April 6, 2012
Job No 10-817-05290

Mr. Sean McCandless
Compliance and Permitting Manager
Energy*Solutions,* Inc.
423 West 300 South, Suite 200
Salt Lake City, Utah 84101

Re: Response to Interrogatory CAW R313-25-8(4)-16/3:
Seismic Hazard Evaluation / Seismic Stability Analysis Update,
Energy*Solutions* Clive Facility
Class A West Embankment
Clive, Tooele County, Utah

Dear Mr. McCandless:

AMEC previously performed geotechnical investigations at the site and summarized the results in a report dated December 13, 2005 (AMEC Job No. 4-817-004769). Changes in the waste embankment configuration required re-evaluation of the findings and conclusions presented in the 2005 report; therefore, we issued an updated report dated February 15, 2011 (AMEC Job No. 10-817-05290). These reports were subsequently submitted to the State of Utah for their review and comments.

In a letter dated October 25, 2011, we provided our responses to comments received from the State of Utah in their Round 1 Interrogatory CAW R313-25-8(4)-16/1. The October 25th letter specifically addressed review comments regarding procedures used in our prior seismic hazard evaluation, the peak ground acceleration (PGA) value used in seismic slope stability analyses of the embankment, and the results summarized in our February 2011 report.

The Round 2 interrogatory comments were presented in their interrogatory CAW R313-25-8(4)-16/2 which was received from the State of Utah on December 8, 2011. We provided our response to the comments in a letter dated December 23, 2011. The December 23rd letter specifically addressed review comments to justify using a “semi-probabilistic” approach to seismic hazard evaluation and incorporated results from a probabilistic seismic hazard analysis (PSHA) using EZ-Frisk for the background seismicity. The Round 3 interrogatory comments were received from the State of Utah on January 30, 2012 (a copy is included in Appendix A). The interrogatory comments that are specifically addressed within this letter are presented in interrogatory CAW R313-25-8(4)-16/3.

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The professional opinions presented in this letter have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this letter.

Interrogatory CAW R313-25-8(4)-16/3 - Review Comments (abbrv., full text in Appendix A)

Seismic Hazard Analysis

- Verify/update PGA's for deterministic fault sources identified in Table 1 of the October 25th letter using a Deterministic Seismic Hazard Analysis (DSHA) with full weight of the Next Generation Attenuation (NGA) relationships.
- Perform a site-specific PSHA to assess PGA associated with background seismicity. Prepare the PSHA with removing foreshocks and aftershocks from the historical earthquake record.
- The design PGA is to be selected as the maximum PGA from consideration of the 84th percentile estimates from deterministic sources and the mean value for probabilistic PGA from background seismicity.

Soil Liquefaction/Cyclic Softening

- Demonstrate that the potential effects of soil liquefaction and/or cyclic softening phenomena in native soils at the Clive Facility have been adequately accounted in the geotechnical analysis supporting the design of the proposed CAW Embankment.
- Clearly justify the selection of soil parameters and design assumptions

Response to Interrogatory *CAW R313-25-8(4)-16/3* – Seismic Hazard Evaluation

In this response, AMEC presents an updated assessment of the seismic hazard for the site consistent with the requirements of the Utah Code of Regulations R313-25-8(5) and the information requested in Interrogatory Round 3. The seismic hazard assessment is based on an assessment of the peak ground acceleration (PGA) associated with the Maximum Credible Earthquake (MCE) for known active or potentially active faults in the site region, and the PGA obtained from a probabilistic seismic hazard analysis (PSHA) to assess the seismic hazard for earthquakes that may occur on unknown faults in the area surrounding the project site (i.e., background seismicity). For fault sources, the PGA is calculated at the 84th percentile level and is based on the maximum rupture length and rupture area for each fault. The return period for ground motions resulting from a background earthquake is identified as 5000 years (equal to a one percent probability of exceedance in 50 years). The approach to select a MCE PGA from the larger of the values associated with the deterministic MCE for faults or the PSHA result for background earthquakes at a 5000 year return period is consistent with the discussions among AMEC, ES, Utah DEQ and their peer reviewer, URS Corporation, and is consistent with the recommendations of the Utah Seismic Safety Commission (2003) and as required by the Utah Division of Water Rights (Dam Safety Section) for assessment of dams.

