTED VALUE
- 66th PERCENTILE
- +- 1 STANDARD DEVIATION

SECONDARY COMPRESSION INDEX STATISTICAL ANALYSIS

CLIVE FACILITY FEDERAL CELL

CLIVE, UTAH

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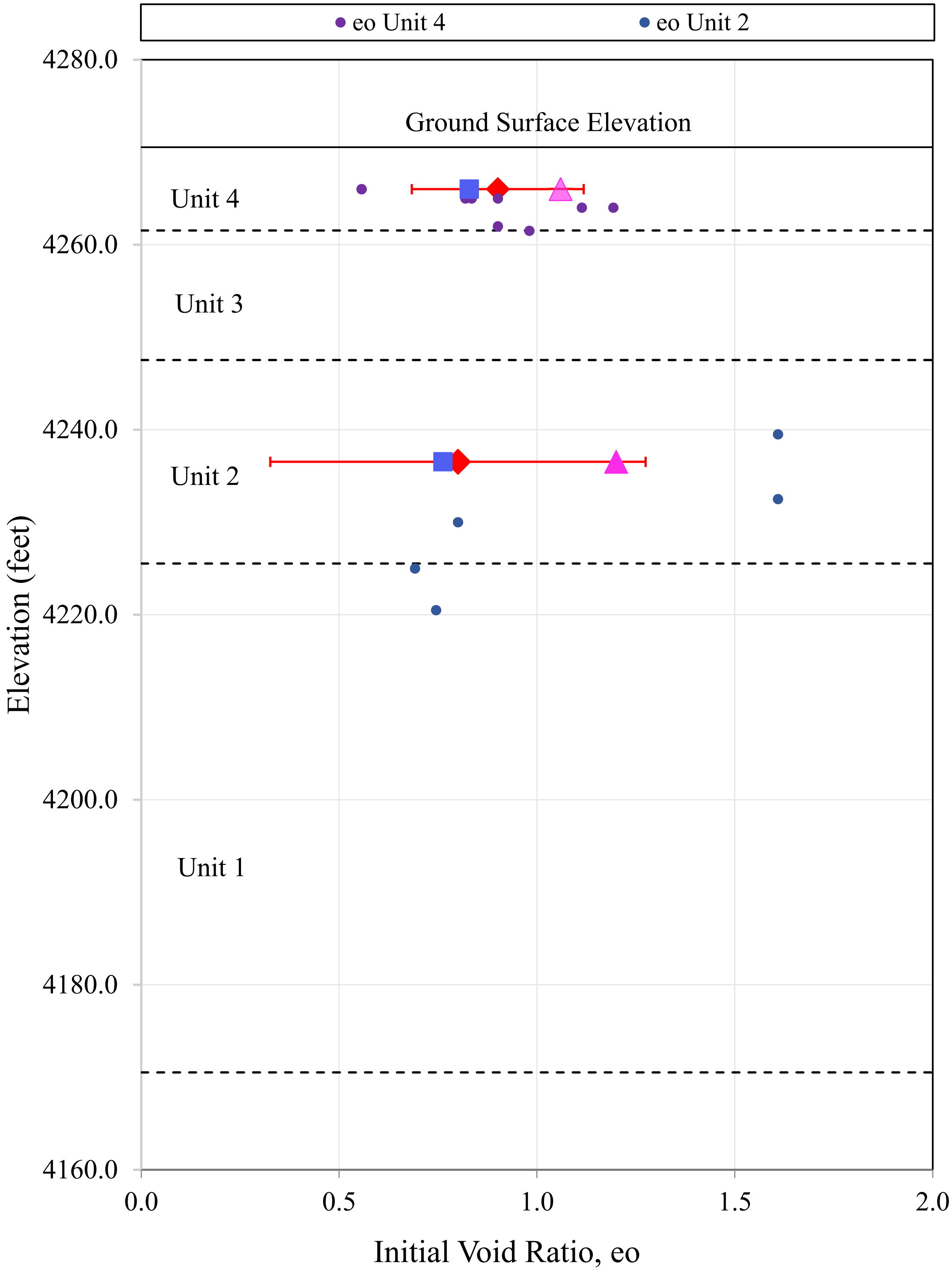
Figure **8**

Project No: SLC1025

JANUARY 2023



LEGEND:

- MEDIAN
- SELECTED VALUE
- 33RD PERCENTILE
- +- 1 STANDARD DEVIATION

INITIAL VOID RATIO STATISTICAL ANALYSIS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

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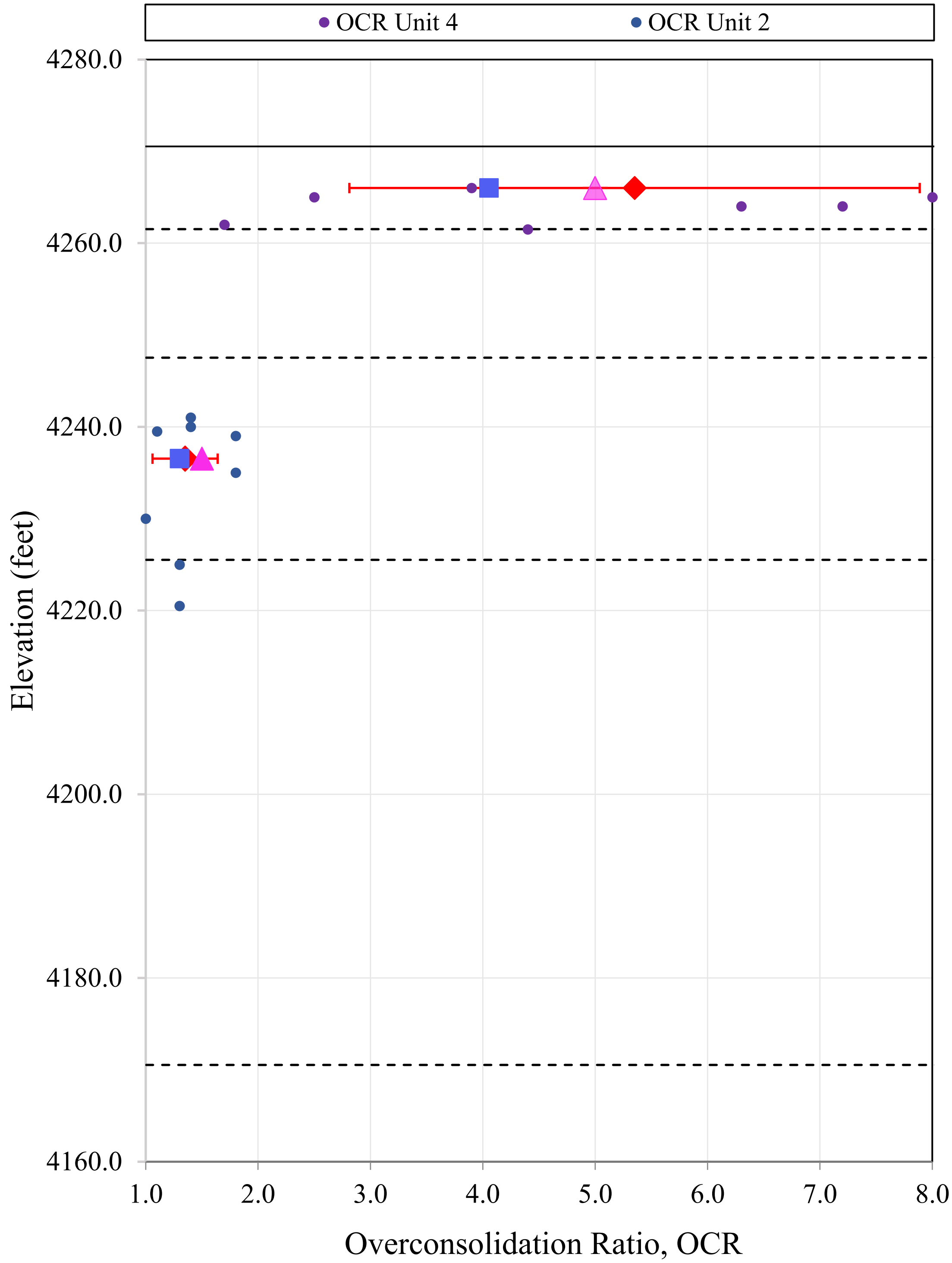
Figure **9**

Project No: SLC1025 | JANUARY 2023



LEGEND:

- MEDIAN
- SELECTED VALUE
- 33RD PERCENTILE
- +- 1 STANDARD DEVIATION

OVERCONSOLIDATION RATIO STATISTICAL ANALYSIS

CLIVE FACILITY FEDERAL CELL

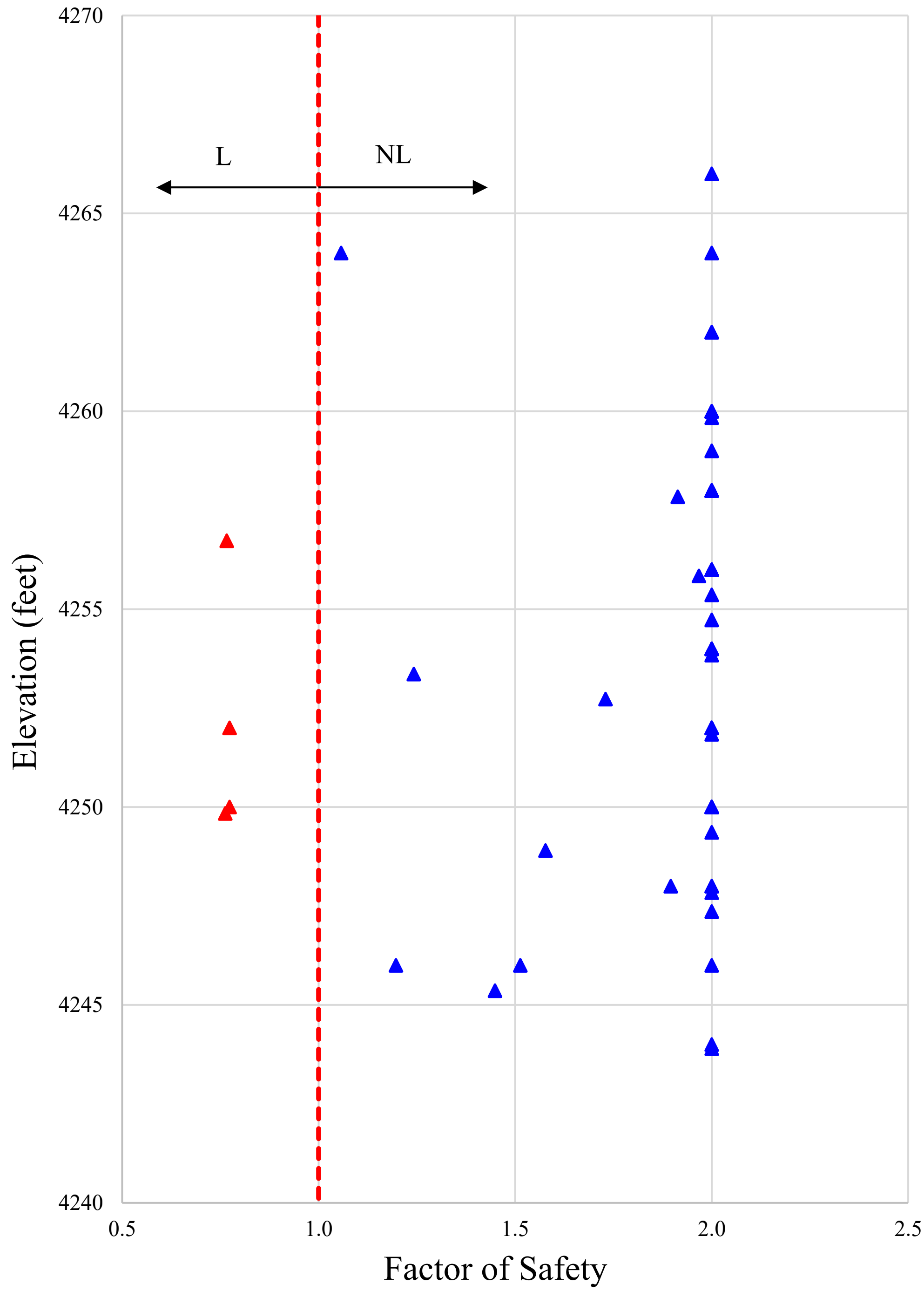
CLIVE, UTAH

Geosyntec consultants

Figure **10**

Project No: SLC1025

JANUARY 2023



Notes:

1. L = Liquefiable
2. NL= Non-liquefiable
3. Liquefaction triggering of Unit 3 sand-like soils is based on the occurrence of a groundwater rise condition of 20 feet.
4. Analysis is based on "Idriss and Boulanger (2008), Soil Liquefaction During Earthquakes, EERI Monograph MNO-12."
5. Evaluation reflects SPT-blow counts from borings GW-17A, -18, 19-A, -19B, -25, -26, -27, -28, -36, -37, -38 (Bingham Environmental, 1992).

LIQUEFACTION TRIGGERING OF UNIT 3 SAND-LIKE SOILS
CLIVE FACILITY FEDERAL CELL
CLIVE, UTAH

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Figure **11**

Project No: SLC1025 | JANUARY 2023

ATTACHMENT A

Table 1 - Summary of Envirocare Lab Data

Boring No.	Depth ft	Soil Description	USCS	W.C. %	γ_{dry} pcf	γ_{tot} pcf	Gradation % Gravel	% Sand	% Silt-clay	Atterberg Limits LL	PL	PI	Su (lab vane) psf	Cc	Cr	Pc psf	OCR	Source / Date
Foundation Soils																		
SC-1	40.0			32%	90	119				24	14	10		0.180	0.040	9500	2.4	D&M, 1984
SC-7	5.0			34%	85.4	114								0.120	0.012	1400	2.5	D&M, 1984
SC-7	35.0			40%	80.9	113								0.216	0.030	6200	1.8	D&M, 1984
SC-8	29.0			67%	60.6	101				54	40	14		0.228	0.034	4000	1.4	D&M, 1984
SC-10	10.0			44%	77.9	112								0.240	0.048	6000	5.4	D&M, 1984
SLC-84	4.0			21%	98.8	119				38	28	10		0.075		4000	8.4	D&M, 1984
SLC-84	8.5			37%	77.6	106				20	16	4		0.239		4000	4.4	D&M, 1984
SLC-84	6.0			42%	65.2	92.3				38	25	13		0.441	0.009	4000	7.2	D&M, 1984
GW-16	4.0	Silty Clay		21%	106	128				34	19	15		0.500	0.007	2000	3.9	Bingham Eng
GW-19A	6.0	Silty Clay		45%	73.2	106				44	26	18		0.160	0.021	4000	6.3	Bingham Eng
GW-19B	23.0	Silty Sand		33%	112	149												Bingham Eng
GW-18	31.0	Silty Clay		46%	79	115				40	22	18		0.100	0.020	6200	1.8	Bingham Eng
GW-17A	8.3	Silty Clay		34%	87.2	117				34	20	14		0.100	0.020	1600	1.7	Bingham Eng
GW-17A	20.0	Silty Sand		16%	104	121												Bingham Eng
GW-17A	28.0	Silty Sand		49%	82.4	123												Bingham Eng
GW-18	5.8									34	21	13						Bingham Eng
GW-18	21.0	Silty Sand		27%	96.7	123												Bingham Eng
GW-16	35.2	Silty Clay		38%	104	143				19	16	3						Bingham Eng
GW-24	6.5									33	21	12						Bingham Eng
GW-24	26.5									30	19	11						Bingham Eng
GW-24	31.5									41	24	17						Bingham Eng
GW-27	9.0									36	23	13						Bingham Eng
GW-27	29.0									36	20	16						Bingham Eng
GW-29	4.0									54	26	28						Bingham Eng
GW-29	9.0									35	22	13						Bingham Eng
GW-29	23.5									28	18	10						Bingham Eng
GW-29	31.5									40	23	17						Bingham Eng
DH-33	4.0									48	24	24						Bingham Eng
DH-33	9.0									39	25	14						Bingham Eng
DH-33	26.5									23	18	5						Bingham Eng
SW-36	5.0									33	21	12						Bingham Eng
GW-36	27.0									44	31	13						Bingham Eng
GW-41	25.0									27	18	9						Bingham Eng
GW-55	7.0									33	21	12						Bingham Eng
DH-51	21.0									24	15	9						Bingham Eng
DH-51	25.0									25	17	8						Bingham Eng
DH-48	26.0									29	20	9						Bingham Eng
DH-1	5		CL-ML	30.9%	87.8	115				35	20	15		0.13	0.01	4600	8	AGRA, 1999
DH-1	12		SM	6.1%			9	77	14									AGRA, 1999
DH-1	22		SM	16.7%				83	17									AGRA, 1999
DH-1	30.5		CL-ML	64.0%	59.9	98.2				43	22	21	920	0.13	0.025	4000	1.4	AGRA, 1999
DH-1	40		CL-ML	30.2%	93.3	121				33	17	16	2800	0.125	0.02	4200	1	AGRA, 1999
DH-1	45		CL-ML	26.1%	98.8	125				28	19	9	1985	0.08	0.02	6000	1.3	AGRA, 1999
DH-1	60		CL-ML	28.6%	92.2	119				28	23	5	1615	0.08	0.02	6000	1.2	AGRA, 1999
DH-1	69		CL-ML	28.0%			1	42	57									AGRA, 1999
DH-1	80		SM	16.7%			13	67	20									AGRA, 1999
DH-1	93		SM	23.1%				57	43									AGRA, 1999
B-2	5		CL	31.5%	85.9	113.0				39	24	15						AMEC, 2004
B-2	10.5		SM	3.4%	112	116.0												AMEC, 2004
B-2	15.5		SM	11.5%	128	142.8												AMEC, 2004
B-2	20.5		SM	19.1%	101	120.8												AMEC, 2004
B-2	23.5		SM	12.6%	102	115.3												AMEC, 2004
B-2	30.5		CL	60.7%	64.6	103.9				48	28	21		0.22	0.01	3200	1.1	AMEC, 2004
B-2	37.5		SC	60.7%	77.2	124.1												AMEC, 2004
B-2	49.5		CL	28.1%	94.1	120.5								0.125	0.004	6000	1.3	AMEC, 2004
B-2	60.5		CL	29.5%	91.8	118.8												AMEC, 2004
B-2	70.5		SC	16.6%	111	129.6												AMEC, 2004
B-2	80.5		SM	22.6%	105	128.5												AMEC, 2004
B-2	90.5		SM	22.7%	103	125.7												AMEC, 2004
B-2	99		CL	27.8%	93.4	119.3												AMEC, 2004

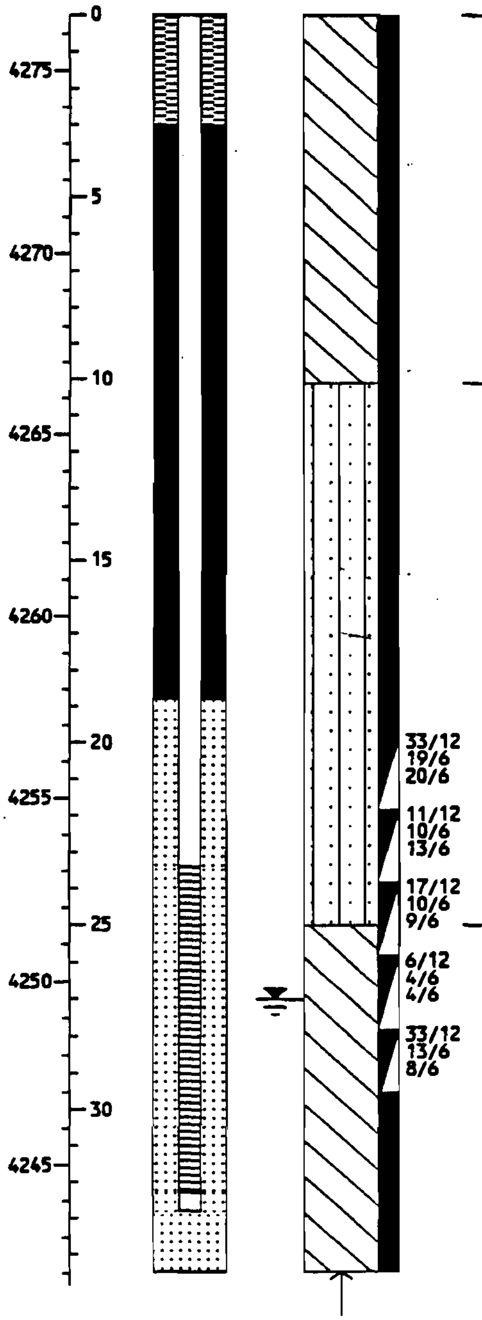
DRILL HOLE LOG

DRILL HOLE NO.: GW-17A

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Northwest corner of LARW cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 27.0
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-8-91
TOC ELEV.: 4278.22
GS ELEV.: 4276.53
LOGGED BY: MT
HOLE NO.: GW-17A

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Light brown grades to tannish gray, slightly silty and sandy, iron oxide staining, soft, moist.	L-1	0.0-4.5	48/54
		L-2	4.5-9.5	48/60
	...grades to light gray clay, with silt lenses, iron oxide staining, very moist.			
		L-3	9.5-14.5	24/60
SM	SILTY SAND: Tan, very silty, fine, medium dense, moist.			
		L-4	14.5-19.5	14/60
		L-5	19.5-24.5	30/60
		B-1	19.8-21.8	24/24
		B-2	21.8-23.8	22/24
		B-3	23.8-25.8	24/24
		L-6	24.5-29.5	40/60
CL	SILTY CLAY: Reddish tan, very sandy, medium stiff, moist.	B-4	25.8-27.8	24/24
		B-5	27.8-29.8	23/24
	...sand and clay interbedding ...grades to light gray to white, stiff, moist.	L-7	29.5-34.5	60/60
	...grades to greenish gray clay, stiff, very moist.			

Soil and sample data from 19.8' to 29.8' came from exploratory drill hole adjacent to GW-17A to verify soil stratigraphy.

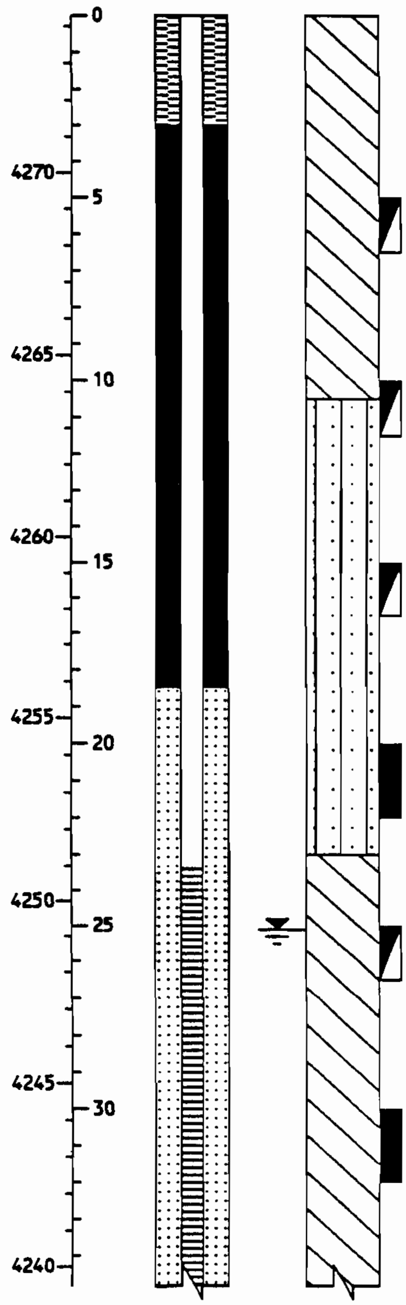
Figure No.

BINGHAM ENVIRONMENTAL

DRILL HOLE LOG
DRILL HOLE NO.: GW-18

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Near SW Corner of LARW Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 25.1
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-9-91
TOC ELEV.: 4276.17
GS ELEV.: 4274.31
LOGGED BY: MT
HOLE NO.: GW-18



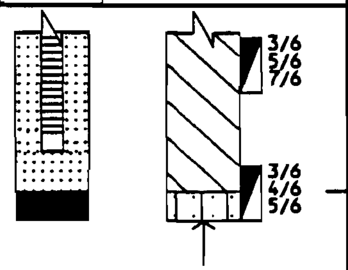
Depth (ft)	Field Test Data	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
0		CL	SILTY CLAY: Light brown grading to tannish gray, slightly silty and sandy, iron oxide staining, soft, moist.			
5	2/6 2/6 2/6		...grades to tan gray clay with iron oxide staining, moist.	B-1	5-6.5	18/18
10	3/6 8/6 6/6			B-2	10-11.5	18/18
10.5		SM	SILTY SAND: Tannish gray, clayey, silty with occasional clay lenses, medium dense, slighlty moist.			
15	10/6 15/6 21/6			B-3	15-16.5	17/18
20				S-4	20-22	24/24
23		CL	SILTY CLAY: Reddish tan, with sand lenses, stiff, slightly wet.			
25	4/6 10/6 12/6			B-5	25-26.5	18/18
30			...grades to white/light gray, silty lenses, defined bedding, soft, wet.	S-6	30-32	24/24

Figure No.

DRILL HOLE LOG
DRILL HOLE NO.: GW-18

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Near SW Corner of LARW Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 25.1
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-9-91
TOC ELEV.: 4276.17
GS ELEV.: 4274.31
LOGGED BY: MT
HOLE NO.: GW-18



ELEVATION / DEPTH	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
35		...grades to light greenish gray clay with clayey sand lenses, very moist.	B-7	35-36.5	18/18
4235			B-8	38.5-40	18/18
40	SM	SILTY SAND: Greenish gray, clayey, medium dense, moist.			

Elevation scale: 4235, 4230, 4225, 4220, 4215, 4210, 4205
Depth scale: 35, 40, 45, 50, 55, 60, 65

Figure No.

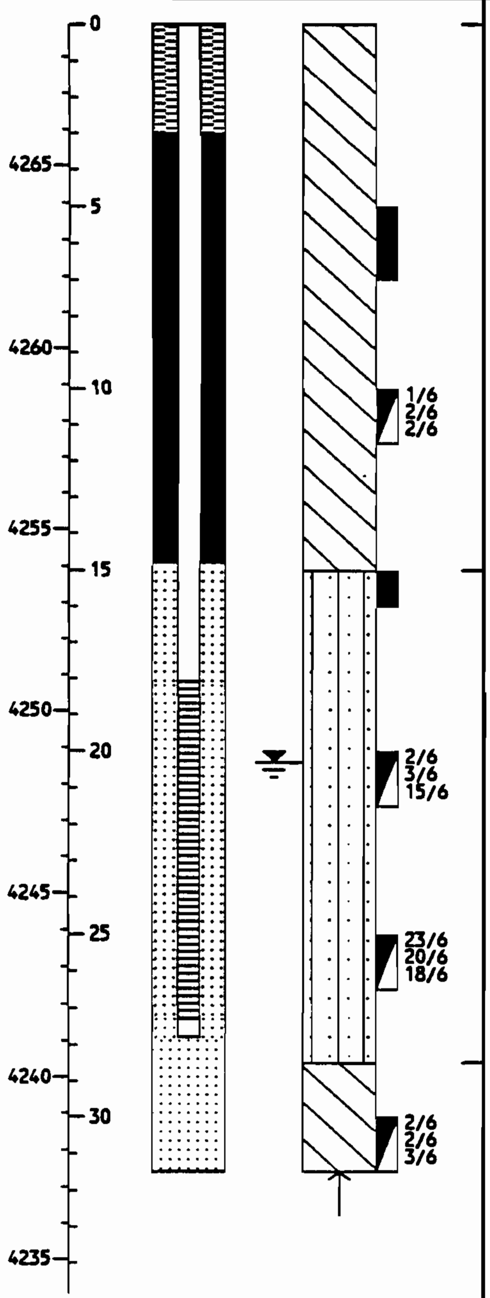
DRILL HOLE LOG

DRILL HOLE NO.: GW-19A

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: SW Corner of Section 32
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 20.3
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-7-91
TOC ELEV.: 4270.41
GS ELEV.: 4268.89
LOGGED BY: MT
HOLE NO.: GW-19A

ELEVATION / DEPTH — WELL DETAILS — SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Light brown, silty, sandy, dry.			
	...grades to light gray clay with silt lenses, soft, moist.			
		S-1	5-7	24/24
		B-2	10-11.5	18/18
SM	SILTY SAND: Tan, silty, dense, moist. * Shelby sample S-3 refused after 12 inches.	S-3	15-16	12/12
		B-4	20-21.5	18/18
	...grades to gray silty sand, medium dense, wet.			
	...grades to tan silty sand with occasional sandy clay lenses, dense, wet.	B-5	25-26.5	18/18
CL	SILTY CLAY: Greenish gray, slightly silty, medium stiff, very moist.	B-6	30-31.5	18/18

Figure No.