The deterministic assessment follows the approach described in our October 25, 2011 letter, and is updated in the following paragraphs. Potential fault sources are shown on Figure B-1.1 and are listed in Table B-1.1 of Appendix B, including an assessment of the fault parameters, source to site distance, and PGA. Specific fault parameters and other information in Table B-1.1 include fault name, slip type, maximum magnitude, location of site on hanging wall or footwall, fault dip, rake, maximum rupture length (fault length), downdip rupture width, distance measures required for ground motion attenuation relationships, and PGA for median and 84th percentile levels. We use a suite of four Next Generation Attenuation (NGA) relationships, namely those of Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008), all of which are applicable for the site conditions and types of sources in Utah and the Intermountain Region. Additional parameters for attenuation relationships include site shear wave velocity, V_{S30}, taken as 305 m/s as described in the October 25 Letter, and depth to top of bedrock ($Z_{1.0}$ and $Z_{2.5}$), taken as default values calculated from the site V_{S30} as recommended by the authors of the NGA relationships (also as described in the October 25 Letter).

The maximum magnitude for each fault is based on rupture of the full length of the fault, and where available is taken as the maximum value published by the Utah Working Group on Earthquake Probabilities (WGUEP, 2011), except for the Stansbury fault as noted below. For faults not assessed in the previous studies, including the Skull Valley fault, the maximum magnitude was assessed using the same methodology as the WGUEP study, based on maximum rupture length, rupture width, and the empirical relationships of Wells and Coppersmith (1994). For short faults where the calculated maximum magnitude is less than M_W 6.5, a maximum magnitude of 6.5 is adopted because this is judged to be a reasonable minimum value of magnitude for earthquakes that rupture to the ground surface.

For the Stansbury fault, the maximum magnitude is assessed as M_W 7.3 based on consideration of the maximum rupture length, fault width, and maximum fault displacement identified in previous investigations (Geomatrix Consultants, 1999; WGUEP, 2011). The value of M_W 7.5 listed in the October 25 Letter and by the WGUEP is judged to be too conservative because it is higher than the maximum value obtained from empirical relationships, considering all combinations of rupture length, rupture width, and maximum fault displacement cited in those previous investigations. We note that it may be reasonable to consider an extreme value with a very low weighting (e.g., less than 10 percent) in a probabilistic analysis, but that it is not reasonable practice to adopt an extreme value for the MCE for a deterministic analysis.

The maximum of the 84th percentile PGA values calculated for the M_{max} events on the fault sources is equal to 0.24 g, as obtained for the Stansbury and the Skull Valley faults (Table B-1.1).

For the PSHA, we used the current version (Ver. 7.62) of commercial program EZ-FRISK to calculate the PGA for the background earthquake. The program developer, Risk Engineering, has prepared input fault and background seismicity files for Utah for use in calculating seismic hazard; these files are based on the same fault source parameters and independent seismicity catalog used by the U.S. Geological Survey (USGS) to prepare the 2008 National Seismic Hazard Maps.

The seismicity catalog is an independent (de-clustered) catalog based on moment magnitude (M_W) that covers the Western United States; the seismicity in the vicinity of the project site is shown on Figure B-1.1. The recurrence rates for the background seismicity are based on the same recurrence models and maximum magnitudes used by USGS, which is a spatially smoothed gridded approach, with a maximum magnitude of 7.0 for Utah (Peterson et al., 2008). As for the deterministic analysis, we use the same suite of four NGA relationships and the site V_{S30} of 305 m/s. The PGA is taken as the weighted average of the mean values for the four NGA relationships at a return period of 5000 years (equal to 0.24 g, Table B-1.1).