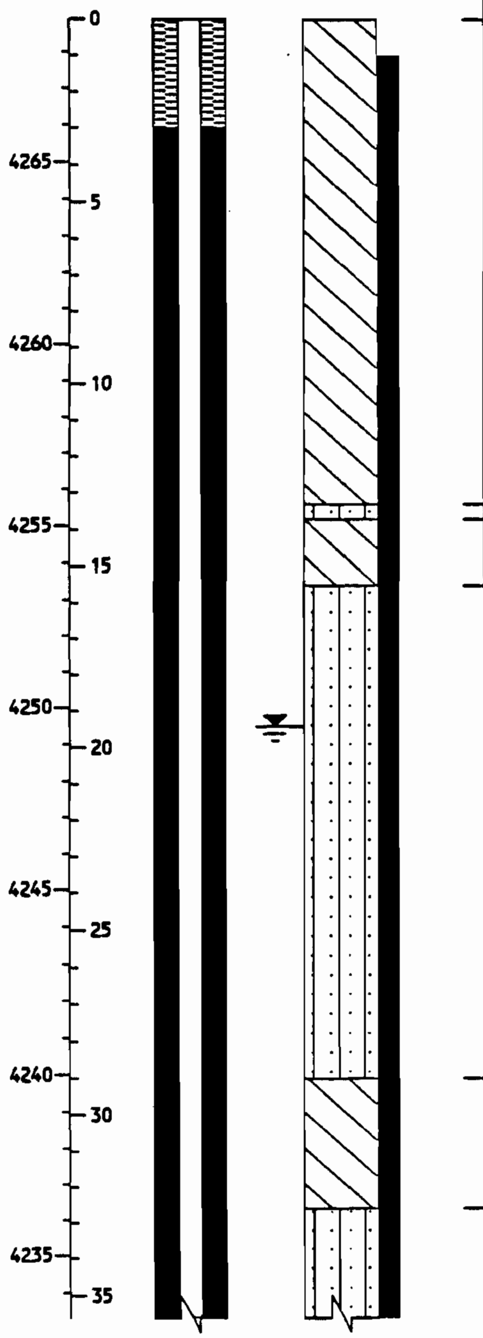
BINGHAM ENVIRONMENTAL

DRILL HOLE LOG
DRILL HOLE NO.: GW-19B

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: SW Corner of Section 32
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 19.4 HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-4,6-91
TOC ELEV.: 4270.43
GS ELEV.: 4268.91
LOGGED BY: MT
HOLE NO.: GW-19B

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



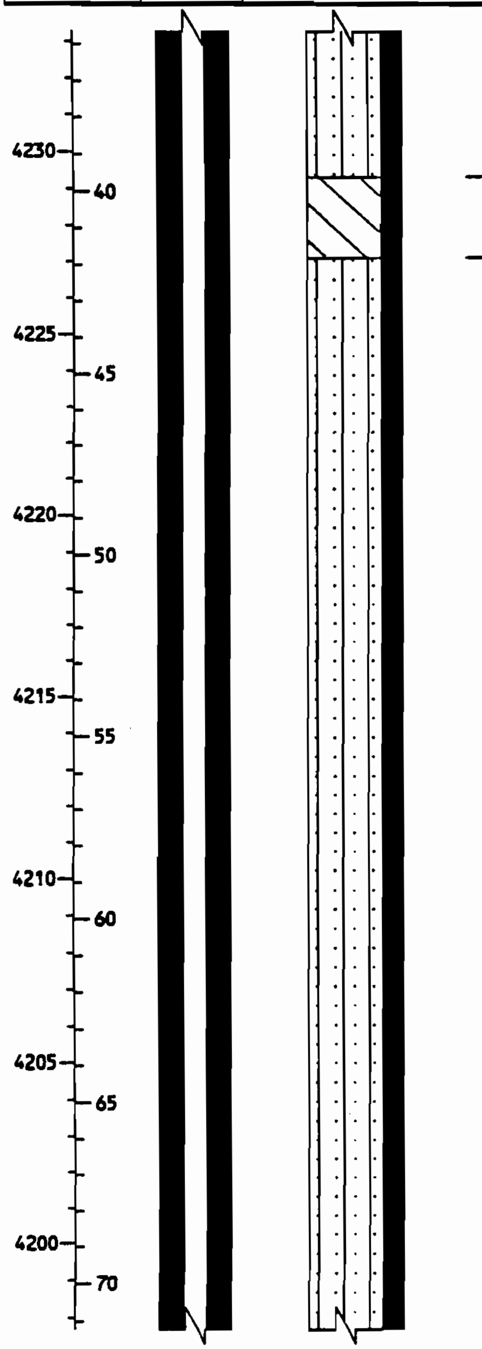
USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Light brown grading to light gray, slightly silty and sandy, gypsum crystals, moist.	L-1	1-4.5	37/42
	...grades to light gray clay with occasional sand lenses, gypsum crystals, moist.	L-2	4.5-9.5	59/60
		L-3	9.5-14.5	60/60
SM	SILTY SAND: Tan, silty, loose, moist.			
CL	SILTY CLAY: Light gray, silty lenses, bedding, moist.	L-4	14.5-19.5	42/60
SM	SILTY SAND: Tan, silty, occasional silt lenses, fine to medium course, loose, moist.			
		L-5	19.5-24.5	58/60
	...grades to brownish gray sand, wet.			
		L-6	24.5-29.5	50/60
CL	SILTY CLAY: Greenish gray, silty, stiff, moist.	L-7	29.5-34.5	56/60
SM	SILTY SAND: Light greenish gray, fine to medium course, medium dense, wet.	L-8	34.5-39.5	60/60

DRILL HOLE LOG

DRILL HOLE NO.: GW-19B

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: SW Corner of Section 32
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 19.4 HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-4,6-91
TOC ELEV.: 4270.43
GS ELEV.: 4268.91
LOGGED BY: MT
HOLE NO.: GW-19B



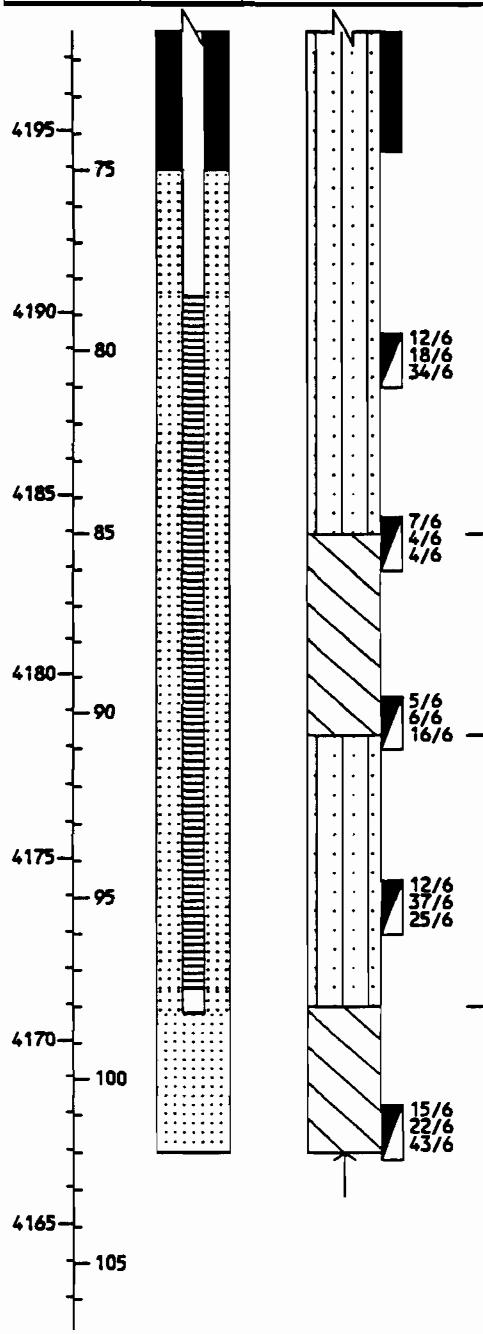
USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Light gray, slightly silty, with occasional sand lenses, very moist, stiff.	L-9	39.5-44.5	60/60
SM	SILTY SAND: Greenish gray, silty, fine, medium dense, wet.			
	...grades to tan sand, wet.	L-10	44.5-49.5	35/60
	...hard drilling between 47.0' and 49.5' (cemented sands).			
		L-11	49.5-54.5	60/60
		L-12	54.5-59.5	54/60
		L-13	59.5-64.5	41/60
		L-14	64.5-69.5	60/60
	...grades to a greenish gray sand with cemented silty clay lenses, wet.	L-15	69.5-74.5	48/60

DRILL HOLE LOG
DRILL HOLE NO.: GW-19B

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: SW Corner of Section 32
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 19.4
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-005
DATE: 2-4,6-91
TOC ELEV.: 4270.43
GS ELEV.: 4268.91
LOGGED BY: MT
HOLE NO.: GW-19B

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
	...very hard drilling between 73.0' and 79.0', out of cemented sand lenses at 79.0'.			
		B-16	79.5-81	17/18
		B-17	84.5-86	2/18
CL	SILTY CLAY: Light gray, slightly silty, stiff, plastic, very moist.			
		B-18	89.5-91	18/18
SM	SILTY SAND: Tan, silty, fine, dense, wet.			
		B-19	94.5-96	18/18
CL	SILTY CLAY: Light tan, very silty, slightly sandy, very stiff, cemented lenses, very moist.			
		B-20	100.7-102.2	18/18

Figure No.

BINGHAM ENVIRONMENTAL

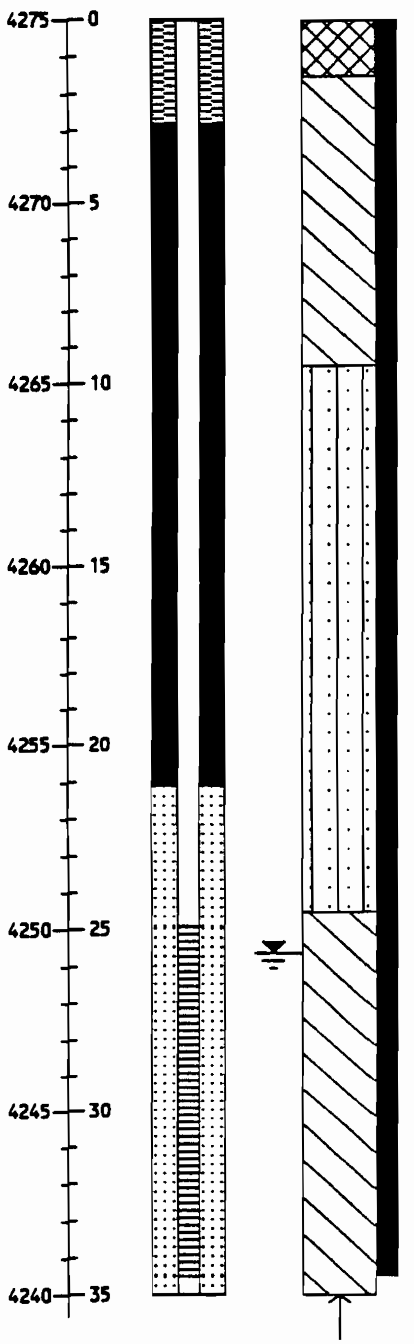
DRILL HOLE LOG

DRILL HOLE NO.: GW-20

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: SW Corner of Controlled Area
DRILLER: Overland Drilling Company
DRILL RIG: CME 750
DEPTH TO WATER: 25.6 HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-2-91
TOC ELEV.: 4276.59
GS ELEV.: 4275.04
LOGGED BY: DCH
HOLE NO.: GW-20

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
GM	FILL: Gray and light tan, gravelly sand, slity, moist.	L-1	0-2	6/24
CL	SILTY CLAY: Brown, slightly sandy, iron oxide staining.	L-2	2-4.5	30/30
	...grades to light gray.	L-3	4.5-7	27/30
		L-4	7-9.5	30/30
SM	SILTY SAND: Tan, fine to medium, moist.	L-5	9.5-12	11/30
		L-6	12-14.5	0/30
		L-7	14.5-17	18/30
		L-8	17-19.5	0/30
		L-9	19.5-22	13/30
	...grades clayey.	L-10	22-24.5	30/30
CL	SILTY CLAY: Reddish tan, sandy, medium stiff, moist.	L-11	24.5-27	13/30
		L-12	27-29.5	30/30
	...grades to light gray/white, stiff, moist.	L-13	29.5-32	30/30
		L-14	32-34.5	30/30
	...grades very moist.			

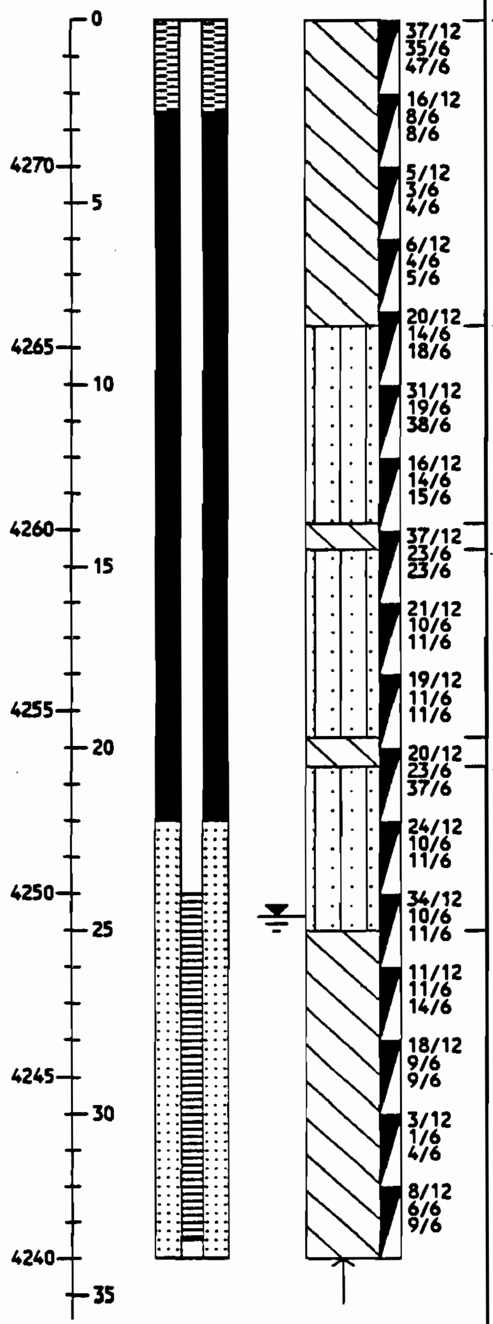
Figure No.

DRILL HOLE LOG
DRILL HOLE NO.: GW-25

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: North Boundary of Future Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 24.6 HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-19-91
TOC ELEV.: 4275.74
GS ELEV.: 4273.99
LOGGED BY: DA
HOLE NO.: GW-25

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Brown, slightly sandy, very hard, moist.	B-1	0-2	16/24
	...grades to very stiff.	B-2	2-4	18/24
		B-3	4-6	23/24
	...grades to grayish white and stiff.	B-4	6-8	24/24
		B-5	8-10	24/24
SM	SILTY SAND: Light brown, fine, medium dense, moist.	B-6	10-12	24/24
	...trace of fine gravel.	B-7	12-14	20/24
CL	SILTY CLAY: Sandy clay lense.	B-8	14-16	24/24
SM	SILTY SAND: Light gray, fine, dense, moist.	B-9	16-18	21/24
	...grades to light brown.	B-10	18-20	24/24
CL	SILTY CLAY: Brown, some fine sand, stiff, moist.	B-11	20-22	24/24
SM	SILTY SAND: Brown, fine, very dense, moist.	B-12	22-24	24/24
	...grades to light gray, medium dense.	B-13	24-26	24/24
CL	SILTY CLAY: Light brown, sandy, very stiff, moist.	B-14	26-28	24/24
	...grades to light gray and wet.	B-15	28-30	24/24
	...grades to soft and wet.	B-16	30-32	24/24
		B-17	32-34	24/24
	...trace of fine sand, very stiff, wet.			

Figure No.

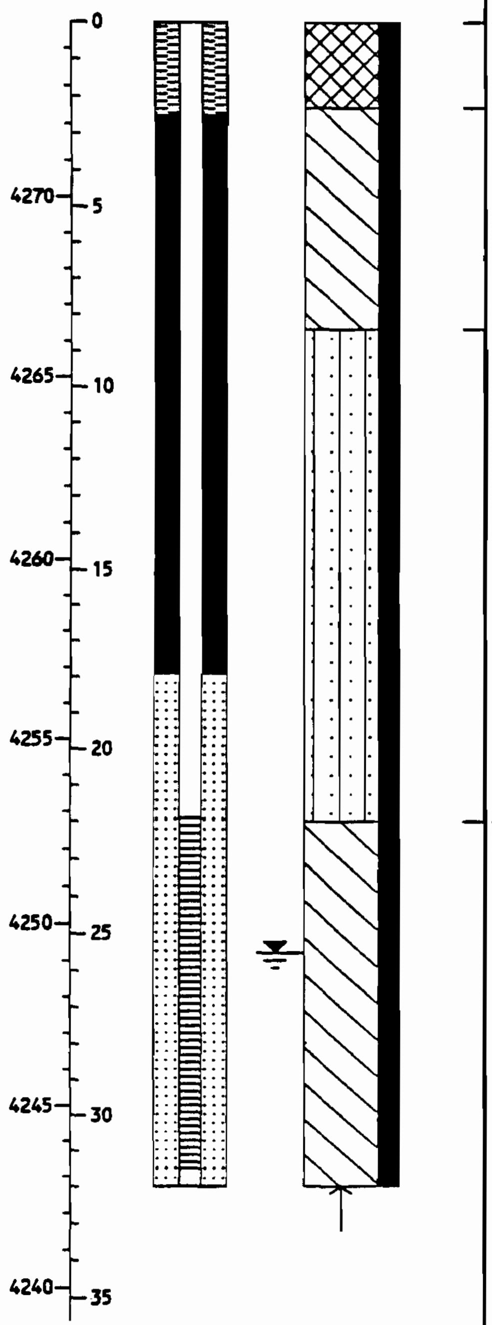
DRILL HOLE LOG

DRILL HOLE NO.: GW-23

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: North Boundary of LARW Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 25.5 HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-5-91
TOC ELEV.: 4276.51
GS ELEV.: 4274.73
LOGGED BY: DCH
HOLE NO.: GW-23

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
GM	FILL: Tan and brown gravel, some cobbles, moist.	L-1	0-2	18/24
CL	SILTY CLAY: Gray with iron oxide staining, trace fine sand, moist.	L-2	2.0-4.5	30/30
	sand, moist.	L-3	4.5-7	16/30
		L-4	7-9.5	30/30
SM	SILTY SAND: Tan, fine to medium, occasional sandy silt lenses, moist.	L-5	9.5-12	12/30
		L-6	12-14.5	0/30
	...occasional gray sandy clay lenses.	L-7	14.5-17	24/30
		L-8	17-19.5	0/30
	...grades to reddish tan sand with occasional clay lenses.	L-9	19.5-22	15/30
CL	SILTY CLAY: Reddish tan, sandy, medium stiff, moist.	L-10	22-24.5	30/30
		L-11	24.5-27	10/30
	...grades to light gray, soft, moist.	L-12	27-29.5	30/30
	...grades wet.	L-13	29.5-32	30/30

Figure No.

BINGHAM ENVIRONMENTAL

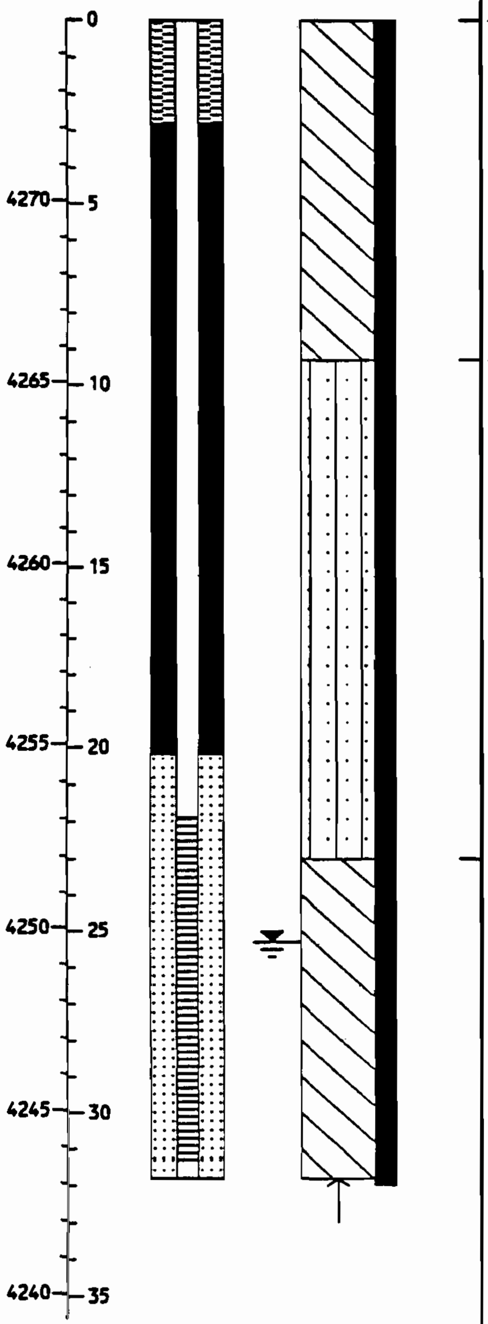
DRILL HOLE LOG

DRILL HOLE NO.: GW-24

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Northwest Corner of LARW Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 25.3
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-3-91
TOC ELEV.: 4276.59
GS ELEV.: 4274.91
LOGGED BY: DCH
HOLE NO.: GW-24

ELEVATION / DEPTH | WELL DETAILS | SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
CL	SILTY CLAY: Brown, trace of fine sand, moist.	L-1	0-2	17/24
	...grades to light gray with iron oxide staining.	L-2	2-4.5	30/30
		L-3	4.5-7	30/30
		L-4	7-9.5	30/30
SM	SILTY SAND: Tan, fine to medium, moist.	L-5	9.5-12	12/30
		L-6	12-14.5	0/30
	...grades less silty.	L-7	14.5-17	15/30
	...grades silty.	L-8	17-19.5	0/30
	...interbedded reddish tan and tan silty sand.	L-9	19.5-22	28/30
		L-10	22-24.5	0/30
CL	SILTY CLAY: Reddish tan, sandy, medium stiff, moist.	L-11	24.5-27	30/30
		L-12	27-29.5	30/30
	...grades to light gray, soft, moist.	L-13	29.5-32	30/30
	...grades to wet.			

Figure No.

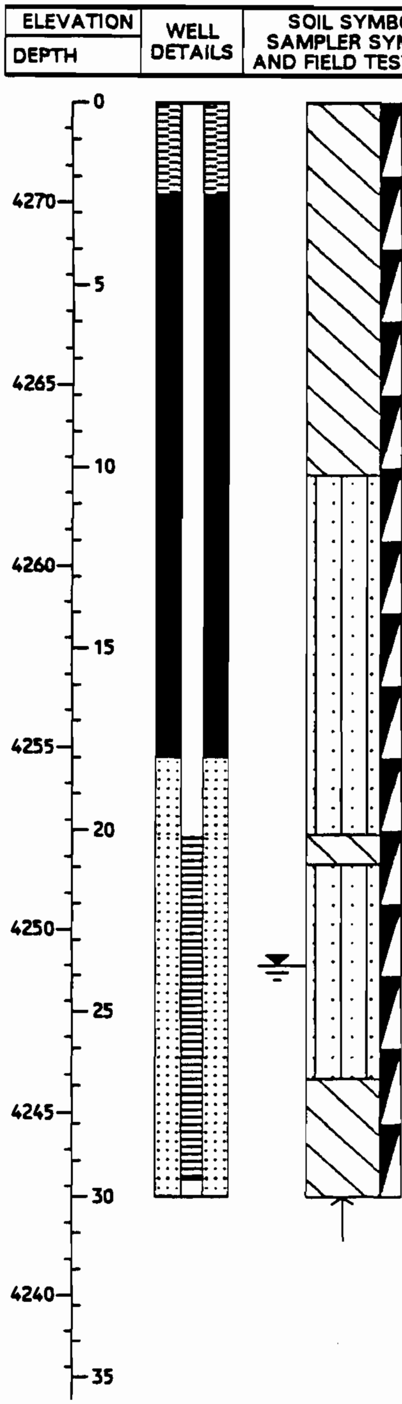
BINGHAM ENVIRONMENTAL

DRILL HOLE LOG

DRILL HOLE NO.: GW-26

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: North Boundary of Future Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 23.7
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-20-91
TOC ELEV.: 4274.16
GS ELEV.: 4272.71
LOGGED BY: DA
HOLE NO.: GW-26



Field Test Data	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
52/12, 43/6, 30/6	CL	SILTY CLAY: Brown, slightly sandy, very hard, moist.	B-1	0-2	12/24
10/12, 4/6, 4/6		...grades to light gray, stiff, moist.	B-2	2-4	21/24
12/12, 7/6, 8/6			B-3	4-6	24/24
4/12, 3/6, 4/6		...grades to grayish white with a trace of fine sand.	B-4	6-8	24/24
2/12, 2/6, 3/6			B-5	8-10	23/24
15/12, 9/6, 14/6	SM	SILTY SAND: Brownish gray, fine, medium dense, moist.	B-6	10-12	19/24
21/12, 21/6, 17/6		...grades to light gray, dense.	B-7	12-14	24/24
45/12, 58/6, 67/6			B-8	14-16	24/24
55/12, 28/6, 23/6			B-9	16-18	24/24
29/12, 20/6, 18/6			B-10	18-20	24/24
11/12, 10/6, 34/6	CL	SILTY CLAY: Light gray, fine sandy, stiff, moist.	B-11	20-22	24/24
65/12, 60/6, 46/6	SM	SILTY SAND: Light gray, fine, dense, moist.	B-12	22-24	18/24
47/12, 42/6, 30/6		...grades to light brown and very dense.	B-13	24-26	24/24
12/12, 9/6, 8/6			B-14	26-28	24/24
3/12, 6/6, 7/6	CL	SILTY CLAY: Light gray, slightly sandy, very stiff, wet. ...grades to grayish green with a trace of fine sand.	B-15	28-30	24/24

Figure No.

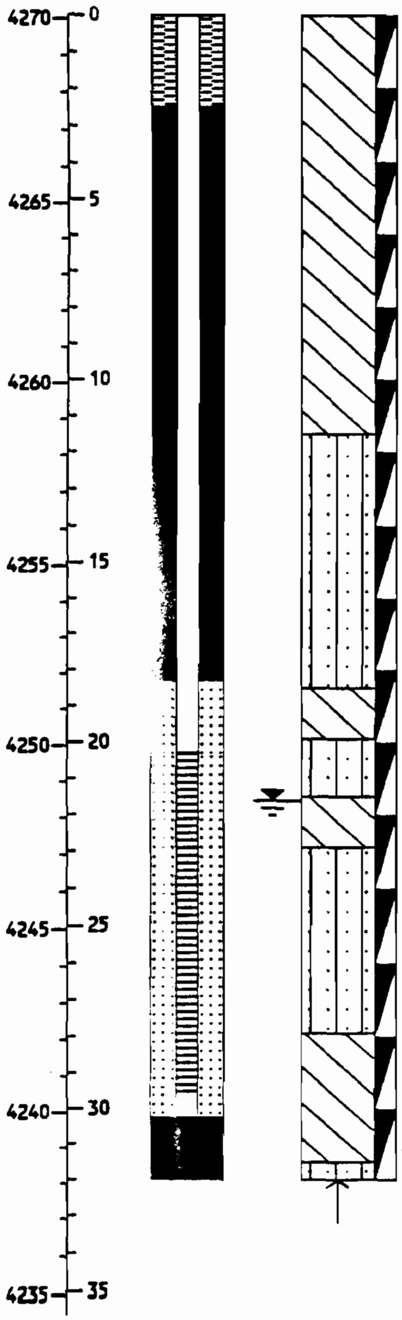
DRILL HOLE LOG

DRILL HOLE NO.: GW-27

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Northwest Corner of Future Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 21.6
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-11-91
TOC ELEV.: 4272.05
GS ELEV.: 4270.12
LOGGED BY: DH & DA
HOLE NO.: GW-27

ELEVATION / DEPTH — WELL DETAILS — SOIL SYMBOLS, SAMPLER SYMBOLS AND FIELD TEST DATA



Field Test Data	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
56/12 31/6 36/6	CL	SILTY CLAY: Brownish tan with iron oxide staining, silty, slightly sandy, moist.	B-1	0-2	6/24
27/12 16/6 15/6			B-2	2-4	5/24
9/12 6/6 7/6		...grades gray.	B-3	4-6	10/24
5/12 2/6 3/6		...thin bedding, roots.	B-4	6-8	24/24
4/12 3/6 3/6			B-5	8-10	24/24
2/12 5/6 8/6			B-6	10-12	24/24
20/12 12/6 21/6	SM	SILTY SAND: Tan, fine to medium course, dense, moist.	B-7	12-14	22/24
60/12 18/6 21/6			B-8	14-16	21/24
19/12 32/6 19/6		...grades light gray.	B-9	16-18	22/24
9/12 4/6 9/6			B-10	18-20	24/24
	CL	SILTY CLAY: Light gray, slightly sandy, stiff, moist.			
18/12 10/6 11/6	SM	SILTY SAND: Light gray, fine, medium dense, moist grading to very moist.	B-11	20-22	24/24
18/12 31/6 29/6	CL	SILTY CLAY: Light gray, slightly sandy, very stiff, moist.	B-12	22-24	24/24
	SM	SILTY SAND: Gray, fine, dense, very moist.			
23/12 35/6 58/6			B-13	24-26	22/24
26/12 15/6 15/6			B-14	26-28	24/24
6/12 4/6 4/6	CL	SILTY CLAY: Green, trace of fine sand, medium stiff, wet.	B-15	28-30	24/24
2/12 3/6 6/6			B-16	30-32	24/24
	SM	SILTY SAND: Greenish gray, clayey, medium dense, wet.			

Figure No.

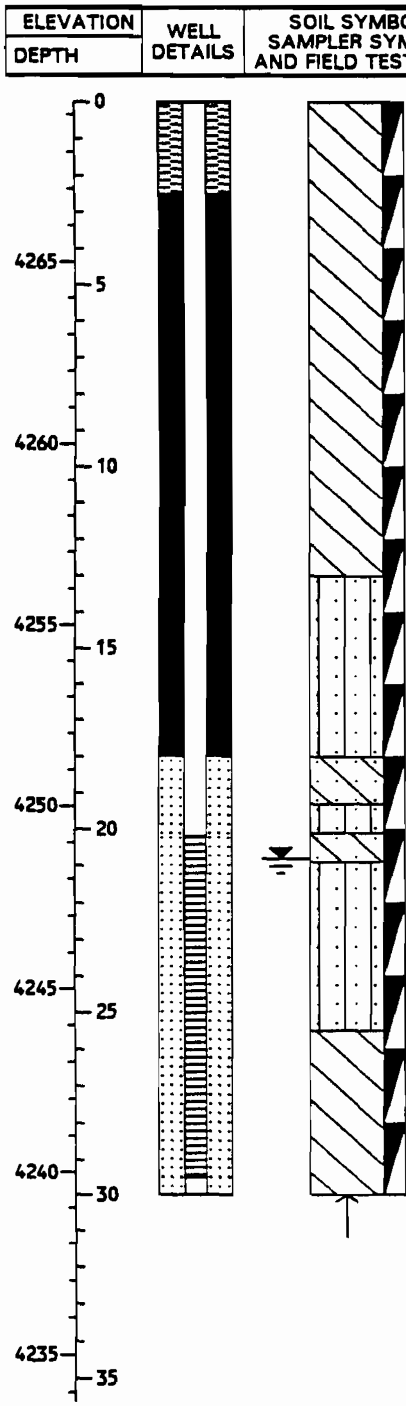
BINGHAM ENVIRONMENTAL

DRILL HOLE LOG

DRILL HOLE NO.: GW-28

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: West Boundary of Future Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 20.8
HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-17-91
TOC ELEV.: 4271.13
GS ELEV.: 4269.36
LOGGED BY: DA
HOLE NO.: GW-28



Field Test Data	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
31/12 39/6 42/6	CL	SILTY CLAY: Light brown, slightly sandy, very stiff, moist.	B-1	0-2	10/24
7/12 5/6 5/6			B-2	2-4	14/24
5/12 3/6 3/6			B-3	4-6	19/24
2/12 2/6 2/6			B-4	6-8	24/24
3/12 2/6 2/6		...grades to light gray, soft, very moist.	B-5	8-10	24/24
2/12 1/6 4/6			B-6	10-12	24/24
14/12 10/6 11/6		...grades stiff.	B-7	12-14	22/24
58/12 51/6 41/6	SM	SILTY SAND: Light gray, fine, medium dense, moist. ...grades wet.	B-8	14-16	22/24
20/12 9/6 8/6			B-9	16-18	23/24
3/12 7/6 11/6	CL	SANDY CLAY: Light gray, with sand lenses, soft, wet.	B-10	18-20	24/24
16/12 47/6 63/6	SM	SILTY SAND: Fine, dense, moist.	B-11	20-22	24/24
	CL	SANDY CLAY: Light gray, soft, wet.			
24/12 13/6 23/6	SM	SILTY SAND: Light brown, fine, very dense grading to medium dense, moist grading to wet.	B-12	22-24	24/24
24/12 9/6 9/6			B-13	24-26	24/24
3/12 1/6 3/6	CL	SILTY CLAY: Light gray, slightly sandy, moist. ...grades to grayish green, soft, very moist.	B-14	26-28	24/24
4/12 2/6 4/6		...grades wet.	B-15	28-30	24/24

Figure No.

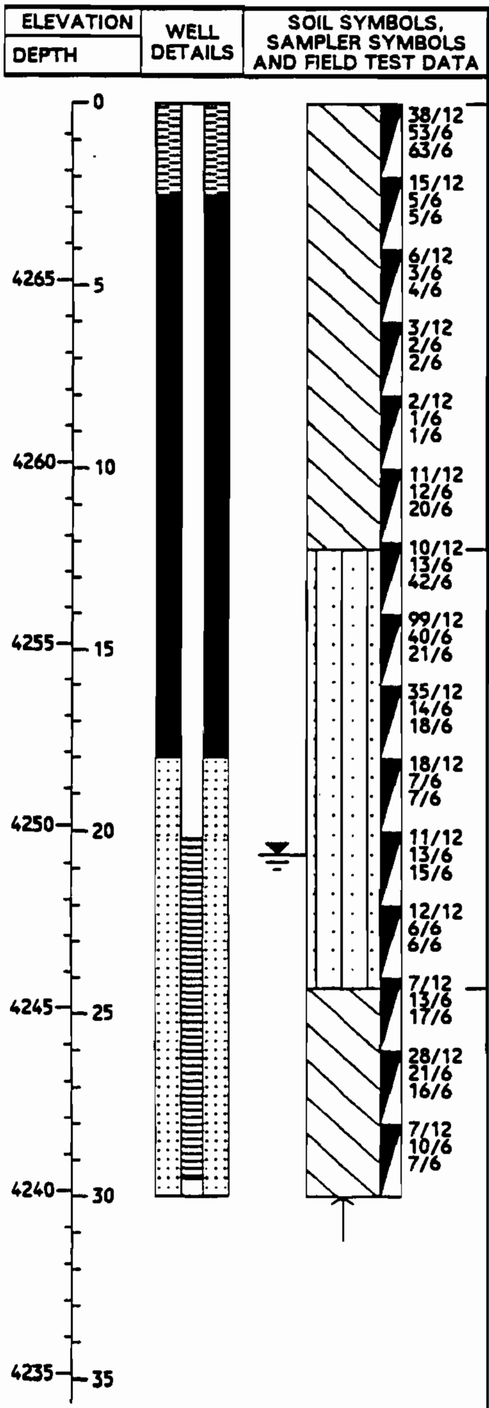
BINGHAM ENVIRONMENTAL

DRILL HOLE LOG

DRILL HOLE NO.: GW-36

PROJECT: Envirocare Landfill
CLIENT/OWNER: Envirocare of Utah
HOLE LOCATION: Future Disposal Cell
DRILLER: Overland Drilling
DRILL RIG: CME 750
DEPTH TO WATER: 20.6   HOLE DIAMETER: 7.75"

PROJECT NO.: 1416-020
DATE: 12-23-91
TOC ELEV.: 4271.59
GS ELEV.: 4269.84
LOGGED BY: DA
HOLE NO.: GW-36



Field Test Data	USCS	Description	Sample Number	Sample Depth (ft)	Recovery (in/in)
38/12 53/6 63/6	CL	SILTY CLAY: Brown, slightly sandy, very hard, moist.	B-1	0-2	12/24
15/12 5/6 5/6		...grades to light gray.	B-2	2.0-4	16/24
6/12 3/6 4/6		...grades very moist.	B-3	4.0-6	24/24
3/12 2/6 2/6		...grades to almost white.	B-4	6.0-8	24/24
2/12 1/6 1/6			B-5	8.0-10	24/24
11/12 12/6 20/6			B-6	10.0-12	24/24
10/12 13/6 42/6	SM	SILTY SAND: Light gray, fine, medium dense to dense, moist.	B-7	12.0-14	24/24
99/12 40/6 21/6			B-8	14.0-16	19/24
35/12 14/6 18/6			B-9	16.0-18	21/24
18/12 7/6 7/6		...grades to brown, wet.	B-10	18.0-20	23/24
11/12 13/6 15/6			B-11	20.0-22	22/24
12/12 6/6 6/6			B-12	22.0-24	24/24
7/12 13/6 17/6	CL	SILTY CLAY: Light gray, slightly sandy, stiff, very moist.	B-13	24.0-26	24/24
28/12 21/6 16/6		...grades silty, less plastic.	B-14	26.0-28	24/24
7/12 10/6 7/6		...grades clayey, more plastic. ...grades to dark gray with green clay, stiff, wet.	B-15	28.0-30	24/24

Figure No.

BINGHAM ENVIRONMENTAL

LOG OF TEST BORING NO. B-2

PROJECT: Envirocare Facility, Near Clive, Utah

JOB NO. 4-817-004769 DATE 10-07-04

RIG TYPE: Marl M-10
BORING TYPE: 4.25" Hollow-Stem Auger
SURFACE ELEV.:
DATUM:

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								CL	dry	**SILTY CLAY** with some fine sand; light gravel; no topsoil; roots present; open structure; loose to 6"; desiccated
5				D	7	86	31.5	CL	moist medium stiff	gray with oxidation stains
								SM	slightly moist medium dense	**SILTY SAND** with trace fine gravel; fine to medium sand; light brown
10				D	67	112	3.4			
15				D	54	128	11.5		moist	grades silty fine sand; light brown
20				D	32	101	19.1		slightly moist	occasional silty clay and silt layers
				D	54	102	12.6		moist	
25										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	37.3	11:00	10-07-04
▼			

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE B-6A

amec

LOG OF TEST BORING NO. B-2

PROJECT: Envirocare Facility, Near Clive, Utah
JOB NO. 4-817-004769 DATE 10-07-04

RIG TYPE: Marl M-10
BORING TYPE: 4.25" Hollow-Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25										
								CL	very moist to wet medium stiff	**SILTY CLAY**; gray a few sand layers and stringers throughout
30				D	8	65	60.7			
35										grades slightly cemented with fine sand; greenish-gray
				D	37	77	60.7	SC	saturated very stiff	**CLAYEY SAND** with some silt; fine sand; slightly cement; greenish-gray
40										
								CL	saturated stiff	**SILTY CLAY** with some fine sand layers; light brown
45										
				D	17	94	28.1			
50										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	37.3	11:00	10-07-04
▼			

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE B-6A (con't)

amec

LOG OF TEST BORING NO. B-2

PROJECT: Envirocare Facility, Near Clive, Utah

JOB NO. 4-817-004769 DATE 10-07-04

RIG TYPE: Marl M-10
BORING TYPE: 4.25" Hollow-Stem Auger
SURFACE ELEV.:
DATUM:

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
50										
55										
60				D	23	92	29.5		very stiff	grades fine sandy silty clay; brown
								SC	saturated dense	**CLAYEY SAND**; some cemented layers; brown
65										
70				D	80	111	16.6			
75										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	37.3	11:00	10-07-04
▼			

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE B-6A (con't)

amec

PROJECT Envirocare Facility
Near Clive, Utah

JOB NO. 4-817-004769 DATE 10-07-04

LOG OF TEST BORING NO. B-2

RIG TYPE Marl M-10
BORING TYPE 4.25" Hollow-Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
75										
								SM	saturated dense	**SILTY SAND**; brown
80				D	93	105	22.6			
85										
						93	27.8			
90				D	29	103	22.7		medium dense	grades with occasional fine gravel
95								CL	saturated hard	**SILTY CLAY** with occasional fine gravel and sand; brown
				D	68					
100										

GROUNDWATER

DEPTH	HOUR	DATE
37.3	11:00	10-07-04

SAMPLE TYPE
A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE B-6A
(con't)

amec

LOG OF TEST BORING NO. B-2

PROJECT: Envirocare Facility, Near Clive, Utah

JOB NO. 4-817-004769 DATE 10-07-04

RIG TYPE: Marl M-10

BORING TYPE: 4.25" Hollow-Stem Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
100										Stopped drilling at 98.5'. Stopped sampling at 100.0'.
105										
110										
115										
120										The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.
125										

GROUNDWATER

	DEPTH	HOUR	DATE
▽	37.3	11:00	10-07-04
▼			

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE B-6A (con't)

amec

LOG OF TEST BORING NO. B-1

PROJECT: Envirocare - New LARW Embankment, West Desert, near Clive, Utah

JOB NO. 9-817-002586 DATE 8/30/99

RIG TYPE: CME 550 All Terraine
BORING TYPE: 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV.: 4270 +/-'
DATUM: USGS

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								CL	dry	**SAND AND SILTY CLAY;** fine sand; no topsoil; brown
									moist	grades to gray
5			=	D	6	88	30.9	CL/ML		grades to layered silty clay to fine sandy silt, gray to light gray, seams to 1/4" thick
10								SC/SM	dense slightly moist	**CLAYEY TO SILTY SAND;** fine to medium sand; thin clay and silt layers; gray
			=	D	29		6.1		-200=14%	
									dry	
										grades to silty fine to medium sand; brown
15										
20										
			=	D	29		16.7		-200=17%	Thin silty clay layers grading to fine sandy silt and clay
25										

GROUNDWATER

DEPTH	HOUR	DATE
25.1		

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

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FIGURE A-1a

LOG OF TEST BORING NO. B-1

PROJECT: Envirocare - New LARW Embankment, West Desert, near Clive, Utah

JOB NO. 9-817-002586 DATE 8/30/99

RIG TYPE: CME 550 All Terraine
BORING TYPE: 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV.: 4270 +/-'
DATUM: USGS

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
25										
30			=	D	6				very moist to saturated soft	
								CL/ ML		SANDY SILT AND CLAY: layered clay and fine sandy silt to clay; sand laminations; 1/4" to 2" layers; light gray
35										
			=	D	25	93	30.2			grades with occasional fine to medium sand and silt layers
40										
45			=	D	23	99	26.1	CL	medium stiff to stiff	grades to layered fine sandy clay and sand to clay with some fine sand; brown
50										

GROUNDWATER

DEPTH	HOUR	DATE
25.1		

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-1b

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LOG OF TEST BORING NO. B-1

PROJECT Envirocare - New LARW Embankment
West Desert, near Clive, Utah

JOB NO. 9-817-002586 DATE 8/30/99

RIG TYPE CME 550 All Terraine
BORING TYPE 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV. 4270 +/-'
DATUM USGS

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
50										SILT AND SAND; layered silty fine to medium sand to fine silty sand; 1/2" to 4" layers? brown
			=	D	25			ML/SM	saturated	
55										grades to layered fine sand and clayey silt to silty clay with silty fine sand; brown
										SILTY CLAY AND SILT; medium; moist; brown
60			=	D	11	92	28.6	ML/CL		
								SM/ML	medium dense	SILT AND SAND; layered fine sandy silt to silty fine to medium sand with trace coarse sand; brown
65										
			=	D	25		28.0			
70										
75										

GROUNDWATER

DEPTH	HOUR	DATE
25.1		

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-1(

AGRA Earth & Environmental

LOG OF TEST BORING NO. B-1

PROJECT: Envirocare - New LARW Embankment, West Desert, near Clive, Utah

JOB NO. 9-817-002586 DATE 8/30/99

RIG TYPE: CME 550 All Terraine
BORING TYPE: 4-1/4" ID Hollow-Stem Auger
SURFACE ELEV.: 4270 +/-'
DATUM: USGS

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows/foot 140 lb. 30" free-fall drop hammer	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
75										
80				D	4		16.7	SM	disturbed	**SILTY SAND**; layered silty fine sand to silty fine to coarse sand with occasional fine sandy silt and clay layers; density increases with depth; gray
85										
									(loose)	
90										
				D	51		23.1		medium dense to dense	
95										Stopped drilling at 93.0'. Stopped sampling at 94.5'. The discussion in the text under the section titled, SUBSURFACE CONDITIONS, is necessary to a proper understanding of the nature of the subsurface materials.
100										

GROUNDWATER

DEPTH	HOUR	DATE
25.1		

SAMPLE TYPE

A - Auger cuttings
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.
D - 3 1/4" O.D. 2.42" I.D. tube sample.
C - California Split Spoon Sample

FIGURE A-1d

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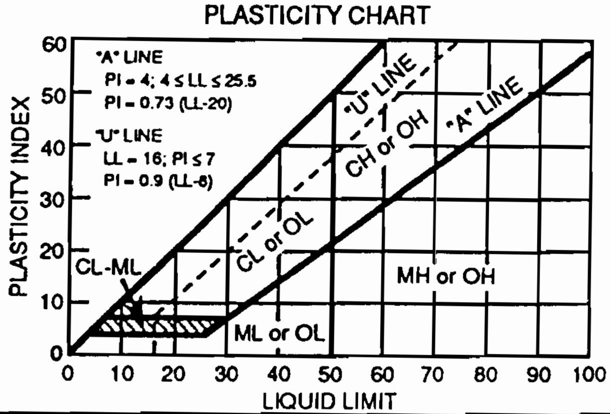
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified for engineering purposes by the Unified Soil Classification System. Grain-size analyses and Atterberg Limits tests often are performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. Graphic symbols are used on boring logs presented in this report. For a more detailed description of the system, see "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)" ASTM Designation: 2488-84 and "Standard Test Method for Classification of Soils for Engineering Purposes" ASTM Designation: 2487-85.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)			SW	Well graded sands, gravelly sands
					SP	Poorly graded sands, gravelly sands
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS (Limits plot below "A" line & hatched zone on plasticity chart)	SILTS OF LOW PLASTICITY (Liquid Limit less than 50)			ML	Inorganic silts, clayey silts of low to medium plasticity
		SILTS OF HIGH PLASTICITY (Liquid Limit 50 or more)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	CLAYS (Limits plot above "A" line & hatched zone on plasticity chart)	CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and silty clays
		CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity
	ORGANIC SILTS AND CLAYS	ORGANIC SILTS AND CLAYS OF LOW PLASTICITY (Liquid Limit less than 50)			OL	Organic silts and clays of low to medium plasticity, sandy organic silts and clays
		ORGANIC SILTS AND CLAYS OF HIGH PLASTICITY (Liquid Limit 50 or more)			OH	Organic silts and clays of high plasticity, sandy organic silts and clays
ORGANIC SOILS		PRIMARILY ORGANIC MATTER (dark in color and organic odor)			PT	Peat

NOTE: Coarse-grained soils with between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone on the plasticity chart have dual classifications.

PLASTICITY CHART



DEFINITION OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Boulders	Above 12 in.
Cobbles	12 in. to 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200 sieve
Coarse sand	No. 4 to No. 10 sieve
Medium sand	No. 10 to No. 40 sieve
Fine sand	No. 40 to No. 200 sieve
Fines (silt and clay)	Less than No. 200 sieve

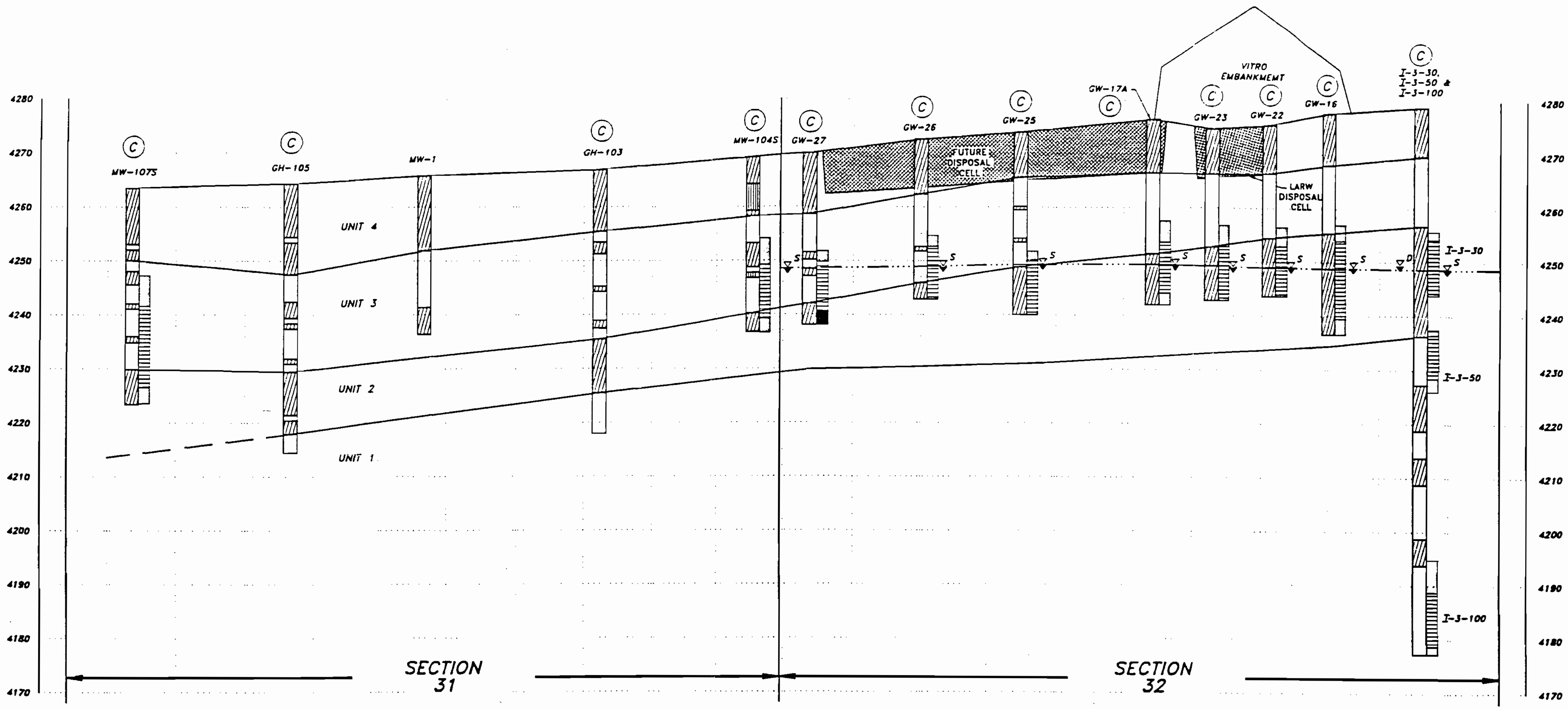
FIGURE B-7

Table 3.1 – Summary of Engineering Properties in Slope Stability Analysis (ref AMEC 12/13/05)

Material / Soil Units	Unit Weight, (pcf)	Angle of Internal Friction, (degrees)	Cohesion Intercept, (psf)	Basis / Reference Source
LARW Embankment Properties				
Rip Rap (Cover)	135	40	0	Appendix B-2, Table B
Clay Cover	123	0	1000	AMEC 1999a, Section 3.2.7
Protective Soil Layer (Debris Free Soil) – silty sand	117.5	38	250	AMEC 1999b[8], Figure A-7
Compressible Debris	101	18	130	AMEC 1999b, Figure 9
CLSM	120	0	15200* (equal to 100 psi)	Specification calls for minimum of 150 psi
Clay Liner	123	0 (28)	1000 (100)	Appendix B-2, Table B (AMEC 5/25/99, Figure A-6)
Embankment Foundation Properties				
		Drained / Undrained	Drained / Undrained	
Unit 4- Upper Clays	118	29 / 0	0 / 2000	CPT correlations Appendix B-1 (or AMEC 2005, App B-1) and AMEC 1999a
Unit 3 - Silty Sands	120	34	0	CPT correlations Appendix B-1 (or AMEC 2005, App B-1) and AMEC 1999a
Unit 2 - Clays and Silts	121	29 / 0	1000 / 2000	CPT correlations Appendix B-1 (or AMEC 2005, App B-1) and AMEC 1999a
Unit 1 - Interbedded Sand, Silt and Clay	120	29	0	CPT correlations Appendix B-1 (or AMEC 2005, App B-1) and AMEC 1999a

* This strength exceeds the strengths of the other materials by a large margin.

8 AMEC (formerly AGRA) (1999b), Task 2 -Summary of Field Strength Tests, Clive Disposal Facility, 75 Miles West of Salt Lake City, Clive, Utah, AGRA Job No. 8-817-002103, dated June 28, 1999.



LEGEND

- SILTY CLAY
- CLAYEY SILT
- SILTY SAND
- GRAVELLY SAND
- BEDROCK
  PALEOZOIC LIMESTONE, DOLOMITE & SHALE; LOCALLY INCLUDE SANDSTONE, QUARTZITE & EVAPORITES.
- SILICA SAND
- WELL SCREEN
- BENTONITE PLUG
- GROUNDWATER LEVEL MEASURED ON JANUARY 15, 1992 ADJUSTED TO FRESHWATER EQUIVALENT HEADS.
- CONTINUOUS SOIL SAMPLING HOLE

0 400 800 1200
Scale in Feet
HORIZONTAL SCALE

0 10 20 30
Scale in Feet
VERTICAL SCALE

Rev.	By	Date	Remarks
2	SLP	1/20/92	REVISE CROSS SECTION & ADD NEW WELLS
1	SLP	9/31/91	REVISE WELL SCREEN

ENVIROCARE OF UTAH

HYDROGEOLOGIC
CROSS SECTION B–B'

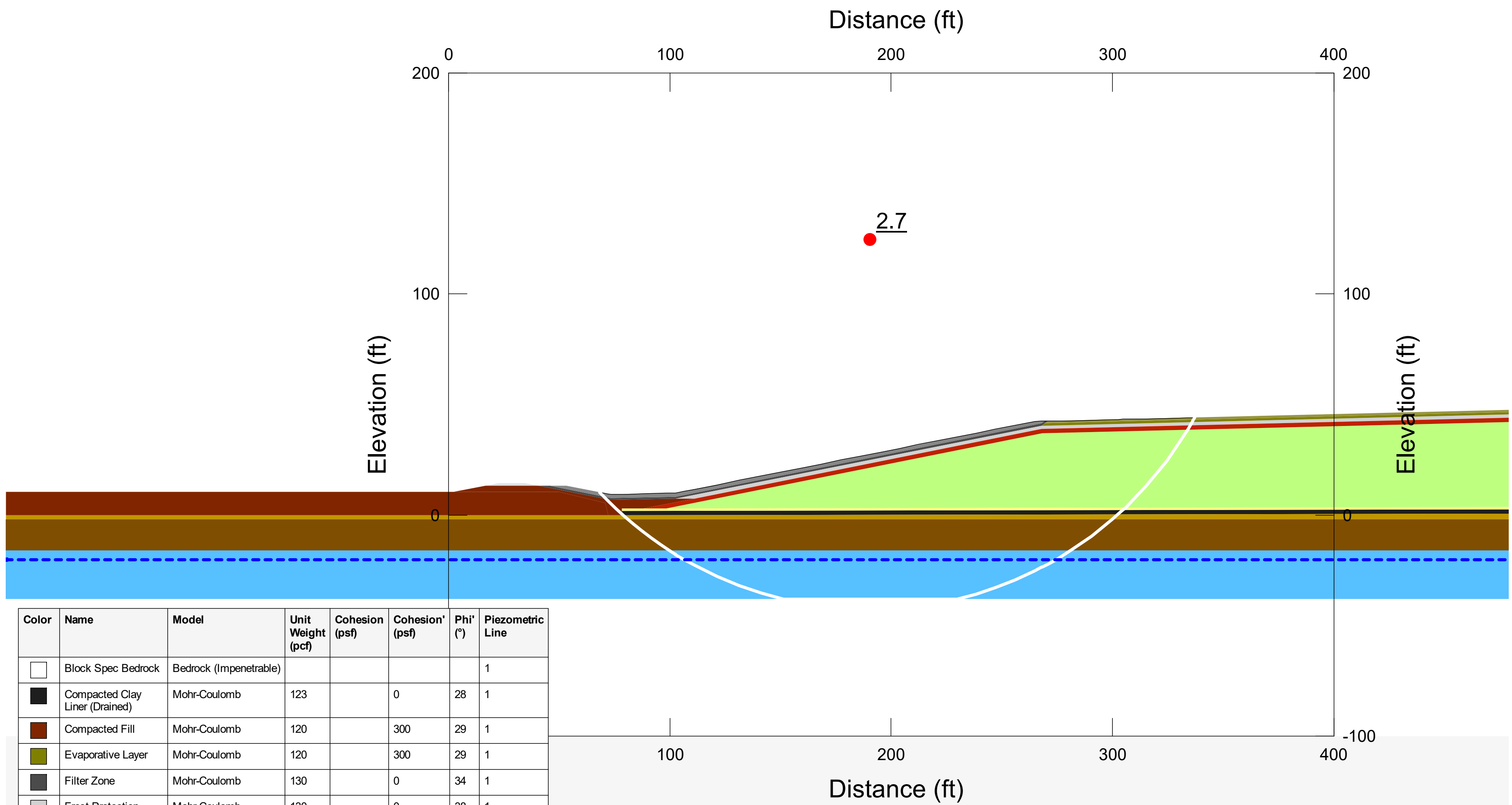
BINGHAM ENVIRONMENTAL
SALT LAKE CITY – (801) 532-2230

FIGURE 5

Date MARCH 1991 | Proj. # 1418-018 | T.M. | CVSECB-B

ATTACHMENT B

P:\PRJ\SDWP\Current Projects\SLC Federal Cell Clive Facility\Engineering Evaluations and Calcs\Slope\W\Federal Cell simplified to critical sections.gsz



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123		0	28	1
	Compacted Fill	Mohr-Coulomb	120		300	29	1
	Evaporative Layer	Mohr-Coulomb	120		300	29	1
	Filter Zone	Mohr-Coulomb	130		0	34	1
	Frost Protection	Mohr-Coulomb	130		0	38	1
	Liner Protective Cover	Mohr-Coulomb	118		250	38	1
	LLRW with CLSM	Mohr-Coulomb	120		0	30	1
	Radon Clay Cover	Mohr-Coulomb	123		1,000	0	1
	Roadbase	Mohr-Coulomb	130		0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135		0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120		200	30	1
	Unit 2 CL/ML (23-45) Undrained	Undrained (Phi=0)	121	1,500			1
	Unit 3 SM (9-23) Drained	Mohr-Coulomb	120		0	34	1
	Unit 4 CL/ML (0-9) Drained	Mohr-Coulomb	118		0	29	1

Energy Solutions Federal Cell
Short Term Undrained GW @ Current Conditions
Unit 2 Adjacent Road Short Term

03/26/2021

Project No. SLC1025

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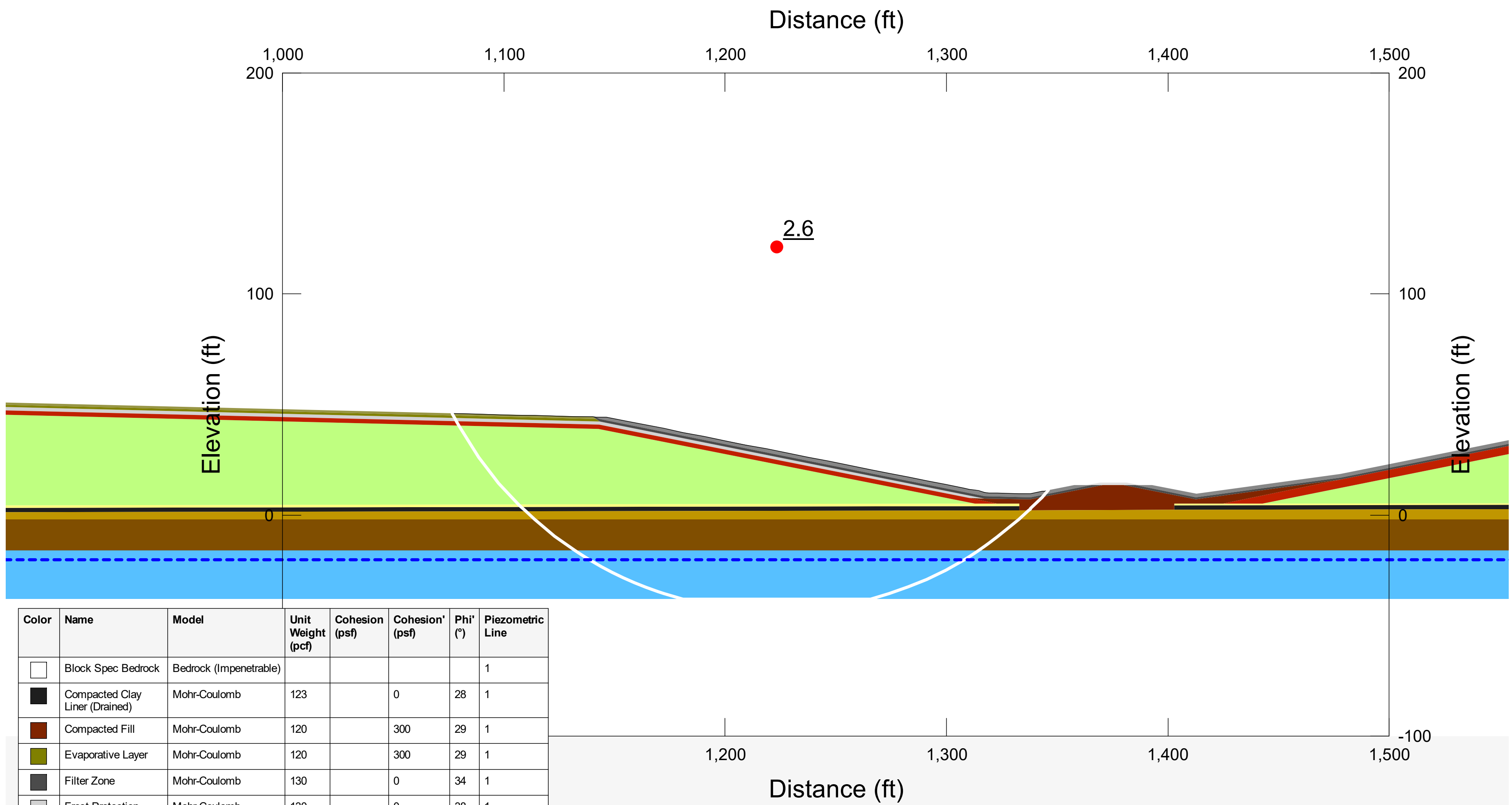
Figure B-1