The largest PGA from the deterministic assessment of fault-specific sources and the probabilistic assessment of the background earthquake is 0.24 g. The maximum magnitude varies from 7.0 to 7.3 for the sources that result in the maximum PGA; we identify the largest value, M_W 7.3, as appropriate for use in the seismic stability analyses for this project.

Response to Interrogatory CAW R313-25-8(4)-16/3 – Soil Liquefaction/Cyclic Softening

The following section of this letter presents responses to the comments received from the State of Utah in regards to soil liquefaction/cyclic softening. Specifically, this section addresses several review comments regarding the presentation and methodologies used in our previous liquefaction hazard analyses, including the consideration of the differences between liquefaction of “sand-like” soil and cyclic-softening of “clay-like” soil. Our responses have incorporated the results of the seismic hazard evaluation presented above. These sections describe the procedures used to perform these evaluations and the means for selecting soil parameters and design assumption.

The subsurface soil profile below the CAW Embankment has been described previously in detail in Table 2.2 of AMEC (2005) and subsequently restated in Table 2.1 of AMEC (2011a). The soil liquefaction/cyclic softening evaluations for this response utilized data from six cone-penetrometer soundings (CPT’s) that were obtained in 1999 and 2004. They are individually identified as CPT99-01, CPT99-02, CPT99-03, CPT04-04, CPT04-05, and CPT04-06. Previous Borings B-1 and B-2 from AMEC 2005a were extended to depths of 94.5 and 100 feet, respectively. Relatively undisturbed samples were obtained from these exploratory excavations using a Dames & Moore type tube sampler. Standard Penetration tests (SPT’s) were not performed at these borings. The design depth for groundwater at the site is at a depth of not less than 22 to 25 feet. For analysis purposes the groundwater is placed at 22 feet below the ground surface.

Similarly to past conclusions the potential for liquefaction of sand-like soil has been determined to be low and the potential for seismic settlement to be on the order of one to two inches. The potential for cyclic-softening was also found to be low.

<u>Soil Liquefaction Evaluation “Sand-like” Soil</u>

Liquefaction is a soil behavior phenomenon in which saturated, medium dense to loose, sand-type soils experience losses of shear strength and stiffness associated with accumulation of excess pore-pressures that develops during seismic shaking. Liquefaction potential was evaluated using

the analysis methodology described in Youd, et al. (2001). More recent publications on liquefaction analyses such as Seed, et al. (2003) and Boulanger and Idriss (2008) were not used in our anlayses because there is not yet consensus among geotechnical/earthquake practitioners regarding the reliability of these methods. In addition to a lack of consensus, Boulanger and Idriss (2008) which is gaining support among practitioners in California, is considered by many to be unconservative for critical facilties. As such, a more widely accepted and conservative method was used to assess liquefaction potential at the site. As described in the previous section the site-specific ground shaking is based on a design PGA of 0.24 g and a Magnitude of 7.3. We use a PGA of 0.28 g and magnitude of 7.3 for our analyses; the larger PGA value was selected in 2005 and has served as the basis for design since that time. Based upon averaged laboratory densities from borings B-1 and B-2, wet and dry densities were calculated to be approximately 121 pcf and 97 pcf, respectively.