Color	Name	Model	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123		0	28	1
	Compacted Fill	Mohr-Coulomb	120		300	29	1
	Evaporative Layer	Mohr-Coulomb	120		300	29	1
	Filter Zone	Mohr-Coulomb	130		0	34	1
	Frost Protection	Mohr-Coulomb	130		0	38	1
	Liner Protective Cover	Mohr-Coulomb	118		250	38	1
	LLRW with CLSM	Mohr-Coulomb	120		0	30	1
	Radon Clay Cover	Mohr-Coulomb	123		1,000	0	1
	Roadbase	Mohr-Coulomb	130		0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135		0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120		200	30	1
	Unit 2 CL/ML (23-45) Undrained	Undrained (Phi=0)	121	1,500			1
	Unit 3 SM (9-23) Drained	Mohr-Coulomb	120		0	34	1
	Unit 4 CL/ML (0-9) Drained	Mohr-Coulomb	118		0	29	1

Energy Solutions Federal Cell
Short Term Undrained GW @ Current Conditions
Unit 2 Adjacent 11e Short Term

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Figure B-2

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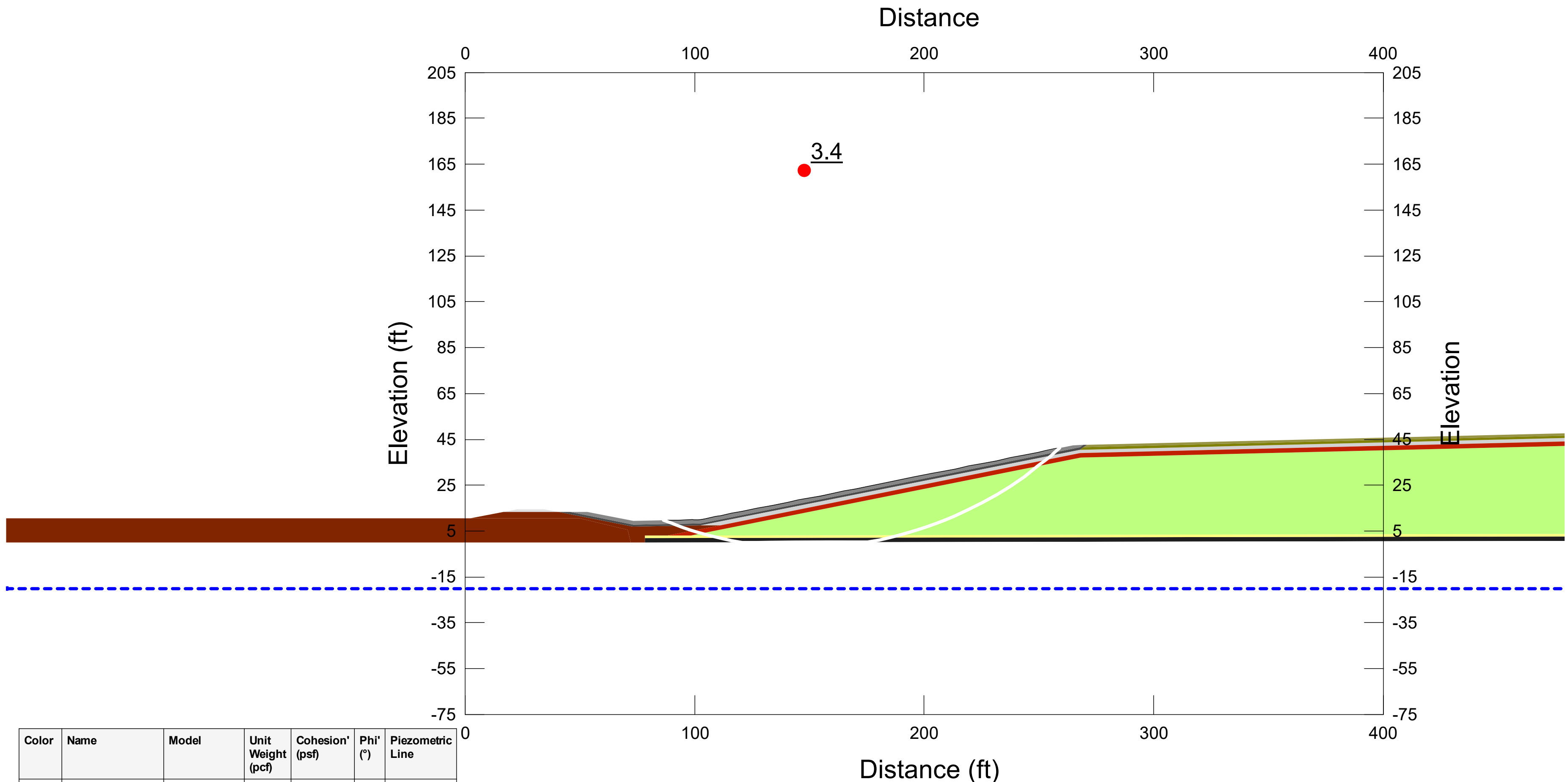

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1

Energy Solutions Federal Cell
Long Term Static Drained GW @ Current Conditions
Clay Liner Adjacent Road

03/26/2021

Project No. SLC1025

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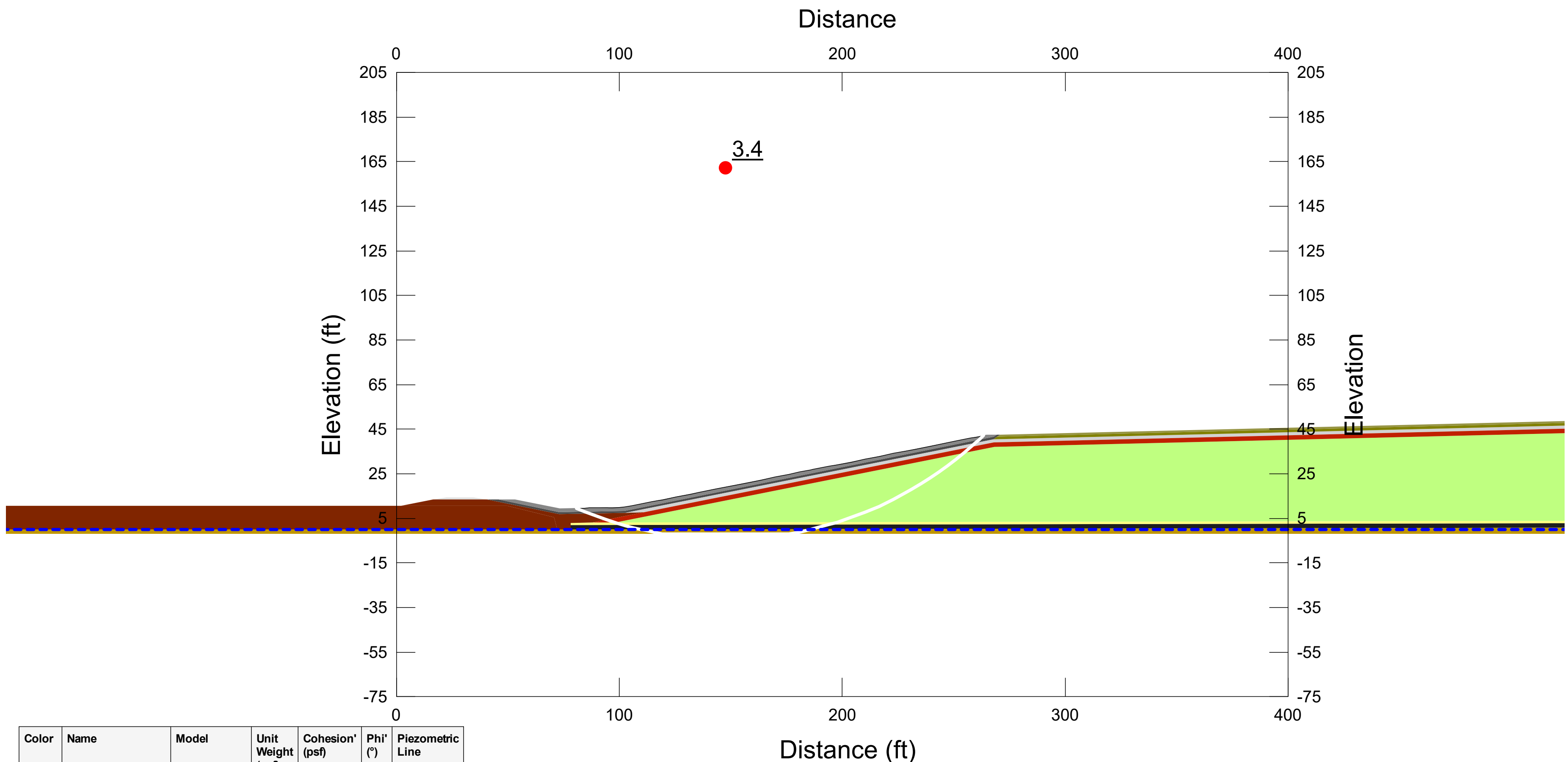
Figure B-3



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1
	Unit 4 CL/ML (0-9) Drained	Mohr-Coulomb	118	0	29	1

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Energy Solutions Federal Cell
Long Term Static Drained GW @ Rise Conditions
Unit 4 Adjacent Road Long Term Drained

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Figure B-4

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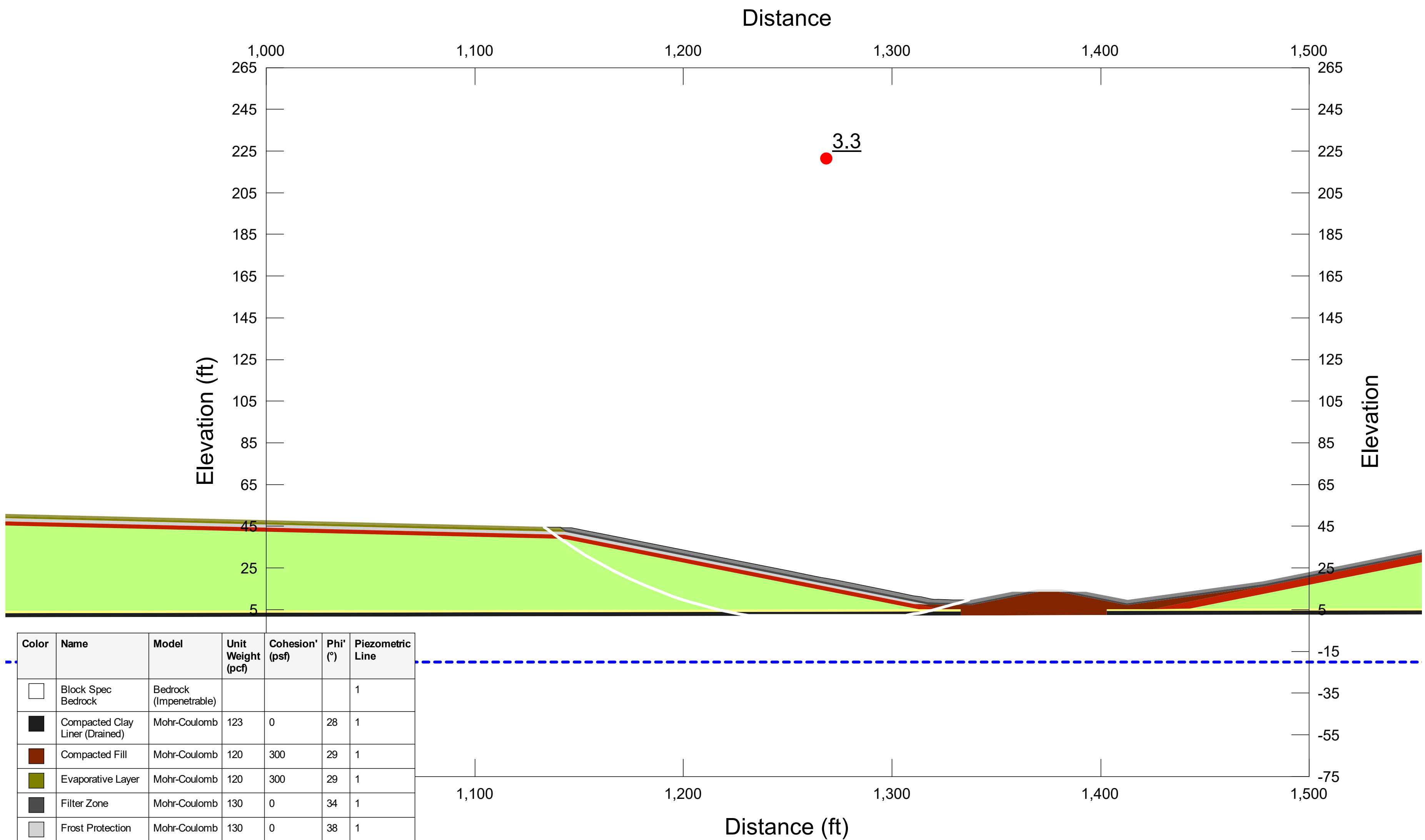

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1

Energy Solutions Federal Cell
Long Term Static Drained GW @ Current Conditions
Clay Liner Adjacent 11e

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Figure B-5

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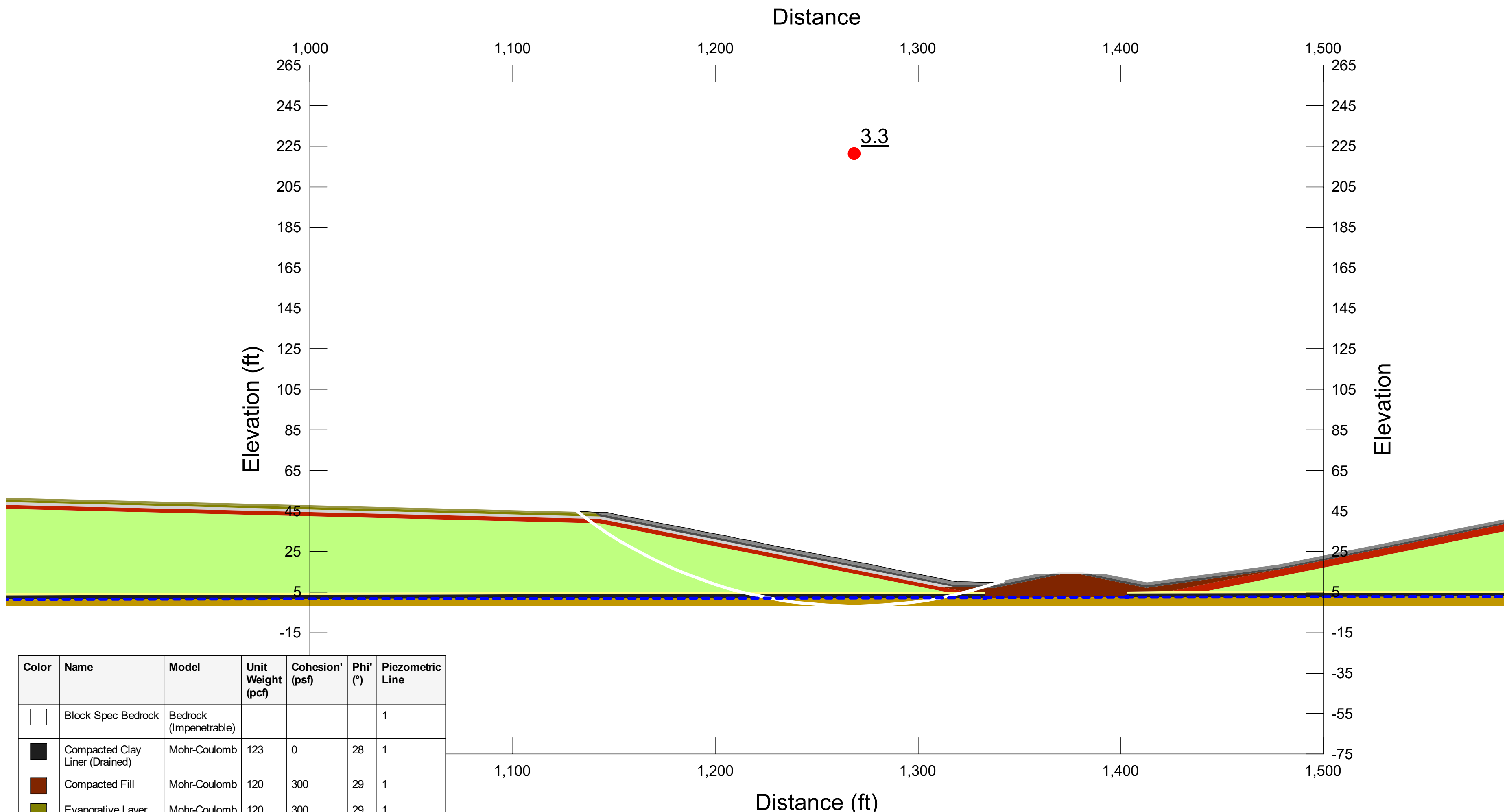

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1
	Unit 4 CL/ML (0-9) Drained	Mohr-Coulomb	118	0	29	1

Energy Solutions Federal Cell
Long Term Static Drained GW @ Rise Conditions
Unit 4 Adjacent 11e Long Term Drained

03/19/2021

Project No. SLC1025

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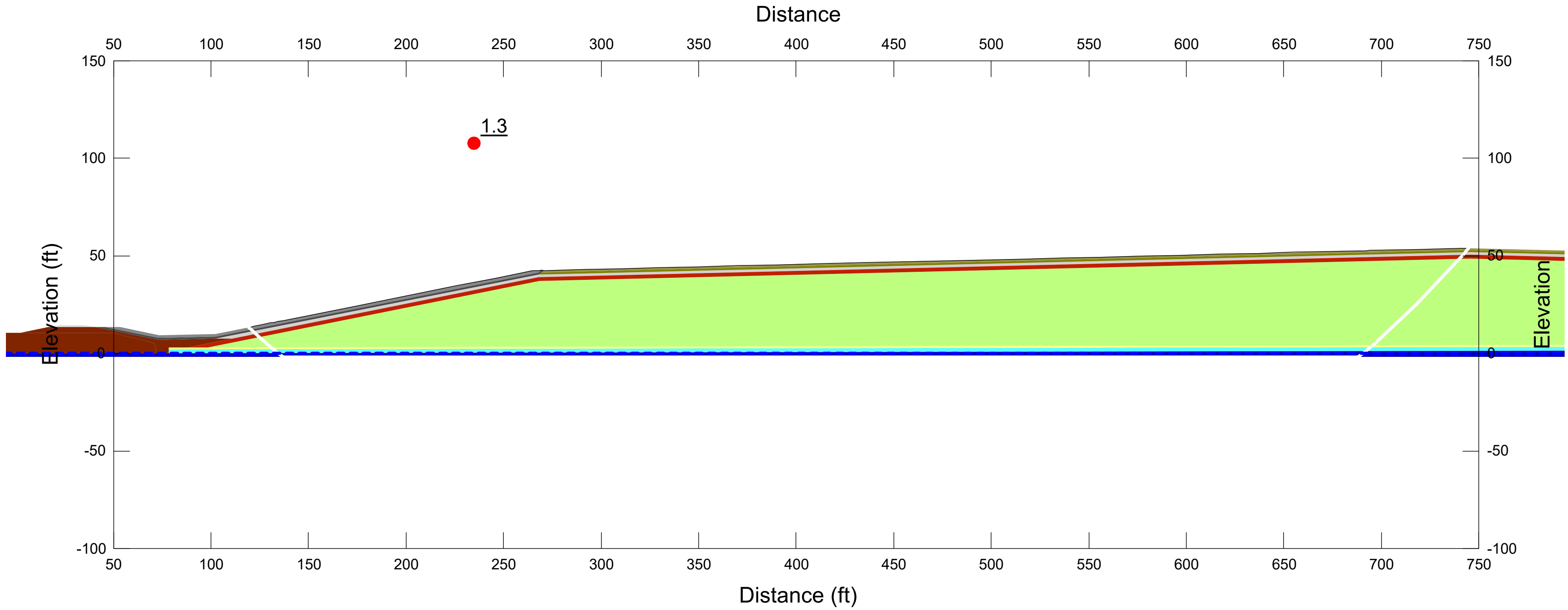
Figure
B-6



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
	Filter Zone	Mohr-Coulomb	130	0	34		1
	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 4 CL/ML (0-9) Undrained	Undrained (Phi=0)	118			1,000	1

Energy Solutions Federal Cell
Pseudostatic Undrained GW @ Rise Conditions
Unit 4 Adjacent Road Seismic

03/17/2021

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Figure B-7

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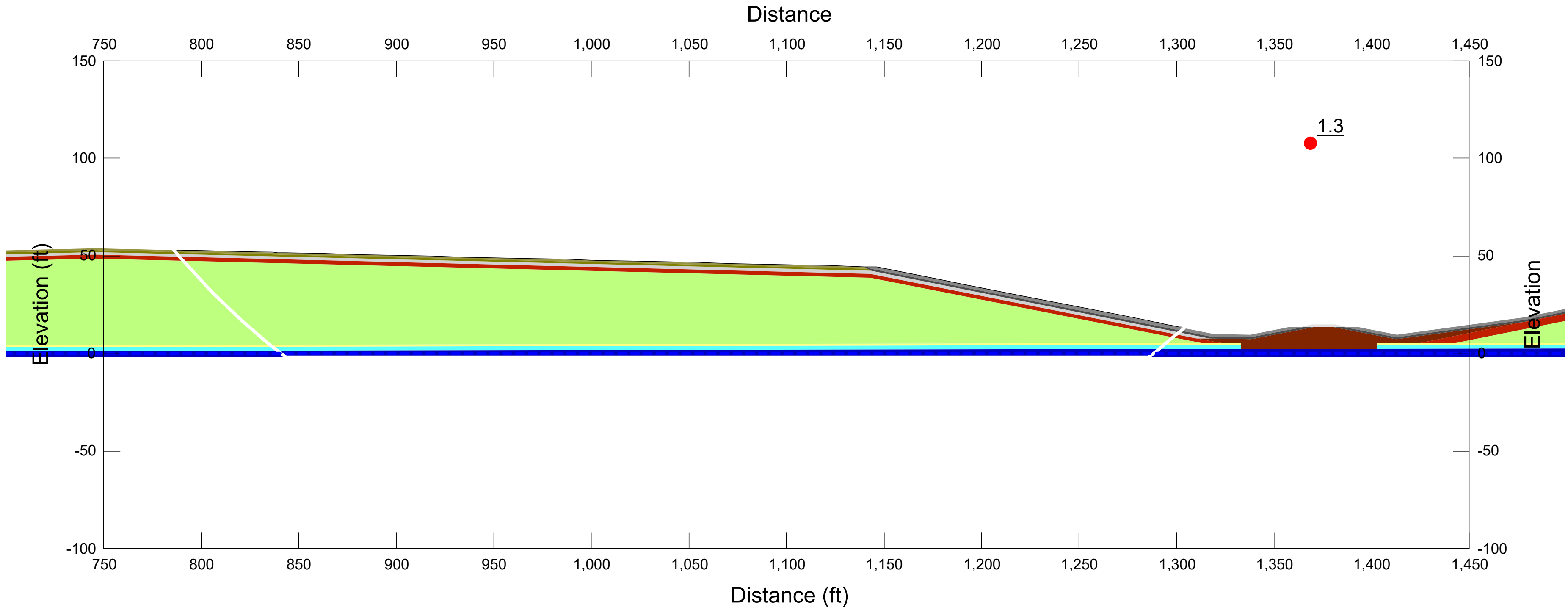

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
	Filter Zone	Mohr-Coulomb	130	0	34		1
	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 4 CL/ML (0-9) Undrained	Undrained (Phi=0)	118			1,000	1

Energy Solutions Federal Cell
Pseudostatic Undrained GW @ Rise Conditions
Unit 4 Adjacent 11e Seismic

03/17/2021

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Figure B-8

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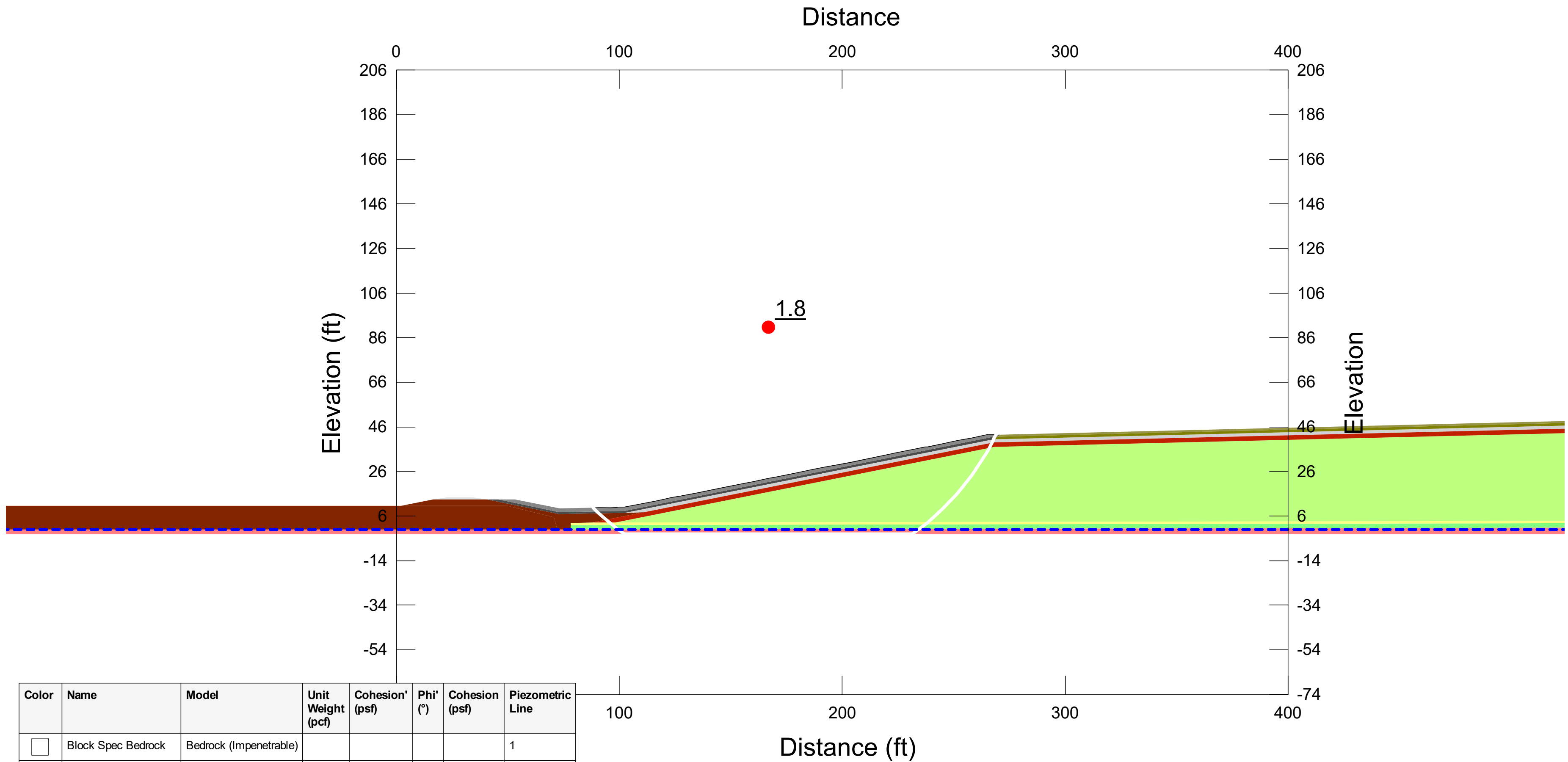

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Clay Liner Undrained Cyclic Softening	Undrained (Phi=0)	123			500	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
	Filter Zone	Mohr-Coulomb	130	0	34		1
	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 4 CL/ML (0-9) Undrained Cyclic Softening	Undrained (Phi=0)	118			500	1

Energy Solutions Federal Cell
Undrained Clay Like Soils GW @ Rise Conditions (Cyclic Softening)
Unit 4 Adjacent Road Softened

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Figure B-9

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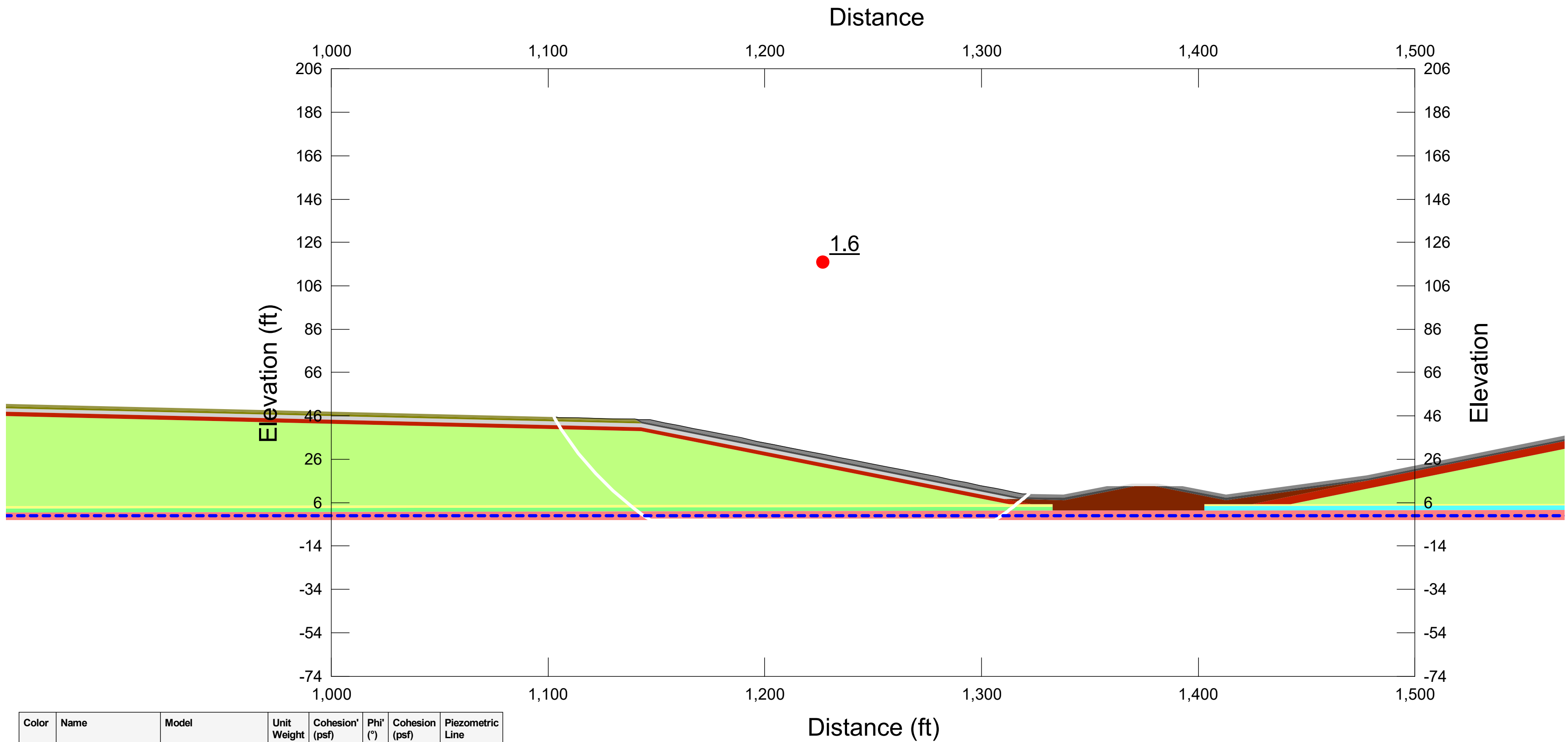

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Clay Liner Undrained Cyclic Softening	Undrained (Phi=0)	123			500	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
	Filter Zone	Mohr-Coulomb	130	0	34		1
	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 4 CL/ML (0-9) Undrained Cyclic Softening	Undrained (Phi=0)	118			500	1

Energy Solutions Federal Cell
Undrained Clay Like Soils GW @ Rise Conditions (Cyclic Softening)
Unit 4 Adjacent 11e Softened

03/17/2021

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Figure B-10

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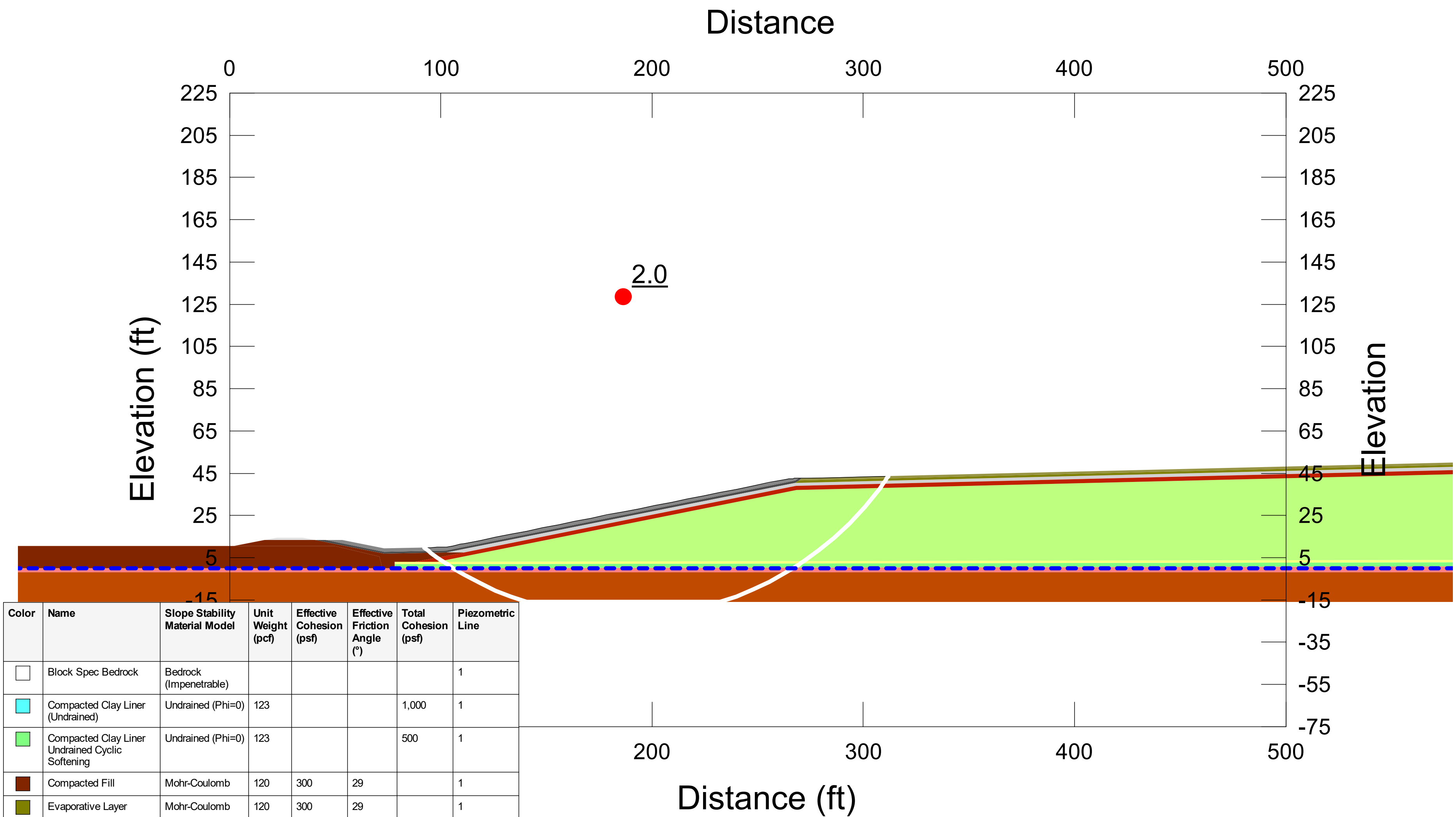

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Total Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Clay Liner Undrained Cyclic Softening	Undrained (Phi=0)	123			500	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
	Filter Zone	Mohr-Coulomb	130	0	34		1
	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 3 SM Liquefied Residual Strength	Undrained (Phi=0)	120			1,000	1
	Unit 4 CL/ML (0-9) Undrained Cyclic Softening	Undrained (Phi=0)	118			500	1

Energy Solutions Federal Cell
Post EQ Residual Strengths GW @ Rise Conditions
Unit 4 Unit 3 Adjacent Road

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Figure
B-11

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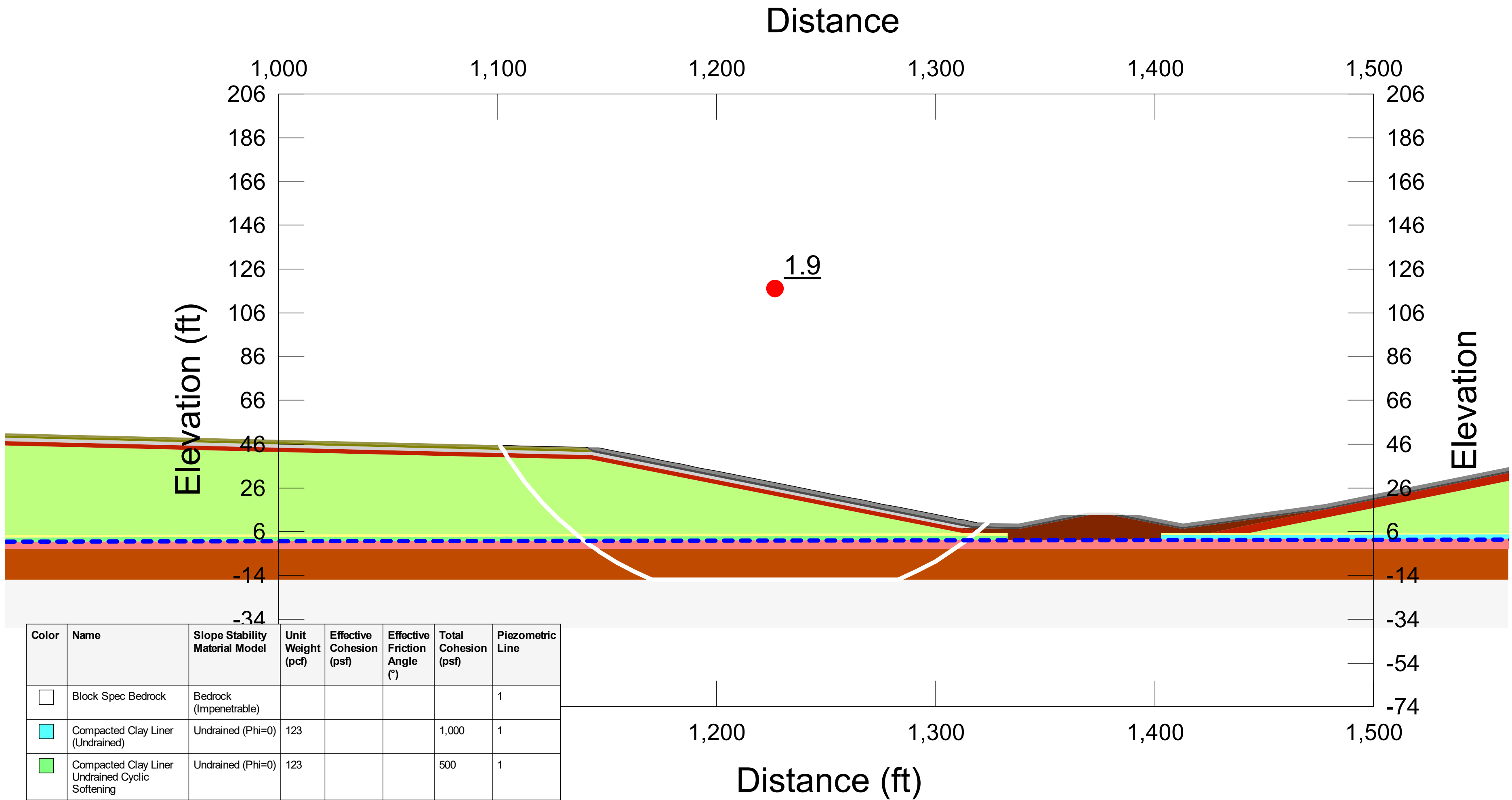

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Total Cohesion (psf)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Undrained)	Undrained (Phi=0)	123			1,000	1
	Compacted Clay Liner Undrained Cyclic Softening	Undrained (Phi=0)	123			500	1
	Compacted Fill	Mohr-Coulomb	120	300	29		1
	Evaporative Layer	Mohr-Coulomb	120	300	29		1
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	Frost Protection	Mohr-Coulomb	130	0	38		1
	Liner Protective Cover	Mohr-Coulomb	118	250	38		1
	LLRW with CLSM	Mohr-Coulomb	120	0	30		1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0		1
	Roadbase	Mohr-Coulomb	130	0	36		1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40		1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30		1
	Unit 3 SM Liquefied Residual Strength	Undrained (Phi=0)	120			1,000	1
	Unit 4 CL/ML (0-9) Undrained Cyclic Softening	Undrained (Phi=0)	118			500	1

Energy Solutions Federal Cell
Post EQ Residual Strengths GW @ Rise Conditions
Unit 4 Unit 3 Adjacent 11e

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Figure B-12

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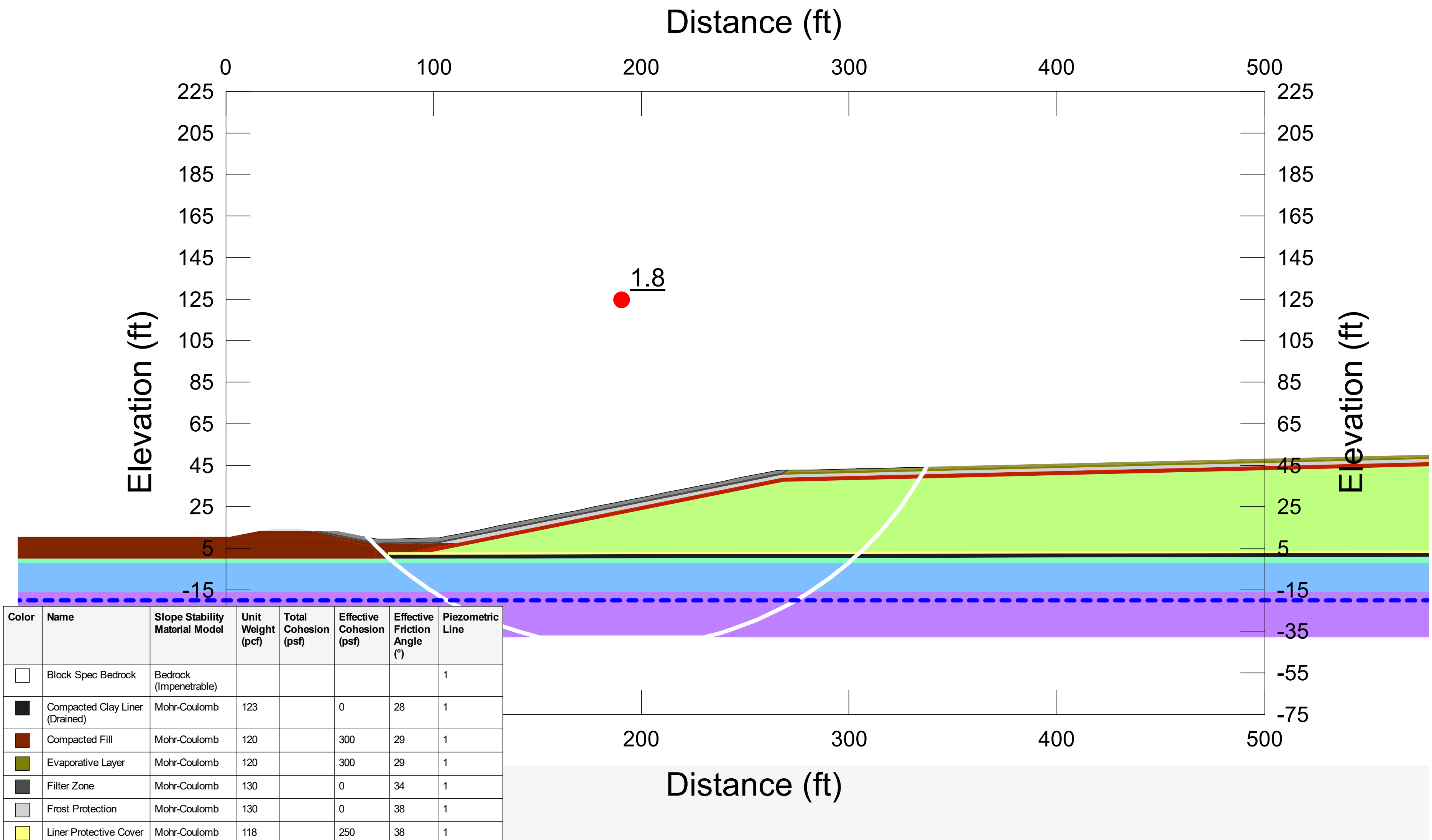
ATTACHMENT B2



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123		0	28	1
	Compacted Fill	Mohr-Coulomb	120		300	29	1
	Evaporative Layer	Mohr-Coulomb	120		300	29	1
	Filter Zone	Mohr-Coulomb	130		0	34	1
	Frost Protection	Mohr-Coulomb	130		0	38	1
	Liner Protective Cover	Mohr-Coulomb	118		250	38	1
	LLRW with CLSM	Mohr-Coulomb	120		0	30	1
	Radon Clay Cover	Mohr-Coulomb	123		1,000	0	1
	Roadbase	Mohr-Coulomb	130		0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135		0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120		200	30	1
	Unit 2 CL/ML (23-45) Undrained - S	Undrained (Phi=0)	121	750			1
	Unit 3 SM (9-23) Drained - S	Mohr-Coulomb	120		0	31	1
	Unit 4 CL/ML (0-9) Undrained - S	Undrained (Phi=0)	118	500			1

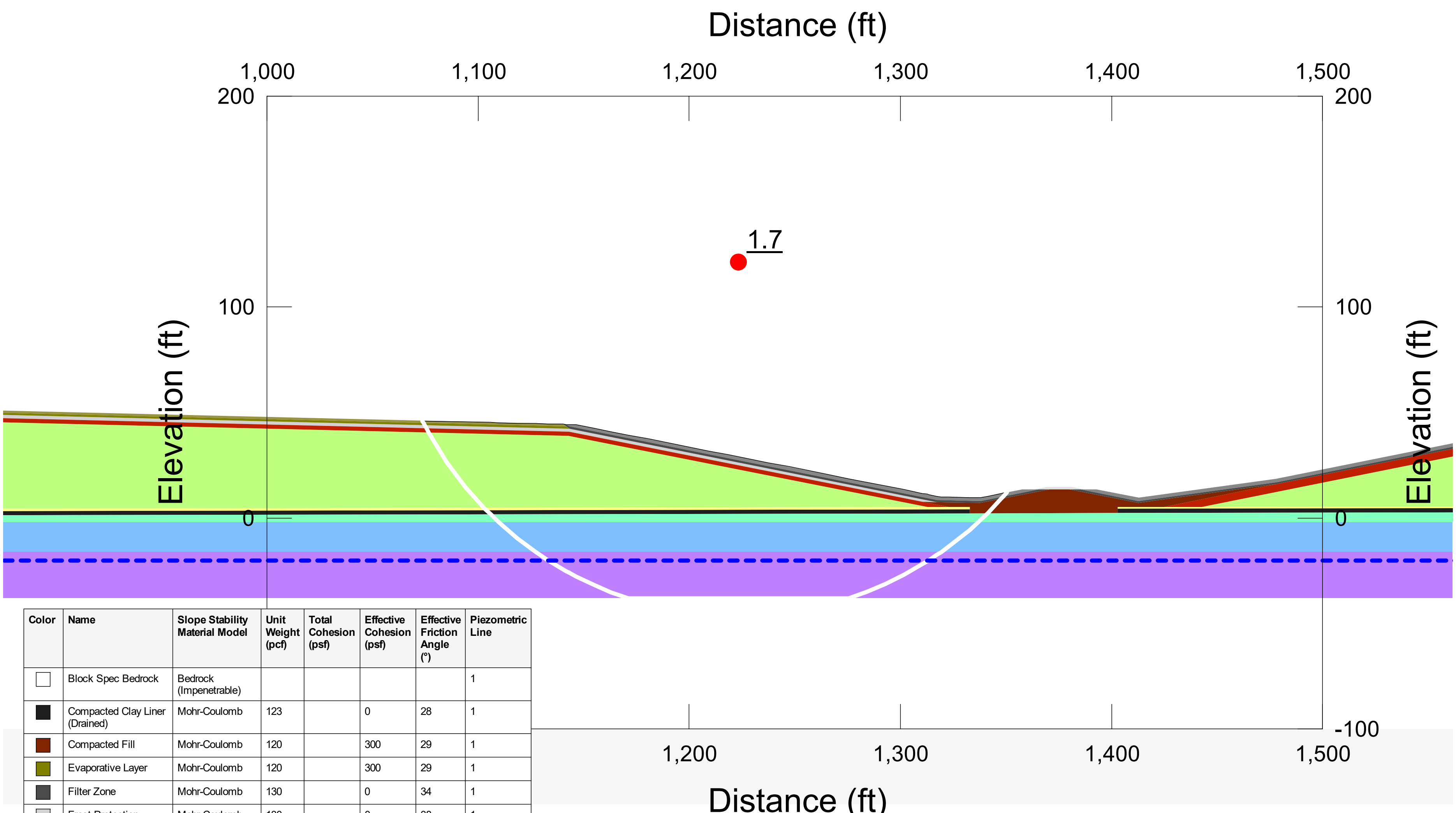
Energy Solutions Federal Cell
Short Term Undrained GW @ Current Conditions Sensitivity
Unit 2 Adjacent Road Short Term - S

Geosyntec consultants

Figure B2-1

01/25/2023 | Project No. SLC1025

\\SanDiego-01\data\Current Projects\SLC Federal Cell Clive Facility\Engineering Evaluations and Calcs\SlopeW Sensitivity\Federal Cell Road adjacent -sens.gsz



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)					1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123		0	28	1
	Compacted Fill	Mohr-Coulomb	120		300	29	1
	Evaporative Layer	Mohr-Coulomb	120		300	29	1
	Filter Zone	Mohr-Coulomb	130		0	34	1
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	Liner Protective Cover	Mohr-Coulomb	118		250	38	1
	LLRW with CLSM	Mohr-Coulomb	120		0	30	1
	Radon Clay Cover	Mohr-Coulomb	123		1,000	0	1
	Roadbase	Mohr-Coulomb	130		0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135		0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120		200	30	1
	Unit 2 CL/ML (23-45) Undrained - S	Undrained (Phi=0)	121	750			1
	Unit 3 SM (9-23) Drained - S	Mohr-Coulomb	120		0	31	1
	Unit 4 CL/ML (0-9) Undrained - S	Undrained (Phi=0)	118	500			1

Energy Solutions Federal Cell
Short Term Undrained GW @ Current Conditions Sensitivity
Unit 2 Adjacent 11e Short Term - S

01/25/2023

Project No. SLC1025

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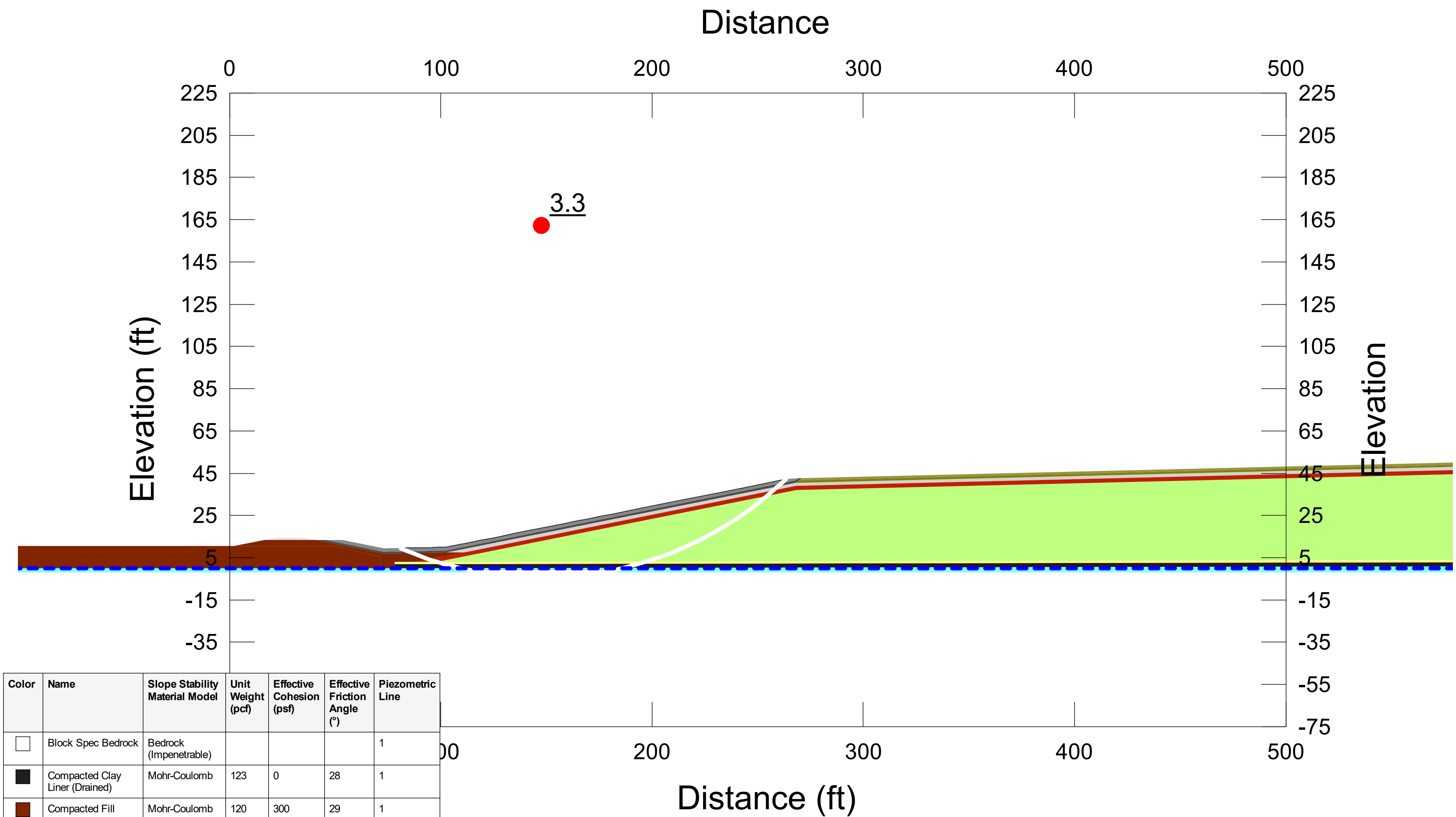
Figure B2-2

\\SanDiego-01\data\Current Projects\SLC Federal Cell Clive Facility\Engineering Evaluations and Calcs\SlopeW Sensitivity\Federal Cell 11e adjacent - sens.gsz



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1
	Unit 4 CL/ML (0-9) Drained - S	Mohr-Coulomb	118	0	27	1

Energy Solutions Federal Cell
Long Term Static Drained GW @ Rise Conditions Sensitivity
Unit 4 Adjacent Road Long Term Drained - S

01/25/2023

Project No. SLC1025

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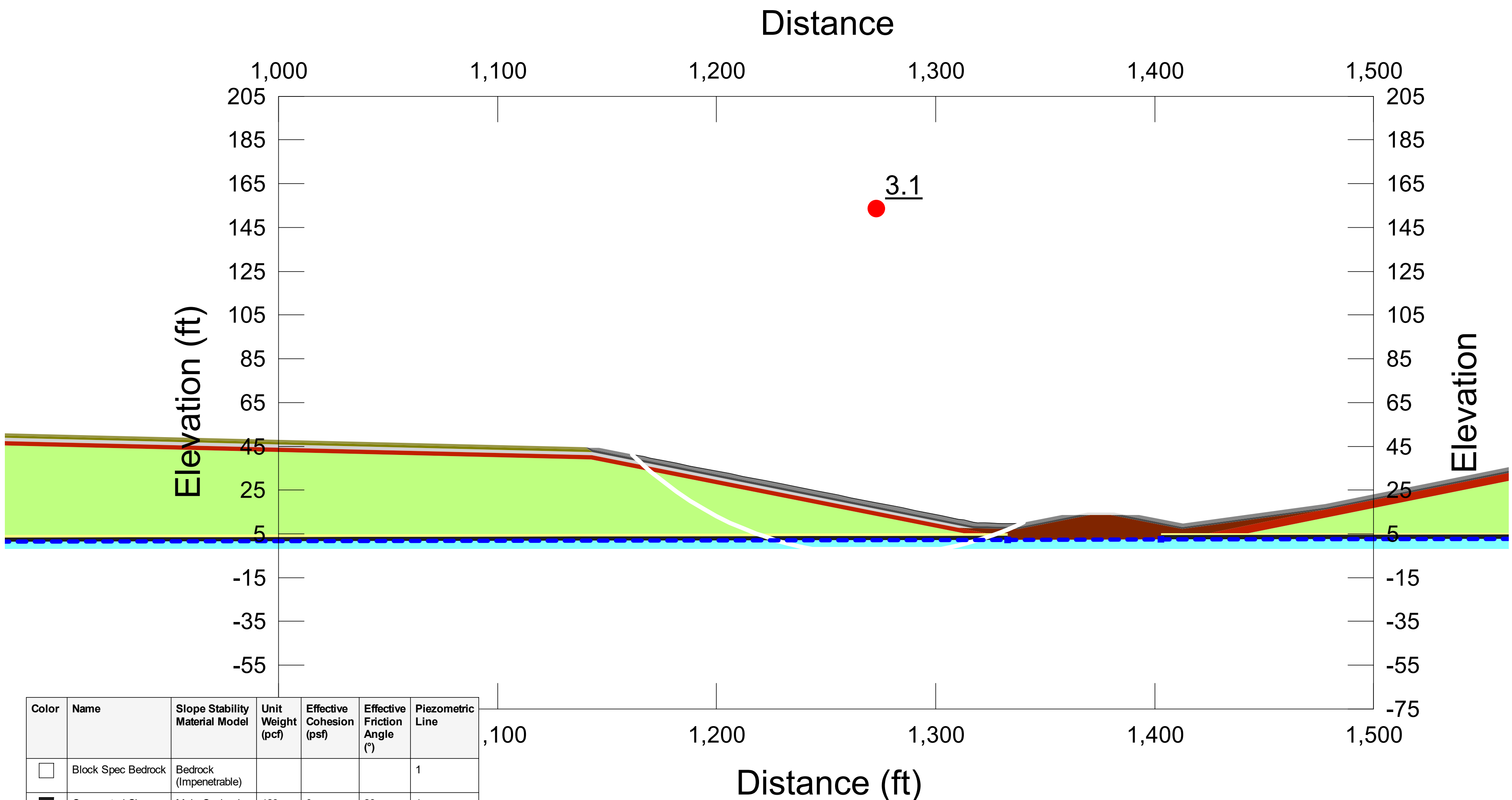
Figure B2-3



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Block Spec Bedrock	Bedrock (Impenetrable)				1
	Compacted Clay Liner (Drained)	Mohr-Coulomb	123	0	28	1
	Compacted Fill	Mohr-Coulomb	120	300	29	1
	Evaporative Layer	Mohr-Coulomb	120	300	29	1
	Filter Zone	Mohr-Coulomb	130	0	34	1
	Frost Protection	Mohr-Coulomb	130	0	38	1
	Liner Protective Cover	Mohr-Coulomb	118	250	38	1
	LLRW with CLSM	Mohr-Coulomb	120	0	30	1
	Radon Clay Cover	Mohr-Coulomb	123	1,000	0	1
	Roadbase	Mohr-Coulomb	130	0	36	1
	Side Rock (Rip Rap)	Mohr-Coulomb	135	0	40	1
	Top Slope Surface Layer	Mohr-Coulomb	120	200	30	1
	Unit 4 CL/ML (0-9) Drained - S	Mohr-Coulomb	118	0	27	1

Energy Solutions Federal Cell
Long Term Static Drained GW @ Rise Conditions Sensitivity
Unit 4 Adjacent 11e Long Term Drained - S

01/25/2023 | Project No. SLC1025

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Figure B2-4

\\SanDiego-01\data\Current Projects\SLC Federal Cell Clive Facility\Engineering Evaluations and Calcs\SlopeW Sensitivity\Federal Cell 11e adjacent - sens.gsz

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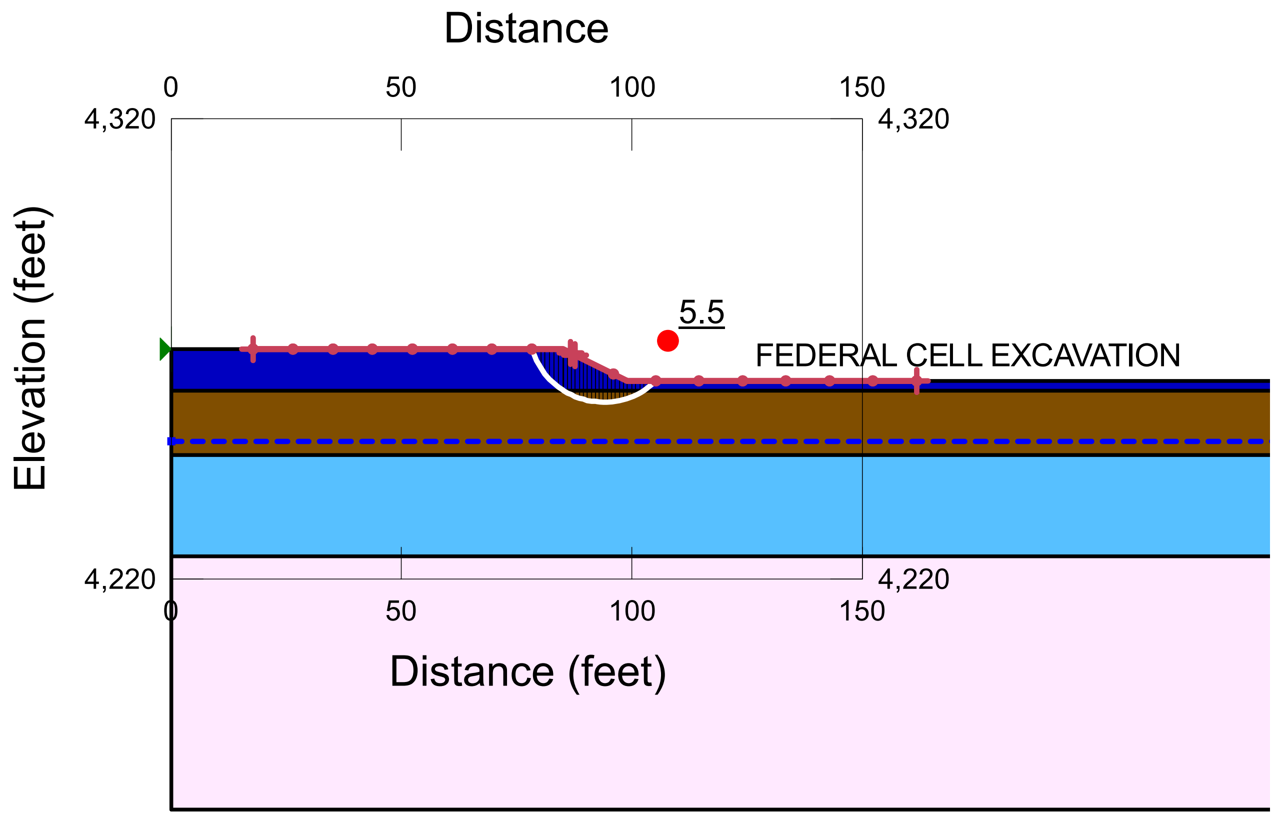
ATTACHMENT B3



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Total Cohesion (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Unit 1 SM/SC with ML/CL Lens (45-100) Drained	Mohr-Coulomb	120		0	29	1
	Unit 2 CL/ML (23-45) Undrained	Undrained (Phi=0)	121	1,500			1
	Unit 3 SM (9-23) Drained	Mohr-Coulomb	120		0	34	1
	Unit 4 CL/ML (0-9) Undrained	Undrained (Phi=0)	118	1,000			1

Energy Solutions
Interim Short Term
Interim Slope Stability

01/20/2023

Project No. SLC1025

Geosyntec consultants

Figure B3-1

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ATTACHMENT C

SLC1025
Earthquake Deformation Analysis
Makdisi & Seed Simplified Method

		Case/Description	k_y	$\ddot{u}_{max}$	y (ft)	H (ft)	y/H	$k_{max}/\ddot{u}_{max}$	k_{max}	k_y/k_{max}	Deformation (cm)	Deformation (mm)	Allowable Deformation (mm)
FS	1	Critical Section Failure Through Unit 4, entire slope face (y/h =1), adjacent 11(e)	0.180	0.580	52.0	52.0	1.0	0.34	0.20	0.91	0.4	4	150-300
Mw:	7.3												
PHGA (g):	0.24												

As a comparison to the above calculated values, the seismic design criteria and performance standards for closure of the Operating Industries, Inc. (OII) landfill provides an example of criteria based on allowable deformations. The OII landfill criteria allow 150 mm (6-inches) of soil deformation within its cover system (Kavazanjian et al. 1998). Other published records of performance criteria for municipal landfills generally indicate an allowable deformation in the cover system to be on the order of 150 to 300 mm (6 to 12-inches, Kavazanjian 1998). In current U.S. practice, the 150 to 300 mm of displacement is commonly accepted as the allowable calculated seismic displacement for a geosynthetic cover system (Seed and Bonaparte, 1992.