Figures illustrating the comparison of penetration resistances versus depth or elevation measured in the six CPTs to the critical penetration resistances for design ground motion hazard levels have been used to depict zones within the subsurface profile that are potentially liquefiable soils. The figures help understand where liquefaction hazards may exist and how severe they may be. The plots of factor of safety against liquefaction are shown on Figures C-1.1 through C-1.6

The liquefaction potential was considered for each of the four geologic units listed in Table 2.1 of AMEC (2011a). Unit 4 and the majority of Unit 3 is in an unsaturated state and can be considered non-liquefiable due to lack of saturation because the water table depth is 22 feet below site grade. The remainder of Unit 3 from 22 to 28 feet below grade is typically too dense to liquefy based on data from five CPTs. Values of $q_{c1n(cs)}$ for 5 of the 6 CPTs in Unit 3 ranged from 105 to 546 with an average of 278. Corrected tip resistance exceeding 160 is considered non-liquefiable based on graphs of Cyclic Resistance Ratio (CRR) versus Corrected CPT Tip Resistance in Robertson and Wride (1998). Also, the factor of safety for Unit 3 was calculated to be above 1 in all CPTs except CPT04-05. In CPT04-05, however, a liquefiable layer from 25.3 to 26.9 feet was encountered. This layer had an average corrected tip resistance of 65 and a corresponding average factor of safety of 0.61.

Unit 2 is primarily clay-like in nature consisting of clay and silt with seams of sand. Sand seams are generally less than 1 foot, but range up to 3 feet. These sand layers are generally dense and have corrected CPT tip resistances greater than 160 with corresponding factors of safety greater than 1. Liquefaction analyses of CPT data within Unit 2 indicate cummulative liquefiable thickness for the individual CPT locations ranging from 2 to 14 inches with no individual liquefiable layer exceeding 6 inches. Several of the individual layers are confined by soft clays. Soft clays below the sand layers may be reducing CPT tip resistances, even after the thin layer correction is applied. Based upon low liquefaction potential due to thin liquefiable layers and the predominantly clay-like soils within this unit, a cyclic shear softening analysis is applied to Unit 2, in lieu of a liquefaction analysis appropirate for sand-like soils.

Unit 1 is characterized as interbedded sand, silt and clay. Generally, soil intervals demonstrating sand-like behavior within this unit are dense with a range of data that has an average corrected tip resistance of 197 and a median corrected tip resistance of 186.4. The data indicates that approximately 98 percent of soils in this unit exibiting sand-like characteristics have a factor of safety greater than 1. Liquefaction analyses using Youd et al. (2001) indicate that CPT's 99-02, 04-04, and 04-05 have no liquefiable materials in Unit 1. CPT99-03 has two 2-inch-thick liquefiable layers and one 4-inch-thick layer at 80.1, 93.5 and 100.1 feet below site grade. CPT99-01 has one 4-inch layer at 101.4 feet below grade, whereas CPT04-06 has one 2-inch layer at 79.1 feet below grade, and one 10-inch layer of liquefiable material at 79.7 feet below grade. Boring B-2 located adjacent to CPT04-05 indicated samples collected below 85 feet were either unsaturated, had estimated $N_{1(60)}$ blow counts above 30, or were cohesive. These soils were classified as not susceptible to liquefaction-type behavior. Penetration tests at 90 feet had corrected blow counts consistent with liquefiable materials. The sample at 90 feet consisted of medium dense silty sand. CPT04-05, however, indicated soils at this elevation behaved as a sand, but were too dense to liquefy. At 98 feet, sample recovery indicated the soil was a hard silty clay with sand and gravel.

Volumetric Strain in "Sand-like" Soil

Resulting liquefiable layers in Units 3 and 1 were then analyzed for post-liquefaction volumetric strain using Ishihara and Yoshimine (1992). The resulting estimated settlements due to post-liquefaction volumetric strain are in the range of 0 to ¾ inch.

By another method, CPT tip resistances were converted to equivalent N_{60} values using a relationship developed by Jefferies and Davies (1993). The resulting N_{60} values were corrected for overburden pressure and then used to develop post-liquefaction volumetric strain based on relationships by Tokimatsu and Seed (1987). The resulting post-liquefaction volumetric strain estimates are also in the range of 0 to ¾ inch for Units 3 and 1. A summary of resulting settlements at various CPTs are presented below.