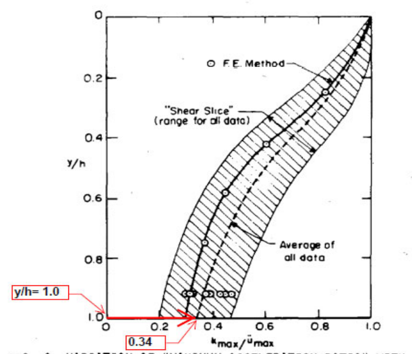
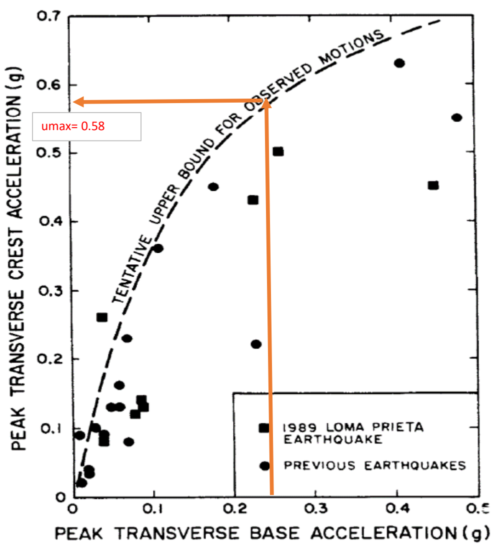
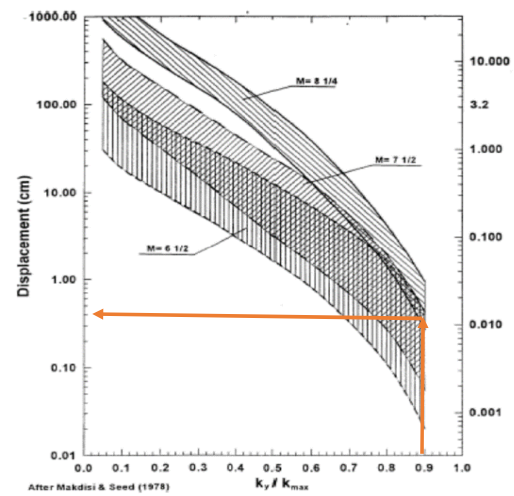


FIG. 9 VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

Reference: Makdisi and Seed [1978], "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformation," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT7, pp 849-867.





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ATTACHMENT D

SETTLEMENT ANALYSES

Site:	CLIVE FEDERAL CELL	Project No.:	SLC1025
Location:	CLIVE UTAH		
Client:	ES	Date:	17-Mar-21
Prepared by:	M.Downing	Reviewed by:	B.Baturay

Theory

Total settlement made up of three (3) components:

Total Settlement s_t = Immediate Settlement (s_i) + Primary Consolidation (s_c) + Secondary Settlement (s_s)

Primary Consolidation s_c

$$S = C_r\, H_o/(1+eo)\, \log[\sigma'c/\sigma'_{vo}] + C_c\, H_o/1+eo\, \log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'c]$$

where C_r =recompression index
C_c = compression index
H_o = initial soil layer thickness
σ'c = effective preconsolidation pressure = OCR σ'_{vo}
σ'_{vo} = initial effective vertical stress
$\Delta\sigma_v$ = change in vertical effective stress
eo=initial void ratio

Secondary Settlement s_s

$$s_s = C_{\alpha\varepsilon}\, H_{100}\, \log(t_2/t_1)$$

where $C_{\alpha\varepsilon}$ = secondary compression index
H_o = thickness of compressible layer at end of primary consolidation
t2 = time for which secondary settlements are calculated (500 years for design life, assume settlement after that is minimul due to log scale projection of creep)
$t_1 = t_{100}$ for primary consolidation - 1 year - estimated by previous analyses of Unit 2 and 4 clay layers (AMEC)

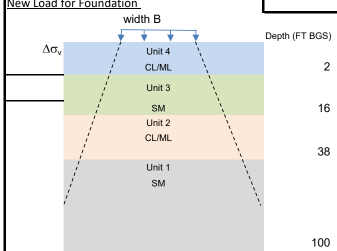
Elastic (Immediate)

Ze=Δσ/Ms *Ho

wher Z =elastic settlement of soil layer
Ho= initial thickness of soil layer
Δσ= change in stress in layer
Ms = constrained modulus of soil estimated with E and v of the insitu soil

CALCULATIONS

New Load for Foundation



Height of Waste and Cover Materials=	52.5 ft	at the tallest point, including cover
Average Unit Weight of Cover and Waste=	120.0 pcf	
$\Delta\sigma_v$ from Loading =	6300.0 psf	
B =	1225.0 ft	Based on Cell Limits
L =	1920.0 ft	
Unit 4 Unit Weight	118.0 pcf	
Unit 3 Unit Weight	120.0 pcf	
Unit 2 Unit Weight	121.0 pcf	
Unit 1 Unit Weight	120.0 pcf	
Unit weight of water	62.4 pcf	
Depth to Water =	18.0 ft	gw @ 25' below current grade, approximately 7 feet of upper material to be removed = 16 feet bgs for modeling

Unit 4 C_c =	0.250	Unit 4 eo =	1.1	Unit 3 Ms =	311,040
Unit 4 Cr =	0.02	Unit 2 eo =	1.2	Unit 1 Ms=	531556
Unit 4 $C_{\alpha\varepsilon}$ =	0.004	Unit 4 OCR =	5	t_1 (t_{100} for primary consolidation)	1
		Unit 2 OCR =	1.5	t2 (compliance period of 10,000 years f)	10000
Unit 2 Cc=	0.2				
Unit 2 Cr =	0.025				
Unit 2 Cαε =	0.00450				

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo}+\Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo}+\Delta\sigma_v$ < σ'c	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
0.0						6300.0										
1.0	0.5	59.0		59.0	2353572.8	6295.8	6354.8	5.0	295.0	1.0	no	0.160	0.840	0.013	0.173	
2.0	1.5	177.0		177.0	2356719.8	6287.4	6464.4	5.0	885.0	1.0	no	0.104	0.896	0.014	0.118	
3.0	2.5	297.0		297.0	2359868.8	6279.0	6576.0			1.0						0.020
4.0	3.5	417.0		417.0	2363019.8	6270.6	6687.6			1.0						0.020
5.0	4.5	537.0		537.0	2366172.8	6262.3	6799.3			1.0						0.020
6.0	5.5	657.0		657.0	2369327.8	6253.9	6910.9			1.0						0.020
7.0	6.5	777.0		777.0	2372484.8	6245.6	7022.6			1.0						0.020
8.0	7.5	897.0		897.0	2375643.8	6237.3	7134.3			1.0						0.020
9.0	8.5	1017.0		1017.0	2378804.8	6229.0	7246.0			1.0						0.020
10.0	9.5	1137.0		1137.0	2381967.8	6220.7	7357.7			1.0						0.020
11.0	10.5	1257.0		1257.0	2385132.8	6212.5	7469.5			1.0						0.020
12.0	11.5	1377.0		1377.0	2388299.8	6204.2	7581.2			1.0						0.020
13.0	12.5	1497.0		1497.0	2391468.8	6196.0	7693.0			1.0						0.020
14.0	13.5	1617.0		1617.0	2394639.8	6187.8	7804.8			1.0						0.020
15.0	14.5	1737.0		1737.0	2397812.8	6179.6	7916.6			1.0						0.020
16.0	15.5	1857.0		1857.0	2400987.8	6171.5	8028.5			1.0						0.020
17.0	16.5	1978.0		1978.0	2404164.8	6163.3	8141.3	1.5	2967.0	1.0	no	0.042	0.958	0.017	0.059	
18.0	17.5	2099.0		2099.0	2407343.8	6155.2	8254.2	1.5	3148.5	1.0	no	0.040	0.960	0.017	0.057	
19.0	18.5	2220.0	31.2	2188.8	2410524.8	6147.0	8335.8	1.5	3283.2	1.0	no	0.039	0.961	0.017	0.056	
20.0	19.5	2341.0	93.6	2247.4	2413707.8	6138.9	8386.3	1.5	3371.1	1.0	no	0.038	0.962	0.017	0.055	
21.0	20.5	2462.0	156.0	2306.0	2416892.8	6130.8	8436.8	1.5	3459.0	1.0	no	0.037	0.963	0.017	0.055	
22.0	21.5	2583.0	218.4	2364.6	2420079.8	6122.8	8487.4	1.5	3546.9	1.0	no	0.036	0.964	0.017	0.054	
23.0	22.5	2704.0	280.8	2423.2	2423268.8	6114.7	8537.9	1.5	3634.8	1.0	no	0.036	0.964	0.017	0.053	
24.0	23.5	2825.0	343.2	2481.8	2426459.8	6106.7	8588.5	1.5	3722.7	1.0	no	0.035	0.965	0.017	0.052	
25.0	24.5	2946.0	405.6	2540.4	2429652.8	6098.6	8639.0	1.5	3810.6	1.0	no	0.034	0.966	0.017	0.052	
26.0	25.5	3067.0	468.0	2599.0	2432847.8	6090.6	8689.6	1.5	3898.5	1.0	no	0.034	0.966	0.017	0.051	
27.0	26.5	3188.0	530.4	2657.6	2436044.8	6082.6	8740.2	1.5	3986.4	1.0	no	0.033	0.967	0.017	0.050	
28.0	27.5	3309.0	592.8	2716.2	2439243.8	6074.7	8790.9	1.5	4074.3	1.0	no	0.032	0.968	0.017	0.050	
29.0	28.5	3430.0	655.2	2774.8	2442444.8	6066.7	8841.5	1.5	4162.2	1.0	no	0.032	0.968	0.017	0.049	
30.0	29.5	3551.0	717.6	2833.4	2445647.8	6058.8	8892.2	1.5	4250.1	1.0	no	0.031	0.969	0.017	0.049	
31.0	30.5	3672.0	780.0	2892.0	2448852.8	6050.8	8942.8	1.5	4338.0	1.0	no	0.031	0.969	0.017	0.048	
32.0	31.5	3793.0	842.4	2950.6	2452059.8	6042.9	8993.5	1.5	4425.9	1.0	no	0.030	0.970	0.017	0.047	
33.0	32.5	3914.0	904.8	3009.2	2455268.8	6035.0	9044.2	1.5	4513.8	1.0	no	0.029	0.971	0.017	0.047	
34.0	33.5	4035.0	967.2	3067.8	2458479.8	6027.1	9094.9	1.5	4601.7	1.0	no	0.029	0.971	0.017	0.046	
35.0	34.5	4156.0	1029.6	3126.4	2461692.8	6019.3	9145.7	1.5	4689.6	1.0	no	0.028	0.972	0.017	0.046	
36.0	35.5	4277.0	1092.0	3185.0	2464907.8	6011.4	9196.4	1.5	4777.5	1.0	no	0.028	0.972	0.017	0.045	
37.0	36.5	4398.0	1154.4	3243.6	2468124.8	6003.6	9247.2	1.5	4865.4	1.0	no	0.027	0.973	0.018	0.045	
38.0	37.5	4519.0	1216.8	3302.2	2471343.8	5995.8	9298.0	1.5	4953.3	1.0	no	0.027	0.973	0.018	0.044	
39.0	38.5	4639.0	1279.2	3359.8	2474564.8	5988.0	9347.8			1.0						0.011
40.0	39.5	4759.0	1341.6	3417.4	2477787.8	5980.2	9397.6			1.0						0.011
41.0	40.5	4879.0	1404.0	3475.0	2481012.8	5972.4	9447.4			1.0						0.011
42.0	41.5	4999.0	1466.4	3532.6	2484239.8	5964.6	9497.2			1.0						0.011
43.0	42.5	5119.0	1528.8	3590.2	2487468.8	5956.9	9547.1			1.0						0.011
44.0	43.5	5239.0	1591.2	3647.8	2490699.8	5949.2	9597.0			1.0						0.011
45.0	44.5	5359.0	1653.6	3705.4	2493932.8	5941.5	9646.9			1.0						0.011
46.0	45.5	5479.0	1716.0	3763.0	2497167.8	5933.8	9696.8			1.0						0.011
47.0	46.5	5599.0	1778.4	3820.6	2500404.8	5926.1	9746.7			1.0						0.011
48.0	47.5	5719.0	1840.8	3878.2	2503643.8	5918.4	9796.6			1.0						0.011

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo} + \Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo} + \Delta\sigma_v < \sigma'c$	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
49.0	48.5	5839.0	1903.2	3935.8	2506884.8	5910.8	9846.6			1.0						0.011
50.0	49.5	5959.0	1965.6	3993.4	2510127.8	5903.1	9896.5			1.0						0.011
51.0	50.5	6079.0	2028.0	4051.0	2513372.8	5895.5	9946.5			1.0						0.011
52.0	51.5	6199.0	2090.4	4108.6	2516619.8	5887.9	9996.5			1.0						0.011
53.0	52.5	6319.0	2152.8	4166.2	2519868.8	5880.3	10046.5			1.0						0.011
54.0	53.5	6439.0	2215.2	4223.8	2523119.8	5872.7	10096.5			1.0						0.011
55.0	54.5	6559.0	2277.6	4281.4	2526372.8	5865.2	10146.6			1.0						0.011
56.0	55.5	6679.0	2340.0	4339.0	2529627.8	5857.6	10196.6			1.0						0.011
57.0	56.5	6799.0	2402.4	4396.6	2532884.8	5850.1	10246.7			1.0						0.011
58.0	57.5	6919.0	2464.8	4454.2	2536143.8	5842.6	10296.8			1.0						0.011
59.0	58.5	7039.0	2527.2	4511.8	2539404.8	5835.1	10346.9			1.0						0.011
60.0	59.5	7159.0	2589.6	4569.4	2542667.8	5827.6	10397.0			1.0						0.011
61.0	60.5	7279.0	2652.0	4627.0	2545932.8	5820.1	10447.1			1.0						0.011
62.0	61.5	7399.0	2714.4	4684.6	2549199.8	5812.6	10497.2			1.0						0.011
63.0	62.5	7519.0	2776.8	4742.2	2552468.8	5805.2	10547.4			1.0						0.011
64.0	63.5	7639.0	2839.2	4799.8	2555739.8	5797.8	10597.6			1.0						0.011
65.0	64.5	7759.0	2901.6	4857.4	2559012.8	5790.4	10647.8			1.0						0.011
66.0	65.5	7879.0	2964.0	4915.0	2562287.8	5783.0	10698.0			1.0						0.011
67.0	66.5	7999.0	3026.4	4972.6	2565564.8	5775.6	10748.2			1.0						0.011
68.0	67.5	8119.0	3088.8	5030.2	2568843.8	5768.2	10798.4			1.0						0.011
69.0	68.5	8239.0	3151.2	5087.8	2572124.8	5760.8	10848.6			1.0						0.011
70.0	69.5	8359.0	3213.6	5145.4	2575407.8	5753.5	10898.9			1.0						0.011
71.0	70.5	8479.0	3276.0	5203.0	2578692.8	5746.2	10949.2			1.0						0.011
72.0	71.5	8599.0	3338.4	5260.6	2581979.8	5738.9	10999.5			1.0						0.011
73.0	72.5	8719.0	3400.8	5318.2	2585268.8	5731.6	11049.8			1.0						0.011
74.0	73.5	8839.0	3463.2	5375.8	2588559.8	5724.3	11100.1			1.0						0.011
75.0	74.5	8959.0	3525.6	5433.4	2591852.8	5717.0	11150.4			1.0						0.011
76.0	75.5	9079.0	3588.0	5491.0	2595147.8	5709.7	11200.7			1.0						0.011
77.0	76.5	9199.0	3650.4	5548.6	2598444.8	5702.5	11251.1			1.0						0.011
78.0	77.5	9319.0	3712.8	5606.2	2601743.8	5695.3	11301.5			1.0						0.011
79.0	78.5	9439.0	3775.2	5663.8	2605044.8	5688.0	11351.8			1.0						0.011
80.0	79.5	9559.0	3837.6	5721.4	2608347.8	5680.8	11402.2			1.0						0.011
81.0	80.5	9679.0	3900.0	5779.0	2611652.8	5673.6	11452.6			1.0						0.011
82.0	81.5	9799.0	3962.4	5836.6	2614959.8	5666.5	11503.1			1.0						0.011
83.0	82.5	9919.0	4024.8	5894.2	2618268.8	5659.3	11553.5			1.0						0.011
84.0	83.5	10039.0	4087.2	5951.8	2621579.8	5652.2	11604.0			1.0						0.011
85.0	84.5	10159.0	4149.6	6009.4	2624892.8	5645.0	11654.4			1.0						0.011
86.0	85.5	10279.0	4212.0	6067.0	2628207.8	5637.9	11704.9			1.0						0.011
87.0	86.5	10399.0	4274.4	6124.6	2631524.8	5630.8	11755.4			1.0						0.011
88.0	87.5	10519.0	4336.8	6182.2	2634843.8	5623.7	11805.9			1.0						0.011
89.0	88.5	10639.0	4399.2	6239.8	2638164.8	5616.6	11856.4			1.0						0.011
90.0	89.5	10759.0	4461.6	6297.4	2641487.8	5609.6	11907.0			1.0						0.011
91.0	90.5	10879.0	4524.0	6355.0	2644812.8	5602.5	11957.5			1.0						0.011
92.0	91.5	10999.0	4586.4	6412.6	2648139.8	5595.5	12008.1			1.0						0.011
93.0	92.5	11119.0	4648.8	6470.2	2651468.8	5588.4	12058.6			1.0						0.011

ATTACHMENT D2

SETTLEMENT ANALYSES (MINIMUM)

Site:	CLIVE FEDERAL CELL	Project No.:	SLC1025
Location:	CLIVE UTAH		
Client:	ES	Date:	20-Jan-23
Prepared by:	M.Downing	Reviewed by:	B.Baturay

Theory

Total settlement made up of three (3) components:

Total Settlement s_t = Immediate Settlement (s_i) + Primary Consolidation (s_c) + Secondary Settlement (s_s)

Primary Consolidation s_c

$$S = C_r H_o/(1+eo) \log[\sigma'c/\sigma'_{vo}] + C_c H_o/1+eo \log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'c]$$

where C_r =recompression index

C_c = compression index

H_o = initial soil layer thickness

$\sigma'c$ = effective preconsolidation pressure = OCR σ'_{vo}

σ'_{vo} = initial effective vertical stress

$\Delta\sigma_v$ = change in vertical effective stress

eo=initial void ratio

Secondary Settlement s_s

$$s_s = C_{\alpha\varepsilon} H_{100} \log(t_2/t_1)$$

where $C_{\alpha\varepsilon}$ = secondary compression index

H_o = thickness of compressible layer at end of primary consolidation

t2 = time for which secondary settlements are calculated (500 years for design life, assume settlement after that is minimul due to log scale projection of creep)

$t_1 = t_{100}$ for primary consolidation - 1 year - estimated by previous analyses of Unit 2 and 4 clay layers (AMEC)

Elastic (Immediate)

Ze=Δσ/Ms *Ho

wher Z =elastic settlement of soil layer

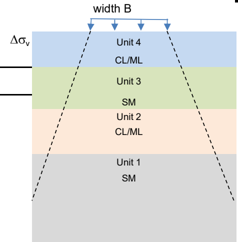
Ho= initial thickness of soil layer

Δσ= change in stress in layer

Ms = constrained modulus of soil estimated with E and v of the insitu soil

CALCULATIONS

New Load for Foundation



Depth (FT BGS): 2, 16, 38, 100

Height of Waste and Cover Materials= 52.5 ft — at the tallest point, including cover

Average Unit Weight of Cover and Waste= 120.0 pcf

$\Delta\sigma_v$ from Loading = 6300.0 psf

B = 1225.0 ft — Based on Cell Limits

L = 1920.0 ft

Unit 4 Unit Weight 103.0 pcf

Unit 3 Unit Weight 109.0 pcf

Unit 2 Unit Weight 100.0 pcf

Unit 1 Unit Weight 123.0 pcf

Unit weight of water 62.4 pcf

Depth to Water = 16.0 ft — gw @ 25' below current grade, approximately 7 feet of upper material to be removed = 16 feet bgs for modeling

Unit 4 C_c =	0.075	Unit 4 eo =	1.12	Unit 3 Ms =	311,040
Unit 4 Cr =	0.005	Unit 2 eo =	1.275	Unit 1 Ms=	531556
Unit 4 $C_{\alpha\varepsilon}$ =	0.00258	Unit 4 OCR =	8	t_1 (t_{100} for primary consolidation)	1
		Unit 2 OCR =	1.6	t2 (compliance period of 10,000 years f)	10000
Unit 2 Cc=	0.069				
Unit 2 Cr =	0.010				
Unit 2 Cαε =	0.00123				

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo} + \Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo} + \Delta\sigma_v < \sigma'c$	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
0.0						6300.0										
1.0	0.5	51.5		51.5	2353572.8	6295.8	6347.3	8.0	412.0	1.0	no	0.044	0.956	0.010	0.054	
2.0	1.5	154.5		154.5	2356719.8	6287.4	6441.9	8.0	1236.0	1.0	no	0.027	0.973	0.010	0.038	
3.0	2.5	263.5		263.5	2359868.8	6279.0	6542.5			1.0						0.020
4.0	3.5	372.5		372.5	2363019.8	6270.6	6643.1			1.0						0.020
5.0	4.5	481.5		481.5	2366172.8	6262.3	6743.8			1.0						0.020
6.0	5.5	590.5		590.5	2369327.8	6253.9	6844.4			1.0						0.020
7.0	6.5	699.5		699.5	2372484.8	6245.6	6945.1			1.0						0.020
8.0	7.5	808.5		808.5	2375643.8	6237.3	7045.8			1.0						0.020
9.0	8.5	917.5		917.5	2378804.8	6229.0	7146.5			1.0						0.020
10.0	9.5	1026.5		1026.5	2381967.8	6220.7	7247.2			1.0						0.020
11.0	10.5	1135.5		1135.5	2385132.8	6212.5	7348.0			1.0						0.020
12.0	11.5	1244.5		1244.5	2388299.8	6204.2	7448.7			1.0						0.020
13.0	12.5	1353.5		1353.5	2391468.8	6196.0	7549.5			1.0						0.020
14.0	13.5	1462.5		1462.5	2394639.8	6187.8	7650.3			1.0						0.020
15.0	14.5	1571.5		1571.5	2397812.8	6179.6	7751.1			1.0						0.020
16.0	15.5	1680.5		1680.5	2400987.8	6171.5	7852.0			1.0						0.020
17.0	16.5	1780.5		1780.5	2404164.8	6163.3	7943.8	1.6	2848.8	1.0	no	0.014	0.986	0.005	0.019	
18.0	17.5	1880.5		1880.5	2407343.8	6155.2	8035.7	1.6	3008.8	1.0	no	0.014	0.986	0.005	0.019	
19.0	18.5	1980.5	156.0	1824.5	2410524.8	6147.0	7971.5	1.6	2919.2	1.0	no	0.014	0.986	0.005	0.019	
20.0	19.5	2080.5	218.4	1862.1	2413707.8	6138.9	8001.0	1.6	2979.4	1.0	no	0.014	0.986	0.005	0.019	
21.0	20.5	2180.5	280.8	1899.7	2416892.8	6130.8	8030.5	1.6	3039.5	1.0	no	0.014	0.986	0.005	0.019	
22.0	21.5	2280.5	343.2	1937.3	2420079.8	6122.8	8060.1	1.6	3099.7	1.0	no	0.013	0.987	0.005	0.018	
23.0	22.5	2380.5	405.6	1974.9	2423268.8	6114.7	8089.6	1.6	3159.8	1.0	no	0.013	0.987	0.005	0.018	
24.0	23.5	2480.5	468.0	2012.5	2426459.8	6106.7	8119.2	1.6	3220.0	1.0	no	0.013	0.987	0.005	0.018	
25.0	24.5	2580.5	530.4	2050.1	2429652.8	6098.6	8148.7	1.6	3280.2	1.0	no	0.013	0.987	0.005	0.018	
26.0	25.5	2680.5	592.8	2087.7	2432847.8	6090.6	8178.3	1.6	3340.3	1.0	no	0.013	0.987	0.005	0.018	
27.0	26.5	2780.5	655.2	2125.3	2436044.8	6082.6	8207.9	1.6	3400.5	1.0	no	0.013	0.987	0.005	0.017	
28.0	27.5	2880.5	717.6	2162.9	2439243.8	6074.7	8237.6	1.6	3460.6	1.0	no	0.012	0.988	0.005	0.017	
29.0	28.5	2980.5	780.0	2200.5	2442444.8	6066.7	8267.2	1.6	3520.8	1.0	no	0.012	0.988	0.005	0.017	
30.0	29.5	3080.5	842.4	2238.1	2445647.8	6058.8	8296.9	1.6	3581.0	1.0	no	0.012	0.988	0.005	0.017	
31.0	30.5	3180.5	904.8	2275.7	2448852.8	6050.8	8326.5	1.6	3641.1	1.0	no	0.012	0.988	0.005	0.017	
32.0	31.5	3280.5	967.2	2313.3	2452059.8	6042.9	8356.2	1.6	3701.3	1.0	no	0.012	0.988	0.005	0.016	
33.0	32.5	3380.5	1029.6	2350.9	2455268.8	6035.0	8385.9	1.6	3761.4	1.0	no	0.011	0.989	0.005	0.016	
34.0	33.5	3480.5	1092.0	2388.5	2458479.8	6027.1	8415.6	1.6	3821.6	1.0	no	0.011	0.989	0.005	0.016	
35.0	34.5	3580.5	1154.4	2426.1	2461692.8	6019.3	8445.4	1.6	3881.8	1.0	no	0.011	0.989	0.005	0.016	
36.0	35.5	3680.5	1216.8	2463.7	2464907.8	6011.4	8475.1	1.6	3941.9	1.0	no	0.011	0.989	0.005	0.016	
37.0	36.5	3780.5	1279.2	2501.3	2468124.8	6003.6	8504.9	1.6	4002.1	1.0	no	0.011	0.989	0.005	0.016	
38.0	37.5	3880.5	1341.6	2538.9	2471343.8	5995.8	8534.7	1.6	4062.2	1.0	no	0.011	0.989	0.005	0.016	
39.0	38.5	4003.5	1404.0	2599.5	2474564.8	5988.0	8587.5			1.0						0.011
40.0	39.5	4126.5	1466.4	2660.1	2477787.8	5980.2	8640.3			1.0						0.011

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo} + \Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo} + \Delta\sigma_v$ < σ'c	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
41.0	40.5	4249.5	1528.8	2720.7	2481012.8	5972.4	8693.1			1.0						0.011
42.0	41.5	4372.5	1591.2	2781.3	2484239.8	5964.6	8745.9			1.0						0.011
43.0	42.5	4495.5	1653.6	2841.9	2487468.8	5956.9	8798.8			1.0						0.011
44.0	43.5	4618.5	1716.0	2902.5	2490699.8	5949.2	8851.7			1.0						0.011
45.0	44.5	4741.5	1778.4	2963.1	2493932.8	5941.5	8904.6			1.0						0.011
46.0	45.5	4864.5	1840.8	3023.7	2497167.8	5933.8	8957.5			1.0						0.011
47.0	46.5	4987.5	1903.2	3084.3	2500404.8	5926.1	9010.4			1.0						0.011
48.0	47.5	5110.5	1965.6	3144.9	2503643.8	5918.4	9063.3			1.0						0.011
49.0	48.5	5233.5	2028.0	3205.5	2506884.8	5910.8	9116.3			1.0						0.011
50.0	49.5	5356.5	2090.4	3266.1	2510127.8	5903.1	9169.2			1.0						0.011
51.0	50.5	5479.5	2152.8	3326.7	2513372.8	5895.5	9222.2			1.0						0.011
52.0	51.5	5602.5	2215.2	3387.3	2516619.8	5887.9	9275.2			1.0						0.011
53.0	52.5	5725.5	2277.6	3447.9	2519868.8	5880.3	9328.2			1.0						0.011
54.0	53.5	5848.5	2340.0	3508.5	2523119.8	5872.7	9381.2			1.0						0.011
55.0	54.5	5971.5	2402.4	3569.1	2526372.8	5865.2	9434.3			1.0						0.011
56.0	55.5	6094.5	2464.8	3629.7	2529627.8	5857.6	9487.3			1.0						0.011
57.0	56.5	6217.5	2527.2	3690.3	2532884.8	5850.1	9540.4			1.0						0.011
58.0	57.5	6340.5	2589.6	3750.9	2536143.8	5842.6	9593.5			1.0						0.011
59.0	58.5	6463.5	2652.0	3811.5	2539404.8	5835.1	9646.6			1.0						0.011
60.0	59.5	6586.5	2714.4	3872.1	2542667.8	5827.6	9699.7			1.0						0.011
61.0	60.5	6709.5	2776.8	3932.7	2545932.8	5820.1	9752.8			1.0						0.011
62.0	61.5	6832.5	2839.2	3993.3	2549199.8	5812.6	9805.9			1.0						0.011
63.0	62.5	6955.5	2901.6	4053.9	2552468.8	5805.2	9859.1			1.0						0.011
64.0	63.5	7078.5	2964.0	4114.5	2555739.8	5797.8	9912.3			1.0						0.011
65.0	64.5	7201.5	3026.4	4175.1	2559012.8	5790.4	9965.5			1.0						0.011
66.0	65.5	7324.5	3088.8	4235.7	2562287.8	5783.0	10018.7			1.0						0.011
67.0	66.5	7447.5	3151.2	4296.3	2565564.8	5775.6	10071.9			1.0						0.011
68.0	67.5	7570.5	3213.6	4356.9	2568843.8	5768.2	10125.1			1.0						0.011
69.0	68.5	7693.5	3276.0	4417.5	2572124.8	5760.8	10178.3			1.0						0.011
70.0	69.5	7816.5	3338.4	4478.1	2575407.8	5753.5	10231.6			1.0						0.011
71.0	70.5	7939.5	3400.8	4538.7	2578692.8	5746.2	10284.9			1.0						0.011
72.0	71.5	8062.5	3463.2	4599.3	2581979.8	5738.9	10338.2			1.0						0.011
73.0	72.5	8185.5	3525.6	4659.9	2585268.8	5731.6	10391.5			1.0						0.011
74.0	73.5	8308.5	3588.0	4720.5	2588559.8	5724.3	10444.8			1.0						0.011
75.0	74.5	8431.5	3650.4	4781.1	2591852.8	5717.0	10498.1			1.0						0.011
76.0	75.5	8554.5	3712.8	4841.7	2595147.8	5709.7	10551.4			1.0						0.011
77.0	76.5	8677.5	3775.2	4902.3	2598444.8	5702.5	10604.8			1.0						0.011
78.0	77.5	8800.5	3837.6	4962.9	2601743.8	5695.3	10658.2			1.0						0.011
79.0	78.5	8923.5	3900.0	5023.5	2605044.8	5688.0	10711.5			1.0						0.011
80.0	79.5	9046.5	3962.4	5084.1	2608347.8	5680.8	10764.9			1.0						0.011
81.0	80.5	9169.5	4024.8	5144.7	2611652.8	5673.6	10818.3			1.0						0.011
82.0	81.5	9292.5	4087.2	5205.3	2614959.8	5666.5	10871.8			1.0						0.011
83.0	82.5	9415.5	4149.6	5265.9	2618268.8	5659.3	10925.2			1.0						0.011
84.0	83.5	9538.5	4212.0	5326.5	2621579.8	5652.2	10978.7			1.0						0.011
85.0	84.5	9661.5	4274.4	5387.1	2624892.8	5645.0	11032.1			1.0						0.011
86.0	85.5	9784.5	4336.8	5447.7	2628207.8	5637.9	11085.6			1.0						0.011
87.0	86.5	9907.5	4399.2	5508.3	2631524.8	5630.8	11139.1			1.0						0.011
88.0	87.5	10030.5	4461.6	5568.9	2634843.8	5623.7	11192.6			1.0						0.011
89.0	88.5	10153.5	4524.0	5629.5	2638164.8	5616.6	11246.1			1.0						0.011
90.0	89.5	10276.5	4586.4	5690.1	2641487.8	5609.6	11299.7			1.0						0.011
91.0	90.5	10399.5	4648.8	5750.7	2644812.8	5602.5	11353.2			1.0						0.011
92.0	91.5	10522.5	4711.2	5811.3	2648139.8	5595.5	11406.8			1.0						0.011
93.0	92.5	10645.5	4773.6	5871.9	2651468.8	5588.4	11460.3			1.0						0.011

SETTLEMENT ANALYSES (MAXIMUM)

Site:	CLIVE FEDERAL CELL	Project No.:	SLC1025
Location:	CLIVE UTAH		
Client:	ES	Date:	20-Jan-23
Prepared by:	M.Downing	Reviewed by:	B.Baturay

Theory

Total settlement made up of three (3) components:

Total Settlement s_t = Immediate Settlement (s_i) + Primary Consolidation (s_c) + Secondary Settlement (s_s)

Primary Consolidation s_c

$$S = C_r\, H_o/(1+eo)\, \log[\sigma'c/\sigma'_{vo}] + C_c\, H_o/1+eo\, \log[(\sigma'_{vo} + \Delta\sigma_v)/\sigma'c]$$

where C_r =recompression index

C_c = compression index

H_o = initial soil layer thickness

$\sigma'c$ = effective preconsolidation pressure = OCR σ'_{vo}

σ'_{vo} = initial effective vertical stress

$\Delta\sigma_v$ = change in vertical effective stress

eo=initial void ratio

Secondary Settlement s_s

$$s_s = C_{\alpha\varepsilon}\, H_{100}\, \log(t_2/t_1)$$

where $C_{\alpha\varepsilon}$ = secondary compression index

H_o = thickness of compressible layer at end of primary consolidation

t2 = time for which secondary settlements are calculated (500 years for design life, assume settlement after that is minimul due to log scale projection of creep)

$t_1 = t_{100}$ for primary consolidation - 1 year - estimated by previous analyses of Unit 2 and 4 clay layers (AMEC)

Elastic (Immediate)

Ze=Δσ/Ms *Ho

wher Z =elastic settlement of soil layer

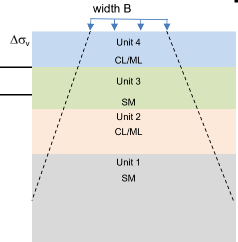
Ho= initial thickness of soil layer

Δσ= change in stress in layer

Ms = constrained modulus of soil estimated with E and v of the insitu soil

CALCULATIONS

New Load for Foundation



Depth (FT BGS): 2, 16, 38, 100

Height of Waste and Cover Materials=	52.5	ft	at the tallest point, including cover
Average Unit Weight of Cover and Waste=	120.0	pcf	
$\Delta\sigma_v$ from Loading =	6300.0	psf	
B =	1225.0	ft	Based on Cell Limits
L =	1920.0	ft	
Unit 4 Unit Weight	103.0	pcf	
Unit 3 Unit Weight	109.0	pcf	
Unit 2 Unit Weight	100.0	pcf	
Unit 1 Unit Weight	123.0	pcf	
Unit weight of water	62.4	pcf	
Depth to Water =	16.0	ft	gw @ 25' below current grade, approximately 7 feet of upper material to be removed = 16 feet bgs for modeling

Unit 4 C_c =	0.300	Unit 4 eo =	0.68	Unit 3 Ms =	311,040
Unit 4 Cr =	0.017	Unit 2 eo =	0.326	Unit 1 Ms=	531556
Unit 4 $C_{\alpha\varepsilon}$ =	0.004	Unit 4 OCR =	2.81	t_1 (t_{100} for primary consolidation)	1
		Unit 2 OCR =	1	t2 (compliance period of 10,000 years f)	10000
Unit 2 Cc=	0.186				
Unit 2 Cr =	0.030				
Unit 2 Cαε =	0.00480				

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo} + \Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo} + \Delta\sigma_v < \sigma'c$	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
0.0						6300.0										
1.0	0.5	51.5		51.5	2353572.8	6295.8	6347.3	2.8	144.7	1.0	no	0.298	0.702	0.011	0.309	
2.0	1.5	154.5		154.5	2356719.8	6287.4	6441.9	2.8	434.1	1.0	no	0.214	0.786	0.013	0.226	
3.0	2.5	263.5		263.5	2359868.8	6279.0	6542.5			1.0						0.020
4.0	3.5	372.5		372.5	2363019.8	6270.6	6643.1			1.0						0.020
5.0	4.5	481.5		481.5	2366172.8	6262.3	6743.8			1.0						0.020
6.0	5.5	590.5		590.5	2369327.8	6253.9	6844.4			1.0						0.020
7.0	6.5	699.5		699.5	2372484.8	6245.6	6945.1			1.0						0.020
8.0	7.5	808.5		808.5	2375643.8	6237.3	7045.8			1.0						0.020
9.0	8.5	917.5		917.5	2378804.8	6229.0	7146.5			1.0						0.020
10.0	9.5	1026.5		1026.5	2381967.8	6220.7	7247.2			1.0						0.020
11.0	10.5	1135.5		1135.5	2385132.8	6212.5	7348.0			1.0						0.020
12.0	11.5	1244.5		1244.5	2388299.8	6204.2	7448.7			1.0						0.020
13.0	12.5	1353.5		1353.5	2391468.8	6196.0	7549.5			1.0						0.020
14.0	13.5	1462.5		1462.5	2394639.8	6187.8	7650.3			1.0						0.020
15.0	14.5	1571.5		1571.5	2397812.8	6179.6	7751.1			1.0						0.020
16.0	15.5	1680.5		1680.5	2400987.8	6171.5	7852.0			1.0						0.020
17.0	16.5	1780.5		1780.5	2404164.8	6163.3	7943.8	1.0	1780.5	1.0	no	0.091	0.909	0.017	0.109	
18.0	17.5	1880.5		1880.5	2407343.8	6155.2	8035.7	1.0	1880.5	1.0	no	0.088	0.912	0.018	0.106	
19.0	18.5	1980.5	156.0	1824.5	2410524.8	6147.0	7971.5	1.0	1824.5	1.0	no	0.090	0.910	0.017	0.107	
20.0	19.5	2080.5	218.4	1862.1	2413707.8	6138.9	8001.0	1.0	1862.1	1.0	no	0.089	0.911	0.017	0.106	
21.0	20.5	2180.5	280.8	1899.7	2416892.8	6130.8	8030.5	1.0	1899.7	1.0	no	0.088	0.912	0.018	0.105	
22.0	21.5	2280.5	343.2	1937.3	2420079.8	6122.8	8060.1	1.0	1937.3	1.0	no	0.087	0.913	0.018	0.104	
23.0	22.5	2380.5	405.6	1974.9	2423268.8	6114.7	8089.6	1.0	1974.9	1.0	no	0.086	0.914	0.018	0.103	
24.0	23.5	2480.5	468.0	2012.5	2426459.8	6106.7	8119.2	1.0	2012.5	1.0	no	0.085	0.915	0.018	0.103	
25.0	24.5	2580.5	530.4	2050.1	2429652.8	6098.6	8148.7	1.0	2050.1	1.0	no	0.084	0.916	0.018	0.102	
26.0	25.5	2680.5	592.8	2087.7	2432847.8	6090.6	8178.3	1.0	2087.7	1.0	no	0.083	0.917	0.018	0.101	
27.0	26.5	2780.5	655.2	2125.3	2436044.8	6082.6	8207.9	1.0	2125.3	1.0	no	0.082	0.918	0.018	0.100	
28.0	27.5	2880.5	717.6	2162.9	2439243.8	6074.7	8237.6	1.0	2162.9	1.0	no	0.081	0.919	0.018	0.099	
29.0	28.5	2980.5	780.0	2200.5	2442444.8	6066.7	8267.2	1.0	2200.5	1.0	no	0.081	0.919	0.018	0.098	
30.0	29.5	3080.5	842.4	2238.1	2445647.8	6058.8	8296.9	1.0	2238.1	1.0	no	0.080	0.920	0.018	0.097	
31.0	30.5	3180.5	904.8	2275.7	2448852.8	6050.8	8326.5	1.0	2275.7	1.0	no	0.079	0.921	0.018	0.097	
32.0	31.5	3280.5	967.2	2313.3	2452059.8	6042.9	8356.2	1.0	2313.3	1.0	no	0.078	0.922	0.018	0.096	
33.0	32.5	3380.5	1029.6	2350.9	2455268.8	6035.0	8385.9	1.0	2350.9	1.0	no	0.077	0.923	0.018	0.095	
34.0	33.5	3480.5	1092.0	2388.5	2458479.8	6027.1	8415.6	1.0	2388.5	1.0	no	0.077	0.923	0.018	0.094	
35.0	34.5	3580.5	1154.4	2426.1	2461692.8	6019.3	8445.4	1.0	2426.1	1.0	no	0.076	0.924	0.018	0.094	
36.0	35.5	3680.5	1216.8	2463.7	2464907.8	6011.4	8475.1	1.0	2463.7	1.0	no	0.075	0.925	0.018	0.093	
37.0	36.5	3780.5	1279.2	2501.3	2468124.8	6003.6	8504.9	1.0	2501.3	1.0	no	0.075	0.925	0.018	0.092	
38.0	37.5	3880.5	1341.6	2538.9	2471343.8	5995.8	8534.7	1.0	2538.9	1.0	no	0.074	0.926	0.018	0.092	
39.0	38.5	4003.5	1404.0	2599.5	2474564.8	5988.0	8587.5			1.0						0.011
40.0	39.5	4126.5	1466.4	2660.1	2477787.8	5980.2	8640.3			1.0						0.011

Depth (ft)	Depth of Midpt (ft)	σ_{vo} (psf)	u (psf)	σ'_{vo} (psf)	Effective Mat Area (sf)	$\Delta\sigma_v$ (psf)	$\sigma'_{vo} + \Delta\sigma_v$ (psf)	OCR	σ'c (psf)	H_o (ft)	$\sigma'_{vo} + \Delta\sigma_v$ < σ'c	$S_{consolidation}$ (ft)	H_{100}	$S_{secondary}$ (ft)	S c+s (ft)	Ze (ft)
41.0	40.5	4249.5	1528.8	2720.7	2481012.8	5972.4	8693.1			1.0						0.011
42.0	41.5	4372.5	1591.2	2781.3	2484239.8	5964.6	8745.9			1.0						0.011
43.0	42.5	4495.5	1653.6	2841.9	2487468.8	5956.9	8798.8			1.0						0.011
44.0	43.5	4618.5	1716.0	2902.5	2490699.8	5949.2	8851.7			1.0						0.011
45.0	44.5	4741.5	1778.4	2963.1	2493932.8	5941.5	8904.6			1.0						0.011
46.0	45.5	4864.5	1840.8	3023.7	2497167.8	5933.8	8957.5			1.0						0.011
47.0	46.5	4987.5	1903.2	3084.3	2500404.8	5926.1	9010.4			1.0						0.011
48.0	47.5	5110.5	1965.6	3144.9	2503643.8	5918.4	9063.3			1.0						0.011
49.0	48.5	5233.5	2028.0	3205.5	2506884.8	5910.8	9116.3			1.0						0.011
50.0	49.5	5356.5	2090.4	3266.1	2510127.8	5903.1	9169.2			1.0						0.011
51.0	50.5	5479.5	2152.8	3326.7	2513372.8	5895.5	9222.2			1.0						0.011
52.0	51.5	5602.5	2215.2	3387.3	2516619.8	5887.9	9275.2			1.0						0.011
53.0	52.5	5725.5	2277.6	3447.9	2519868.8	5880.3	9328.2			1.0						0.011
54.0	53.5	5848.5	2340.0	3508.5	2523119.8	5872.7	9381.2			1.0						0.011
55.0	54.5	5971.5	2402.4	3569.1	2526372.8	5865.2	9434.3			1.0						0.011
56.0	55.5	6094.5	2464.8	3629.7	2529627.8	5857.6	9487.3			1.0						0.011
57.0	56.5	6217.5	2527.2	3690.3	2532884.8	5850.1	9540.4			1.0						0.011
58.0	57.5	6340.5	2589.6	3750.9	2536143.8	5842.6	9593.5			1.0						0.011
59.0	58.5	6463.5	2652.0	3811.5	2539404.8	5835.1	9646.6			1.0						0.011
60.0	59.5	6586.5	2714.4	3872.1	2542667.8	5827.6	9699.7			1.0						0.011
61.0	60.5	6709.5	2776.8	3932.7	2545932.8	5820.1	9752.8			1.0						0.011
62.0	61.5	6832.5	2839.2	3993.3	2549199.8	5812.6	9805.9			1.0						0.011
63.0	62.5	6955.5	2901.6	4053.9	2552468.8	5805.2	9859.1			1.0						0.011
64.0	63.5	7078.5	2964.0	4114.5	2555739.8	5797.8	9912.3			1.0						0.011
65.0	64.5	7201.5	3026.4	4175.1	2559012.8	5790.4	9965.5			1.0						0.011
66.0	65.5	7324.5	3088.8	4235.7	2562287.8	5783.0	10018.7			1.0						0.011
67.0	66.5	7447.5	3151.2	4296.3	2565564.8	5775.6	10071.9			1.0						0.011
68.0	67.5	7570.5	3213.6	4356.9	2568843.8	5768.2	10125.1			1.0						0.011
69.0	68.5	7693.5	3276.0	4417.5	2572124.8	5760.8	10178.3			1.0						0.011
70.0	69.5	7816.5	3338.4	4478.1	2575407.8	5753.5	10231.6			1.0						0.011
71.0	70.5	7939.5	3400.8	4538.7	2578692.8	5746.2	10284.9			1.0						0.011
72.0	71.5	8062.5	3463.2	4599.3	2581979.8	5738.9	10338.2			1.0						0.011
73.0	72.5	8185.5	3525.6	4659.9	2585268.8	5731.6	10391.5			1.0						0.011
74.0	73.5	8308.5	3588.0	4720.5	2588559.8	5724.3	10444.8			1.0						0.011
75.0	74.5	8431.5	3650.4	4781.1	2591852.8	5717.0	10498.1			1.0						0.011
76.0	75.5	8554.5	3712.8	4841.7	2595147.8	5709.7	10551.4			1.0						0.011
77.0	76.5	8677.5	3775.2	4902.3	2598444.8	5702.5	10604.8			1.0						0.011
78.0	77.5	8800.5	3837.6	4962.9	2601743.8	5695.3	10658.2			1.0						0.011
79.0	78.5	8923.5	3900.0	5023.5	2605044.8	5688.0	10711.5			1.0						0.011
80.0	79.5	9046.5	3962.4	5084.1	2608347.8	5680.8	10764.9			1.0						0.011
81.0	80.5	9169.5	4024.8	5144.7	2611652.8	5673.6	10818.3			1.0						0.011
82.0	81.5	9292.5	4087.2	5205.3	2614959.8	5666.5	10871.8			1.0						0.011
83.0	82.5	9415.5	4149.6	5265.9	2618268.8	5659.3	10925.2			1.0						0.011
84.0	83.5	9538.5	4212.0	5326.5	2621579.8	5652.2	10978.7			1.0						0.011
85.0	84.5	9661.5	4274.4	5387.1	2624892.8	5645.0	11032.1			1.0						0.011
86.0	85.5	9784.5	4336.8	5447.7	2628207.8	5637.9	11085.6			1.0						0.011
87.0	86.5	9907.5	4399.2	5508.3	2631524.8	5630.8	11139.1			1.0						0.011
88.0	87.5	10030.5	4461.6	5568.9	2634843.8	5623.7	11192.6			1.0						0.011
89.0	88.5	10153.5	4524.0	5629.5	2638164.8	5616.6	11246.1			1.0						0.011
90.0	89.5	10276.5	4586.4	5690.1	2641487.8	5609.6	11299.7			1.0						0.011
91.0	90.5	10399.5	4648.8	5750.7	2644812.8	5602.5	11353.2			1.0						0.011
92.0	91.5	10522.5	4711.2	5811.3	2648139.8	5595.5	11406.8			1.0						0.011
93.0	92.5	10645.5	4773.6	5871.9	2651468.8	5588.4	11460.3			1.0						0.011

ATTACHMENT E

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SLC Federal Cell Clive Fa
Location: Salt Lake City, Utah

Project Number: SLC1025
Prepared By: M.Downing

Checked by:
Date: 3/11/2021

Boring: GW-36
Date: 23-Dec-91
By: Overland Drilling

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Hollow Stem Auger
Ground Elevation (ft)[2]: 0.00

$a_{max\ (ground\ surface)}$: 0.24 g[3]
Earthquake Magnitude: 7.3 [3]
MSF: 1.05 [4]
Assumed depth to groundwater at time of earthquake (ft)[24]: 0.0
Assumed depth to groundwater at time of drilling (ft)[24]: 20.6

Depth (ft)	Elevation (ft)	Soil Unit Weight (pcf)	Soil Unit	USCS Class	Borehole Diameter (mm)	Sample Type	ER[5] (%)	N_{field} (blows/ft)	σ_v (psf)	σ_v', during drilling (psf)	σ_v', during EQ[24] (psf)	N_{field} Correction Factors: C_{rod}[6]	C_{energy}[7]	C_b[8]	Cs[9]	C_{SPT}[10]	N_{60} (blows/ft)
0	0.0																
12.0	-12.0	118	Unit 4 Silty CLAY	CL	108.0	SPT	72	9	1416	1416	667	0.80	1.20	1.00	1.00	1.00	9
14.0	-14.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	55	1656	1656	782	0.85	1.20	1.00	1.00	1.00	56
16.0	-16.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	61	1896	1896	898	0.85	1.20	1.00	1.00	1.00	62
18.0	-18.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	32	2136	2136	1013	0.85	1.20	1.00	1.00	1.00	33

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"

[2] Boring location known to exist somewhere in Section 32 of the Clive Facility

[3] a_{max} and earthquake magnitude based on parameters presented in the seismis hazard analysis by AMEC 2012

[4] ``

[5] Estimated to result in C_{energy} of 0.8 assuming Autohammer

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. CSPT is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] $C_N=(P_a/\sigma'_v)^m \leq 1.7$ accounts for effective overburden stress (Equation 33, I&B 2008)

Boring: GW-36 (continued from previous page)

Date: 23-Dec-91

By: Overland Drilling

Fines Content %	Fines Content Method	[11] m	[12] C_N	[13] $(N_1)_{60}$ (blows/ft)	[14] $\Delta(N_1)_{60}$	[15] $(N_1)_{60cs}$ (blows/ft)	[16] α	[17] β	[18] r_d	[19] C_σ	[20] K_σ	[21] $CRR_{M7.5,\sigma'vc}$	[22] $CSR_{M7.5,\sigma'vc}$	[25] $\Delta(N_1)_{60}$-FC%	$(N_1)_{60CS-Sr}$	FS	[27] γ_{lim}	[28] Fα	[29] γ_{max}	[30] ΔHi	[31] εv	[32] Δsi	Cum Settlen
																							0.00
100.0	Est	0.477	1.21	10	5.5	16	-0.17	0.02	0.97	0.115	1.100	0.16	0.277	5	15	0.59							
15.0	Est	0.264	1.07	60	3.3	63	-0.22	0.02	0.96	0.300	1.100	50.00	0.274	1	61	182.15							
15.0	Est	0.264	1.03	64	3.3	67	-0.26	0.03	0.96	0.300	1.100	50.00	0.272	1	65	183.97							
15.0	Est	0.324	1.00	33	3.3	36	-0.30	0.03	0.95	0.275	1.100	1.32	0.269	1	34	4.90							
																					Settlement	0.00	ft
																					Settlement	0.0	in

[13] $(N_1)_{60}=N_{60}*C_N$ is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}=\exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2]$ represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}=(N_1)_{60} + \Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] $\alpha(z) = -1.012-1.126\sin((z/11.73)+5.133)$ in which z is depth in meters (Equation 23, I&B 2008)

[17] $\beta(z) = 0.106+0.118\sin((z/11.28)+5.142)$ in which z is depth in meters (Equation 24, I&B 2008)

[18] $r_d=\exp[\alpha(z)+\beta(z)M]$ is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] $C_\sigma=1/(18.9-2.55\mathrm{sqrt}[(N_1)_{60cs}]\leq 0.3$ is the coefficient for K_σ (Equation 56, I&B 2008)

[20] $K_\sigma = 1-C_\sigma\ln(\sigma_{vo}'/P_a)\leq 1.1$ is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}=0.65(a_{max}/g)(\sigma_v/\sigma_v')r_d(1/MSF)(1/K_\sigma)$ is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable

[24] Groundwater assumed to be at a depth of 170 feet below ground surface during the field investigation (for blow count correction)

[25] Fines content correction for liquefied shear strength from Seed 1987 (Table 4, pg 126, I&B 2008)

[26] MOD-CAL refers to 2.5-inch ID sampler

[27] $\gamma_{lim} = 1.859[1.1 - \mathrm{sqrt}((N_1)_{60cs}/46)]^3 > 0$ but less than 50% = limiting shear strain (Equation 86, I&B, 2008)

[28] $F\alpha = 0.032 + 0.69\mathrm{sqrt}[(N_1)_{60cs}] - 0.13(N_1)_{60cs}$, where $(N_1)_{60cs}$ is limited to values > 7 (Equation 93, I&B, 2008)

[29] $\gamma_{max} = \min[\gamma_{lim}, 0.35(2-FS)((1-F\alpha)/(FS-F\alpha)]$ for 2 > FS > Fα; if FS < Fα, $\gamma_{max} = \gamma_{lim}$ (Equations 91 & 92, I&B, 2008)

[30] ΔHi = Layer thickness (ft)

[31] $\varepsilon_v = 1.5\exp(-0.369\mathrm{sqrt}[(N_1)_{60cs}]$ x $[\min(0.08, \gamma_{max})]$ = post liquefaction volumetric strain (Equation 96, I&B, 2008)

[32] ΔSi = (Δhi)(εv)

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Project: SLC Federal Cell Clive Fa
Location: Salt Lake City, Utah

Project Number: SLC1025
Prepared By: M.Downing

Checked by:
Date:

Boring: GW-37
Date: 23-Dec-91
By: Overland Drilling

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Hollow Stem Auger
Ground Elevation (ft)[2]: 0.00

$a_{max\ (ground\ surface)}$: 0.24 g[3]
Earthquake Magnitude: 7.3 [3]
MSF: 1.05 [4]
Assumed depth to groundwater at time of earthquake (ft)[24]: 0.0
Assumed depth to groundwater at time of drilling (ft)[24]: 19.2

Depth (ft)	Elevation (ft)	Soil Unit Weight (pcf)	Soil Unit	USCS Class	Borehole Diameter (mm)	Sample Type	ER[5] (%)	N_{field} (blows/ft)	σ_v (psf)	σ_v', during drilling (psf)	σ_v', during EQ[24] (psf)	N_{field} Correction Factors C_{rod}[6]	C_{energy}[7]	C_b[8]	Cs[9]	C_{SPT}[10]	N_{60} (blows/ft)
0	0.0																
7.0	-7.0	118	Unit 4 Silty CLAY	CL	108.0	SPT	72	11	826	826	389	0.75	1.20	1.00	1.00	1.00	10
10.0	-10.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	27	1186	1186	562	0.80	1.20	1.00	1.00	1.00	26
12.0	-12.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	25	1426	1426	677	0.80	1.20	1.00	1.00	1.00	24
14.0	-14.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	29	1666	1666	792	0.85	1.20	1.00	1.00	1.00	30
16.0	-16.0	120	CLAY lens	CL	108.0	SPT	72	22	1906	1906	908	0.85	1.20	1.00	1.00	1.00	22
17.0	-17.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	30	2026	2026	965	0.85	1.20	1.00	1.00	1.00	31

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"

[2] Boring location known to exist somewhere in Section 32 of the Clive Facility

[3] a_{max} and earthquake magnitude based on parameters presented in the seismis hazard analysis by AMEC 2012

[4] ``

[5] Estimated to result in C_{energy} of 0.8 assuming Autohammer

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. CSPT is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] $m=0.784-0.0768\text{sqrt}((N_1)_{60cs})\geq0.264$ is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] $C_N=(P_a/\sigma'_v)^m\leq1.7$ accounts for effective overburden stress (Equation 33, I&B 2008)

Boring: GW-37 (continued from previous page)