Liquefaction Settlement Ishihara and Yoshimine (1992)						
Geologic Units	CPT99-01	CPT99-02	CPT99-03	CPT04-04	CPT04-05	CPT04-06
3	0	0	0	0	0.68	0
1	0.2	0	0.23	0	0	0.32
Liquefaction Settlement Tokimatsu and Seed (1987)						
Geologic Units	CPT99-01	CPT99-02	CPT99-03	CPT04-04	CPT04-05	CPT04-06
3	0.05	0.17	0	0.08	0.65	0
1	0.18	0.03	0.37	0	0.01	0.47

Lateral Spread Evaluation

Earthquake induced lateral spreads occur on sloping ground or at sites with free-face conditions such a ditch or river bank. The Clive Facility is located on the ancient Bonneville lakebed, which is likely too flat to generate lateral spread. Additionally, the site of the waste embankment is not adjacent to known free face conditions that might contribute to lateral movement during a liquefaction event. The topographic site characteristics are not consistent with known lateral spread events.

The density profile of sand-like soils is laterally and vertically variable, but is generally dense. Thin liquefiable layers are encountered in the soil profile, but they tend to be discontinuous and are generally not encountered in adjacent CPTs or borings. The lack of a continuous liquefiable layer beneath the site reduces the likelihood of lateral spread due to the resistance of non-liquefiable material between discontinuous layers, Youd et al. (2009).

Due to site topography, thin discontinuous liquefiable layers, and a generally dense profile with significant density variability across short distances and at variable depths, it is our opinion that the likelihood of liquefaction-induced lateral spread occuring at the site is very low.

Cyclic Softening of "Clay-like" Soil

In addition to the liquefaction evaluation of the "sand-like" soils as presented in the previous section of this report, we have evaluated the potential for cyclic softening in "clay-like" soils underlying the site. The potential for cyclic softening was evaluated using the procedures published by Boulanger and Idriss (2004) and Boulanger and Idriss (2007).

Based on the results of the explorations at the site and knowledge of the site geology, an idealized geologic cross-section was developed indicating the different geologic units and their predominant soil type using the Unified Soil Classification Systems (USCS). The geologic cross-sections were presented in Figures 5A and 5B of AMEC (2011a). The cross-sections show that geologic Units 2 and 4 consist of predominately clayey soils; geologic Unit 3 consists of sandy soils and Unit 1 consists of interbedded sand, silt and clay layers.

The results of over 30 Atterberg limits tests conducted on samples in Units 2 and 4 indicate that the plasticity index of these soils vary from 7 to 20 percent. Based on these results, the soils are considered to exhibit "clay-like" behavior and are not anticipated to be susceptible to liquefaction. Therefore, an evaluation of cyclic softening was performed in Units 2 and 4, which could be subject to cyclic softening and associated loss of shear strength and vertical strains during ground shaking.

To conduct an evaluation of cyclic softening, undrained shear strength (S_u) and overconsolidation ratio (OCR) or preconsolidation stress (σ_p') are required. The undrained shear strength can be obtained from field vane shear tests or using empirical correlation of cone penetration test (CPT) tip resistance with s_u or using the SHANSEP approach (Ladd and Foott, 1974). Vane shear testing was not performed as part of the geotechnical investigations performed at the site. Therefore,

SHANSEP model along with consolidation test results for OCR were used for evaluation of cyclic softening. In addition, the potential for cyclic softening was checked using the CPT data. Both of these evaluation procedures are described in the following sections.

(1) <u>SHANSEP Model</u>
$S_u / \sigma_p' = m\,[OCR]^n$, where m = 0.26 and n = 0.79 (based on lab testing of Lake Bonneville clays conducted for I-15 improvements project)

(2) <u>CPT Correlation with S_u</u>
$S_u = (Q_t - \sigma_v) / N_k$, where Q_t = cone tip resistance, σ_v = total stress and N_k = varies from 10 to 20 (average of 15)

For the SHANSEP model, the σ_p' and OCR were estimated from laboratory consolidation tests on high quality samples of the clayey soils. An OCR of 5.3 for Unit 4 and 1.2 for Unit 2 were used in the evaluation as they were previously used for other settlement calculations as reported in AMEC (2011a). Using these parameters, the undrained shear strength was estimated in Units 2 and 4 for every 2-foot depth increment. It is noted that the S_u estimated thus far is based on site conditions that existed at the time of sampling. Since embankments with average height of about 60 feet will be constructed, it is expected that the underlying soils (including Units 2 and 4) will be subject to additional stresses from embankment loading and will undergo settlement over a period of time. As the consolidation of the clay units continue over a period of time, the effective stresses within these units come into equilibrium with the applied embankment stresses. We anticipate that the stress equilibrium will occur after a period of about 2 years or so (assuming 90% consolidation of Units 2 and 4). The OCR at the end of this period is anticipated to be around 1.0. The post-construction S_u is then estimated using the OCR, appropriate stresses under embankment loading and SHANSEP model. If desired, additional borings may be performed at the end of 2 year period to evaluate the OCR of the soils.

Another important parameter that governs the factor of safety against cyclic softening is the presence of static shear stresses (τ_s) within the units such as under the embankment loading at the site. The pre-existence of static shear stresses usually tends to reduce the cyclic resistance of the clays. The average static shear stresses normalized to the undrained shear strength (τ_s/S_u) can be obtained from the static slope stability analysis using the undrained shear strength properties of the soils. Based on such analysis, a static factor of safety of 2.64 for a failure surface through Unit 2 and a static factor of safety of 4.19 for a failure surface through Unit 4 were computed. The results of these slope stability analyses are included in Appendix C as Figures C-1.7 and C-1.8.

For evaluating the seismic factor of safety against cyclic softening, the cyclic stress ratio (CSR), cyclic resistance (CRR), magnitude scaling factor (MSF) and stress reduction factor (r_d) were computed using equations published in Boulanger and Idriss (2004) and Boulanger and Idriss (2007). The results of the evaluation of cyclic softening using the SHANSEP model are presented in Tables C-1.1 and C-1.2, which are included in Appendix C. The results in Table C-1.1 are based on a design earthquake magnitude of 7.3 and PGA of 0.24 g and those in Table C-1.2 are

based on the same design earthquake magnitude and a slightly higher PGA of 0.28 g which was used in all prior seismic calculations.

Based on the results presented in Table C-1.1 which were computed for the design earthquake event (M_w = 7.3 and PGA = 0.24 g), the factors of safety against cyclic softening within Units 2 and 4 are greater than 1.0 and therefore cyclic softening of the clay-like soils at the site is considered to be negligible. However, using a PGA of 0.28 g, the factor of safety within Unit 2 is greater than 1.0, but the factor of safety within shallower Unit 4 is slightly lower than 1.0 (about 0.92). Since Unit 4 is above current and design groundwater level and the assumed post-construction OCR of 1.0 is probably conservative for this shallower unit, it is our opinion that the potential for cyclic softening within Unit 4 is overestimated and evaluation based on a higher PGA of 0.28 g adds unnecessary conservatism.

Cyclic softening was also evaluated using the data from six CPTs performed at the site. Undrained shear strength (S_u) was computed using the CPT tip resistance and the equations presented on the previous page with an N_k of 15. Accounting for the interlayered nature of the clay and silt sequences within Units 2 and 4, the average tip resistance and the computed S_u within each Unit are presented in the table below.

CPT No.	Tip Resistance, Q_c (ksf)	Average Undrained shear strength, S_u (psf)
CPT99-01	60 – 70	3,630 – 4,545
CPT99-02	30 – 50	1,995 – 2,910
CPT99-03	38 – 50	2,500 – 3,160
CPT04-04	28 – 55	1,840 – 3,330
CPT04-05	33 – 80	2,165 – 4,960
CPT04-06	30 – 60	1,950 – 3,700

Comparing the estimated S_u values based on the CPT data with those computed from SHANSEP model and laboratory consolidation testing, it is evident that the S_u computed using the CPT data are much greater and would only result in a higher factor of safety than those evaluated using the SHANSEP model.