Date: 23-Dec-91

By: Overland Drilling

Fines Content %	Fines Content Method	[11] m	[12] C_N	$(N_1)_{60}$ [13] (blows/ft)	$\Delta(N_1)_{60}$ [14]	$(N_1)_{60cs}$ [15] (blows/ft)	[16] α	[17] β	[18] r_d	[19] C_σ	[20] K_σ	[21] $CRR_{M7.5,\sigma'vc}$	[22] $CSR_{M7.5,\sigma'vc}$	[25] $\Delta(N_1)_{60}$-FC%	$(N_1)_{60CS-Sr}$	FS	[27] γ_{lim}	[28] Fα	[29] γ_{max}	[30] ΔHi	[31] εv	[32] Δsi	Cum Settlen
																							0.00
100.0	Est	0.437	1.51	15	5.5	20	-0.08	0.01	0.99	0.136	1.100	0.21	0.282	5	20	0.75							0.00
15.0	Est	0.332	1.21	31	3.3	35	-0.14	0.02	0.98	0.257	1.100	1.04	0.278	1	32	3.73							
15.0	Est	0.357	1.15	28	3.3	31	-0.17	0.02	0.97	0.212	1.100	0.55	0.275	1	29	1.99							
15.0	Est	0.328	1.08	32	3.3	35	-0.22	0.02	0.96	0.266	1.100	1.17	0.273	1	33	4.29							
100.0	Est	0.372	1.04	23	5.5	29	-0.26	0.03	0.96	0.192	1.100	0.42	0.270	5	28	1.55							
15.0	Est	0.334	1.01	31	3.3	34	-0.28	0.03	0.95	0.252	1.100	0.96	0.269	1	32	3.59							

Settlement	0.00	ft
Settlement	0.0	in

[13] $(N_1)_{60}=N_{60}*C_N$ is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}=\exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2]$ represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}=(N_1)_{60}+\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] $\alpha(z) = -1.012-1.126\sin((z/11.73)+5.133)$ in which z is depth in meters (Equation 23, I&B 2008)

[17] $\beta(z) = 0.106+0.118\sin((z/11.28)+5.142)$ in which z is depth in meters (Equation 24, I&B 2008)

[18] $r_d=\exp[\alpha(z)+\beta(z)M]$ is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] $C_\sigma=1/(18.9-2.55\text{sqrt}[(N_1)_{60cs}]\leq 0.3$ is the coefficient for K_σ (Equation 56, I&B 2008)

[20] $K_\sigma = 1-C_\sigma\ln(\sigma_{vo}'/P_a)\leq 1.1$ is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}=0.65(a_{max}/g)(\sigma_v/\sigma_v')r_d(1/MSF)(1/K_\sigma)$ is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable

[24] Groundwater assumed to be at a depth of 170 feet below ground surface during the field investigation (for blow count correction)

[25] Fines content correction for liquefied shear strength from Seed 1987 (Table 4, pg 126, I&B 2008)

[26] MOD-CAL refers to 2.5-inch ID sampler

[27] $\gamma_{lim} = 1.859[1.1 - \text{sqrt}((N_1)_{60cs}/46)]^3 > 0$ but less than 50% = limiting shear strain (Equation 86, I&B, 2008)

[28] $F\alpha = 0.032 + 0.69\text{sqrt}[(N_1)_{60cs}] - 0.13(N_1)_{60cs}$, where $(N_1)_{60cs}$ is limited to values > 7 (Equation 93, I&B, 2008)

[29] $\gamma_{max} = \min[\gamma_{lim}, 0.35(2-FS)((1-F\alpha)/(FS-F\alpha)]$ for 2 > FS > Fα; if FS < Fα, $\gamma_{max} = \gamma_{lim}$ (Equations 91 & 92, I&B, 2008)

[30] ΔHi = Layer thickness (ft)

[31] $\varepsilon_v = 1.5\exp(-0.369\text{sqrt}[(N_1)_{60cs}]$ x $[\min(0.08, \gamma_{max})]$ = post liquefaction volumetric strain (Equation 96, I&B, 2008)

[32] ΔSi = (Δhi)(εv)

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SLC Federal Cell Clive Fa
Location: Salt Lake City, Utah

Project Number: SLC1025
Prepared By: M.Downing

Checked by:
Date:

Boring: GW-38
Date: 24-Dec-91
By: Overland Drilling

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Hollow Stem Auger
Ground Elevation (ft)[2]: 0.00

$a_{max\ (ground\ surface)}$: 0.24 g[3]
Earthquake Magnitude: 7.3 [3]
MSF: 1.05 [4]
Assumed depth to groundwater at time of earthquake (ft)[24]: 0.0
Assumed depth to groundwater at time of drilling (ft)[24]: 20.7

Depth (ft)	Elevation (ft)	Soil Unit Weight (pcf)	Soil Unit	USCS Class	Borehole Diameter (mm)	Sample Type	ER[5] (%)	N_{field} (blows/ft)	σ_v (psf)	σ_v', during drilling (psf)	σ_v', during EQ[24] (psf)	N_{field} Correction Factors C_{rod}[6]	C_{energy}[7]	C_b[8]	Cs[9]	C_{SPT}[10]	N_{60} (blows/ft)
0	0.0																
7.0	-7.0	118	Unit 4 Silty CLAY	CL	108.0	SPT	72	15	826	826	389	0.75	1.20	1.00	1.00	1.00	14
10.0	-10.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	21	1186	1186	562	0.80	1.20	1.00	1.00	1.00	20
12.0	-12.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	63	1426	1426	677	0.80	1.20	1.00	1.00	1.00	60
14.0	-14.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	31	1666	1666	792	0.85	1.20	1.00	1.00	1.00	32
16.0	-16.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	20	1906	1906	908	0.85	1.20	1.00	1.00	1.00	20
18.0	-18.0	120	Unit 3 Silty Sand	SM	108.0	SPT	72	25	2146	2146	1023	0.85	1.20	1.00	1.00	1.00	26

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"
[2] Boring location known to exist somewhere in Section 32 of the Clive Facility
[3] a_{max} and earthquake magnitude based on parameters presented in the seismis hazard analysis by AMEC 2012
[4] ``
[5] Estimated to result in C_{energy} of 0.8 assuming Autohammer
[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)
[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)
[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)
[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)
[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. CSPT is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)
[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)
[12] $C_N=(P_a/\sigma'_v)^m \leq 1.7$ accounts for effective overburden stress (Equation 33, I&B 2008)

Boring: GW-38 (continued from previous page)

Date: 24-Dec-91

By: Overland Drilling

Fines Content %	Fines Content Method	[11] m	[12] C_N	$(N_1)_{60}$ [13] (blows/ft)	$\Delta(N_1)_{60}$ [14]	$(N_1)_{60cs}$ [15] (blows/ft)	[16] α	[17] β	[18] r_d	[19] C_σ	[20] K_σ	[21] $CRR_{M7.5,\sigma'vc}$	[22] $CSR_{M7.5,\sigma'vc}$	[25] $\Delta(N_1)_{60}$-FC%	$(N_1)_{60CS-Sr}$	FS	[27] γ_{lim}	[28] Fα	[29] γ_{max}	[30] ΔHi	[31] εv	[32] Δsi	Cum Settlen
																							0.02
100.0	Est	0.399	1.46	20	5.5	25	-0.08	0.01	0.99	0.164	1.100	0.29	0.282	5	25	1.04							0.02
15.0	Est	0.375	1.24	25	3.3	28	-0.14	0.02	0.98	0.188	1.100	0.40	0.278	1	26	1.43							
15.0	Est	0.264	1.11	67	3.3	70	-0.17	0.02	0.97	0.300	1.100	50.00	0.275	1	68	181.73							
15.0	Est	0.315	1.08	34	3.3	37	-0.22	0.02	0.96	0.300	1.100	1.91	0.273	1	35	7.02							
15.0	Est	0.404	1.04	21	3.3	25	-0.26	0.03	0.96	0.160	1.100	0.28	0.270	1	22	1.03	9.4%	0.26	3.2%	2.0	0.8%	0.02	-0.02
15.0	Est	0.373	0.99	25	3.3	29	-0.30	0.03	0.95	0.190	1.100	0.41	0.268	1	26	1.53							

Settlement	0.02	ft
Settlement	0.2	in

[13] $(N_1)_{60}=N_{60}*C_N$ is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}=\exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2]$ represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}=(N_1)_{60}+\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] $\alpha(z) = -1.012-1.126\sin((z/11.73)+5.133)$ in which z is depth in meters (Equation 23, I&B 2008)

[17] $\beta(z) = 0.106+0.118\sin((z/11.28)+5.142)$ in which z is depth in meters (Equation 24, I&B 2008)

[18] $r_d=\exp[\alpha(z)+\beta(z)M]$ is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] $C_\sigma=1/(18.9-2.55\text{sqrt}[(N_1)_{60cs}]\leq 0.3$ is the coefficient for K_σ (Equation 56, I&B 2008)

[20] $K_\sigma = 1-C_\sigma\ln(\sigma_{vo}'/P_a)\leq 1.1$ is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}=0.65(a_{max}/g)(\sigma_v/\sigma_v')r_d(1/MSF)(1/K_\sigma)$ is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable

[24] Groundwater assumed to be at a depth of 170 feet below ground surface during the field investigation (for blow count correction)

[25] Fines content correction for liquefied shear strength from Seed 1987 (Table 4, pg 126, I&B 2008)

[26] MOD-CAL refers to 2.5-inch ID sampler

[27] $\gamma_{lim} = 1.859[1.1 - \text{sqrt}((N_1)_{60cs}/46)]^3 > 0$ but less than 50% = limiting shear strain (Equation 86, I&B, 2008)

[28] $F\alpha = 0.032 + 0.69\text{sqrt}[(N_1)_{60cs}] - 0.13(N_1)_{60cs}$, where $(N_1)_{60cs}$ is limited to values > 7 (Equation 93, I&B, 2008)

[29] $\gamma_{max} = \min[\gamma_{lim}, 0.35(2-FS)((1-F\alpha)/(FS-F\alpha)]$ for 2 > FS > Fα; if FS < Fα, $\gamma_{max} = \gamma_{lim}$ (Equations 91 & 92, I&B, 2008)

[30] ΔHi = Layer thickness (ft)

[31] $\varepsilon_v = 1.5\exp(-0.369\text{sqrt}[(N_1)_{60cs}]$ x $[\min(0.08, \gamma_{max})]$ = post liquefaction volumetric strain (Equation 96, I&B, 2008)

[32] ΔSi = (Δhi)(εv)

Geosyntec consultants

ATTACHMENT E2

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: Federal Cell | Project Number: SLC1025 | Checked by: B.Baturay
Location: Salt Lake City, Utah | Prepared By: M.Downing | Date: 1/19/2023

Boring: Multiple | Hammer Type: Automatic 140 lb./30-in. | $a_{max\ (ground\ surface)}$: 0.24 g[3]
Date: - | Drilling Method: Hollow Stem Auger | Earthquake Magnitude: 7.3 [3] | 1288
By: Overland Drilling | Ground Elevation (ft)[2]: 0.00 | MSF: 1.05 [4]

Assumed depth to groundwater at time of earthquake (ft)[24]: 0.0
Assumed depth to groundwater at time of drilling (ft)[24]: 20.0

Depth (ft)	Elevation (ft)	Soil Unit Weight (pcf)	Soil Unit	USCS Class	Borehole Diameter (mm)	Sample Type	ER[5] (%)	N_{field} (blows/ft)	σ_v (psf)	σ_v', during drilling (psf)	σ_v', during EQ[24] (psf)	N_{field} Correction Factors: C_{rod}[6]	C_{energy}[7]	C_b[8]	Cs[9]	C_{SPT}[10]	N_{60} (blows/ft)	Fines Content %
0	0.0																	
10.0	4259.84	120	Silty Sand	SM	196.0	SPT	82	54	1160	1160	536	0.80	1.37	1.14	1.00	1.00	67	15.0
12.0	4257.84	120	Silty Sand	SM	196.0	SPT	82	19	1392	1392	643	0.80	1.37	1.14	1.00	1.00	24	15.0
14.0	4255.84	120	Silty Sand	SM	196.0	SPT	82	19	1624	1624	750	0.85	1.37	1.14	1.00	1.00	25	15.0
16.0	4253.84	120	Silty Sand	SM	196.0	SPT	82	32	1856	1856	858	0.85	1.37	1.14	1.00	1.00	42	15.0
18.0	4251.84	120	Silty Sand	SM	196.0	SPT	82	21	2088	2088	965	0.85	1.37	1.14	1.00	1.00	28	15.0
20.0	4249.84	120	Silty Sand	SM	196.0	SPT	82	12	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	18	15.0
22.0	4247.84	120	Silty Sand	SM	196.0	SPT	82	59	2552	2427	1179	0.95	1.37	1.14	1.00	1.00	87	15.0
19.8	4256.7	120	Silty Sand	SM	196.0	SPT	82	12	2297	2297	1061	0.95	1.37	1.14	1.00	1.00	18	15.0
21.8	4254.73	120	Silty Sand	SM	196.0	SPT	82	23	2529	2416	1168	0.95	1.37	1.14	1.00	1.00	34	15.0
23.8	4252.73	120	Silty Sand	SM	196.0	SPT	82	19	2761	2524	1276	0.95	1.37	1.14	1.00	1.00	28	15.0
10.0	4264	120	Silty Sand	SM	196.0	SPT	82	14	1160	1160	536	0.80	1.37	1.14	1.00	1.00	17	15.0
15.0	4259	120	Silty Sand	SM	196.0	SPT	82	36	1740	1740	804	0.85	1.37	1.14	1.00	1.00	48	15.0
20	4248.9	120	Silty Sand	SM	196	SPT	82	18	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	27	15.0
25	4243.9	120	Silty Sand	SM	196	SPT	82	38	2900	2588	1340	0.95	1.37	1.14	1.00	1.00	56	15.0
8	4266	120	Silty Sand	SM	196	SPT	82	32	928	928	429	0.75	1.37	1.14	1.00	1.00	37	15.0
10	4264	120	Silty Sand	SM	196	SPT	82	57	1160	1160	536	0.80	1.37	1.14	1.00	1.00	71	15.0
12	4262	120	Silty Sand	SM	196	SPT	82	29	1392	1392	643	0.80	1.37	1.14	1.00	1.00	36	15.0
16	4258	120	Silty Sand	SM	196	SPT	82	21	1856	1856	858	0.85	1.37	1.14	1.00	1.00	28	15.0
18	4256	120	Silty Sand	SM	196	SPT	82	22	2088	2088	965	0.85	1.37	1.14	1.00	1.00	29	15.0
22	4252	120	Silty Sand	SM	196	SPT	82	21	2552	2427.2	1179	0.95	1.37	1.14	1.00	1.00	31	15.0
24	4250	120	Silty Sand	SM	196	SPT	82	21	2784	2534.4	1286	0.95	1.37	1.14	1.00	1.00	31	15.0
12	4258	120	Silty Sand	SM	196	SPT	82	33	1392	1392	643	0.80	1.37	1.14	1.00	1.00	41	15.0
14	4256	120	Silty Sand	SM	196	SPT	82	39	1624	1624	750	0.85	1.37	1.14	1.00	1.00	52	15.0
16	4254	120	Silty Sand	SM	196	SPT	82	51	1856	1856	858	0.85	1.37	1.14	1.00	1.00	68	15.0
18	4252	120	Silty Sand	SM	196	SPT	82	13	2088	2088	965	0.85	1.37	1.14	1.00	1.00	17	15.0
20	4250	120	Silty Sand	SM	196	SPT	82	21	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	31	15.0
24	4246	120	Silty Sand	SM	196	SPT	82	93	2784	2534.4	1286	0.95	1.37	1.14	1.00	1.00	138	15.0
26	4244	120	Silty Sand	SM	196	SPT	82	30	3016	2641.6	1394	0.95	1.37	1.14	1.00	1.00	44	15.0
14	4255.36	120	Silty Sand	SM	196	SPT	82	92	1624	1624	750	0.85	1.37	1.14	1.00	1.00	122	15.0
16	4253.36	120	Silty Sand	SM	196	SPT	82	17	1856	1856	858	0.85	1.37	1.14	1.00	1.00	23	15.0
20	4249.36	120	Silty Sand	SM	196	SPT	82	110	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	163	15.0
22	4247.36	120	Silty Sand	SM	196	SPT	82	36	2552	2427.2	1179	0.95	1.37	1.14	1.00	1.00	53	15.0
24	4245.36	120	Silty Sand	SM	196	SPT	82	18	2784	2534.4	1286	0.95	1.37	1.14	1.00	1.00	27	15.0
10	4262	120	Silty Sand	SM	196	SPT	82	25	1160	1160	536	0.80	1.37	1.14	1.00	1.00	31	15.0
12	4260	120	Silty Sand	SM	196	SPT	82	38	1392	1392	643	0.80	1.37	1.14	1.00	1.00	47	15.0
14	4258	120	Silty Sand	SM	196	SPT	82	125	1624	1624	750	0.85	1.37	1.14	1.00	1.00	166	15.0
16	4256	120	Silty Sand	SM	196	SPT	82	51	1856	1856	858	0.85	1.37	1.14	1.00	1.00	68	15.0
18	4254	120	Silty Sand	SM	196	SPT	82	38	2088	2088	965	0.85	1.37	1.14	1.00	1.00	50	15.0
22	4250	120	Silty Sand	SM	196	SPT	82	106	2552	2427.2	1179	0.95	1.37	1.14	1.00	1.00	157	15.0
24	4248	120	Silty Sand	SM	196	SPT	82	72	2784	2534.4	1286	0.95	1.37	1.14	1.00	1.00	107	15.0
26	4246	120	Silty Sand	SM	196	SPT	82	17	3016	2641.6	1394	0.95	1.37	1.14	1.00	1.00	25	15.0
8	4260	120	Silty Sand	SM	196	SPT	82	27	928	928	429	0.75	1.37	1.14	1.00	1.00	32	15.0
10	4258	120	Silty Sand	SM	196	SPT	82	25	1160	1160	536	0.80	1.37	1.14	1.00	1.00	31	15.0
12	4256	120	Silty Sand	SM	196	SPT	82	29	1392	1392	643	0.80	1.37	1.14	1.00	1.00	36	15.0
14	4254	120	Silty Sand	SM	196	SPT	82	22	1624	1624	750	0.85	1.37	1.14	1.00	1.00	29	15.0
16	4252	120	Silty Sand	SM	196	SPT	82	30	1856	1856	858	0.85	1.37	1.14	1.00	1.00	40	15.0
18	4250	120	Silty Sand	SM	196	SPT	82	13	2088	2088	965	0.85	1.37	1.14	1.00	1.00	17	15.0
20	4248	120	Silty Sand	SM	196	SPT	82	19	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	28	15.0
8	4260	120	Silty Sand	SM	196	SPT	82	21	928	928	429	0.75	1.37	1.14	1.00	1.00	25	15.0
10	4258	120	Silty Sand	SM	196	SPT	82	63	1160	1160	536	0.80	1.37	1.14	1.00	1.00	79	15.0
12	4256	120	Silty Sand	SM	196	SPT	82	31	1392	1392	643	0.80	1.37	1.14	1.00	1.00	39	15.0
14	4254	120	Silty Sand	SM	196	SPT	82	20	1624	1624	750	0.85	1.37	1.14	1.00	1.00	27	15.0
16	4252	120	Silty Sand	SM	196	SPT	82	25	1856	1856	858	0.85	1.37	1.14	1.00	1.00	33	15.0
18	4250	120	Silty Sand	SM	196	SPT	82	29	2088	2088	965	0.85	1.37	1.14	1.00	1.00	38	15.0
20	4248	120	Silty Sand	SM	196	SPT	82	21	2320	2320	1072	0.95	1.37	1.14	1.00	1.00	31	15.0
22	4246	120	Silty Sand	SM	196	SPT	82	18	2552	2427.2	1179	0.95	1.37	1.14	1.00	1.00	27	15.0

Notes:

Evaluation reflects SPT-blow counts from borings GW-17A, -18, 19-A, -19B, -25, -26, -27, -28, -36, -37, -38 (Bingham Environmental, 1992) for Unti 3 sand-like soils

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"

[2] Boring location known to exist somewhere in Section 32 of the Clive Facility

[3] a_{max} and earthquake magnitude based on parameters presented in the seismis hazard analysis by AMEC 2012

[4] Magnitude scaling factor, (6.9 e^-Magnitude/4)-0.058, up to 1.8.

[5] Estimated to result in C_{energy} of 0.8 assuming Autohammer

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. CSPT is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] $m=0.784-0.0768\text{sqrt}((N_1)_{60cs})\geq 0.264$ is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] $C_N=(P_a/\sigma'_v)^m \leq 1.7$ accounts for effective overburden stress (Equation 33, I&B 2008)

Boring: Multiple (continued from previous page)
Date: -
By: Overland Drilling

Fines Content Method	[11] m	[12] C_N	$(N_1)_{60}$ [13] (blows/ft)	$\Delta(N_1)_{60}$ [14]	$(N_1)_{60cs}$ [15] (blows/ft)	[16] α	[17] β	[18] r_d	[19] C_σ	[20] K_σ	[21] $CRR_{M7.5,\sigma'vc}$	[22] $CSR_{M7.5,\sigma'vc}$	[25] $\Delta(N_1)_{60}$-FC%	$(N_1)_{60CS-Sr}$	FS
Est	0.264	1.17	79	3.3	82	-0.14	0.02	0.98	0.300	1.100	50.00	0.285	1	80	2.00
Est	0.358	1.16	28	3.3	31	-0.17	0.02	0.97	0.211	1.100	0.54	0.283	1	29	1.91
Est	0.357	1.10	28	3.3	31	-0.22	0.02	0.96	0.212	1.100	0.55	0.281	1	29	1.97
Est	0.264	1.04	44	3.3	47	-0.26	0.03	0.96	0.300	1.100	101.20	0.278	1	45	2.00
Est	0.355	1.00	28	3.3	31	-0.30	0.03	0.95	0.215	1.100	0.58	0.276	1	29	2.00
Est	0.438	0.96	17	3.3	20	-0.35	0.04	0.94	0.135	1.092	0.21	0.276	1	18	0.76
Est	0.264	0.96	84	3.3	88	-0.40	0.04	0.93	0.300	1.100	50.00	0.271	1	85	2.00
Est	0.437	0.96	17	3.3	20	-0.34	0.04	0.94	0.136	1.094	0.21	0.276	1	18	0.77
Est	0.324	0.96	33	3.3	36	-0.39	0.04	0.93	0.277	1.100	1.36	0.272	1	34	2.00
Est	0.366	0.94	26	3.3	30	-0.44	0.05	0.92	0.200	1.100	0.47	0.269	1	27	1.73
Est	0.397	1.27	22	3.3	25	-0.14	0.02	0.98	0.166	1.100	0.30	0.285	1	23	1.06
Est	0.264	1.05	50	3.3	54	-0.24	0.03	0.96	0.300	1.100	50.00	0.280	1	51	2.00
Est	0.370	0.97	26	3.3	29	-0.35	0.04	0.94	0.194	1.100	0.43	0.274	1	27	1.58
Est	0.264	0.95	53	3.3	57	-0.47	0.05	0.92	0.300	1.100	50.00	0.267	1	54	2.00
Est	0.264	1.24	47	3.3	50	-0.10	0.01	0.98	0.300	1.100	537.05	0.287	1	48	2.00
Est	0.264	1.17	83	3.3	87	-0.14	0.02	0.98	0.300	1.100	50.00	0.285	1	84	2.00
Est	0.275	1.12	41	3.3	44	-0.17	0.02	0.97	0.300	1.100	18.51	0.283	1	42	2.00
Est	0.347	1.05	29	3.3	32	-0.26	0.03	0.96	0.228	1.100	0.69	0.278	1	30	2.00
Est	0.346	1.00	29	3.3	33	-0.30	0.03	0.95	0.230	1.100	0.71	0.276	1	30	2.00
Est	0.343	0.95	30	3.3	33	-0.40	0.04	0.93	0.235	1.100	0.75	0.271	1	31	2.00
Est	0.346	0.94	29	3.3	33	-0.45	0.05	0.92	0.229	1.100	0.70	0.269	1	30	2.00
Est	0.264	1.12	46	3.3	49	-0.17	0.02	0.97	0.300	1.100	369.99	0.283	1	47	2.00
Est	0.264	1.07	55	3.3	59	-0.22	0.02	0.96	0.300	1.100	50.00	0.281	1	56	2.00
Est	0.264	1.04	70	3.3	73	-0.26	0.03	0.96	0.300	1.100	50.00	0.278	1	71	2.00
Est	0.435	1.01	17	3.3	21	-0.30	0.03	0.95	0.137	1.100	0.21	0.276	1	18	0.77
Est	0.340	0.97	30	3.3	33	-0.35	0.04	0.94	0.241	1.100	0.82	0.274	1	31	2.00
Est	0.264	0.95	131	3.3	135	-0.45	0.05	0.92	0.300	1.100	50.00	0.269	1	132	2.00
Est	0.268	0.94	42	3.3	45	-0.50	0.06	0.91	0.300	1.100	33.96	0.266	1	43	2.00
Est	0.264	1.07	131	3.3	134	-0.22	0.02	0.96	0.300	1.100	50.00	0.281	1	132	2.00
Est	0.385	1.05	24	3.3	27	-0.26	0.03	0.96	0.177	1.100	0.35	0.278	1	25	1.24
Est	0.264	0.98	159	3.3	162	-0.35	0.04	0.94	0.300	1.100	50.00	0.274	1	160	2.00
Est	0.264	0.96	51	3.3	55	-0.40	0.04	0.93	0.300	1.100	50.00	0.271	1	52	2.00
Est	0.376	0.93	25	3.3	28	-0.45	0.05	0.92	0.187	1.093	0.39	0.271	1	26	1.45
Est	0.295	1.19	37	3.3	41	-0.14	0.02	0.98	0.300	1.100	4.94	0.285	1	38	2.00
Est	0.264	1.12	53	3.3	56	-0.17	0.02	0.97	0.300	1.100	50.00	0.283	1	54	2.00
Est	0.264	1.07	178	3.3	181	-0.22	0.02	0.96	0.300	1.100	50.00	0.281	1	179	2.00
Est	0.264	1.04	70	3.3	73	-0.26	0.03	0.96	0.300	1.100	50.00	0.278	1	71	2.00
Est	0.264	1.00	51	3.3	54	-0.30	0.03	0.95	0.300	1.100	50.00	0.276	1	52	2.00
Est	0.264	0.96	152	3.3	155	-0.40	0.04	0.93	0.300	1.100	50.00	0.271	1	153	2.00
Est	0.264	0.95	102	3.3	105	-0.45	0.05	0.92	0.300	1.100	50.00	0.269	1	103	2.00
Est	0.390	0.92	23	3.3	26	-0.50	0.06	0.91	0.172	1.072	0.33	0.273	1	24	1.20
Est	0.280	1.26	40	3.3	43	-0.10	0.01	0.98	0.300	1.100	12.94	0.287	1	41	2.00
Est	0.295	1.19	37	3.3	41	-0.14	0.02	0.98	0.300	1.100	4.94	0.285	1	38	2.00
Est	0.275	1.12	41	3.3	44	-0.17	0.02	0.97	0.300	1.100	18.51	0.283	1	42	2.00
Est	0.329	1.09	32	3.3	35	-0.22	0.02	0.96	0.264	1.100	1.13	0.281	1	33	2.00
Est	0.272	1.04	41	3.3	45	-0.26	0.03	0.96	0.300	1.100	24.52	0.278	1	42	2.00
Est	0.435	1.01	17	3.3	21	-0.30	0.03	0.95	0.137	1.100	0.21	0.276	1	18	0.77
Est	0.360	0.97	27	3.3	31	-0.35	0.04	0.94	0.208	1.100	0.52	0.274	1	28	1.90
Est	0.327	1.31	32	3.3	35	-0.10	0.01	0.98	0.269	1.100	1.22	0.287	1	33	2.00
Est	0.264	1.17	92	3.3	95	-0.14	0.02	0.98	0.300	1.100	50.00	0.285	1	93	2.00
Est	0.264	1.12	43	3.3	46	-0.17	0.02	0.97	0.300	1.100	67.57	0.283	1	44	2.00
Est	0.347	1.10	29	3.3	32	-0.22	0.02	0.96	0.227	1.100	0.68	0.281	1	30	2.00
Est	0.312	1.04	35	3.3	38	-0.26	0.03	0.96	0.300	1.100	2.16	0.278	1	36	2.00
Est	0.287	1.00	39	3.3	42	-0.30	0.03	0.95	0.300	1.100	8.03	0.276	1	40	2.00
Est	0.340	0.97	30	3.3	33	-0.35	0.04	0.94	0.241	1.100	0.82	0.274	1	31	2.00
Est	0.373	0.95	25	3.3	29	-0.40	0.04	0.93	0.190	1.100	0.41	0.271	1	26	1.51

[13] $(N_1)_{60}=N_{60}*C_N$ is the overburden corrected penetration resistance (Equation 31, I&B 2008)
[14] $\Delta(N_1)_{60}=\exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2]$ represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)
[15] $(N_1)_{60cs}=(N_1)_{60} + \Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)
[16] $\alpha(z) = -1.012-1.126\sin((z/11.73)+5.133)$ in which z is depth in meters (Equation 23, I&B 2008)
[17] $\beta(z) = 0.106+0.118\sin((z/11.28)+5.142)$ in which z is depth in meters (Equation 24, I&B 2008)
[18] $r_d=\exp[\alpha(z)+\beta(z)M]$ is shear stress reduction coefficient (Equation 22, I&B 2008)
[19] $C_\sigma=1/(18.9-2.55\text{sqrt}[(N_1)_{60cs}]\leq 0.3$ is the coefficient for K_σ (Equation 56, I&B 2008)
[20] $K_\sigma = 1-C_\sigma\ln(\sigma_{vo}'/P_a)\leq 1.1$ is the overburden correction factor (Equation 54, I&B 2008)
[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)
[22] $CSR_{M7.5,\sigma'vc}=0.65(a_{max}/g)(\sigma_v/\sigma_v')r_d(1/MSF)(1/K_\sigma)$ is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)
[23] NL = non-liquefiable; L = potentially liquefiable
[24] Groundwater assumed to be at a depth of 20 feet below ground surface during the field investigation (for blow count correction)
[25] Fines content correction for liquefied shear strength from Seed 1987 (Table 4, pg 126, I&B 2008)
[26] MOD-CAL refers to 2.5-inch ID sampler
[27] $\gamma_{lim} = 1.859[1.1 - \text{sqrt}((N_1)_{60cs}/46)]^3 > 0$ but less than 50% = limiting shear strain (Equation 86, I&B, 2008)
[28] $F\alpha = 0.032 + 0.69\text{sqrt}[(N_1)_{60cs}] - 0.13(N_1)_{60cs}$, where $(N_1)_{60cs}$ is limited to values > 7 (Equation 93, I&B, 2008)
[29] $\gamma_{max} = \min[\gamma_{lim}, 0.35(2-FS)((1-F\alpha)/(FS-F\alpha)]$ for 2 > FS > Fα; if FS < Fα, $\gamma_{max} = \gamma_{lim}$ (Equations 91 & 92, I&B, 2008)
[30] ΔHi = Layer thickness (ft)
[31] $\varepsilon_v = 1.5\exp(-0.369\text{sqrt}[(N_1)_{60cs}]$ x $[\min(0.08, \gamma_{max})]$ = post liquefaction volumetric strain (Equation 96, I&B, 2008)
[32] ΔSi = (Δhi)(εv)