It is also noted that the S_u values presented in the above table are representative of strengths at the time of investigation. As the soils consolidate under embankment loading over the period of time, the tip resistance as well as the shear strength of the underlying soils will improve with time. Therefore, the actual shear strength of the soils at the end of consolidation period (assumed to be

about 2 years) will be higher than those presented in the table resulting in a much higher factor of safety against cyclic softening.

Discussion and Conclusions

Based on the deterministic seismic hazard assessment (DSHA) results presented in our October 25, 2011, report, and described above in the Seismic Hazard Evaluation section, the maximum 84th-percentile PGA from fault-specific earthquake sources is computed to be 0.24 g for a maximum magnitude earthquake occurring on the Stansbury fault (M 7.3) or the Skull Valley fault (M 7.1). Our probabilistic analysis of the background seismicity shows the mean PGA for a return period of 5000 years is 0.24 g for non-fault-specific earthquakes up to M 7. Therefore, the largest PGA computed for the Energy*Solutions* site is 0.24 g based on deterministic procedures and using NGA models for fault-specific sources and on probabilistic procedures for the background earthquake with a recurrence interval of 5,000 years.

Liquefaction and cyclic softening evaluations, as well as previous embankment stability calculations, were based on a PGA of 0.28 g, which is higher than the updated maximum 84th percentile PGA calculated with deterministic procedures in the present study; it is also higher than the median PGA value for background earthquakes with 5000-year recurrence intervals calculated with PSHA procedures. Therefore, we are of the opinion that the results of seismic stability calculations of the embankments at the Energy*Solutions* site presented in our report dated February 15, 2011 (AMEC 2011a) remain appropriate and applicable because the updated PGA for the site does not exceed the PGA previously used.

It has been a pleasure to be of professional service to you. Please contact us with questions or if we can be of further assistance.

Sincerely,

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Attachments:

Appendix A – Round 3 Interrogatory CAW R313-25-8(4)-16/3, URS, 1/30/2012
Appendix B -- Figures and Tables for Seismic Hazard Evaluation
Appendix C -- Figures and Tables for Soil Liquefaction and Cyclic Softening Evaluation

It has been a pleasure to be of professional service to you. Please contact us with questions or if we can be of further assistance.

Sincerely,

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Appendix A – Round 3 Interrogatory CAW R313-25-8(4)-16/3, URS, 1/30/2012
Appendix B -- Figures and Tables for Seismic Hazard Evaluation
Appendix C -- Figures and Tables for Soil Liquefaction and Cyclic Softening Evaluation

References

Abrahamson, N.A., and Silva, W.J., 2008, Summary of the Abrahamson & Silva NGA Ground Motion Relations: Earthquake Spectra, vol. 24, no. 1, p. 67-97.

AMEC Earth & Environmental, Inc., 2005, Geotechnical Report, Combined Embankment Study, Envirocare, Clive, Utah. AMEC Job No. 4-817-004769. December 13, 2005.

AMEC Earth & Environmental, Inc., 2011a, Geotechnical Update Report, Class A West Embankment Study, Energy Solutions Clive Facility, Clive, Tooele County, Utah. AMEC Job No. 10-817-05290. February 15, 2011.

AMEC, Earth & Environmental, Inc., 2011b, "Response to Interrogatory CAW R313-25-8(4)-16/1: Seismic Hazard Analysis", Job Number 10-817-05290, October 25, 2011.

AMEC, Earth & Environmental, Inc., 2011c, "Response to Interrogatory CAW R313-25-8(4)-16/2: Seismic Hazard Analysis", Energy*Solutions* Clive Facility, Class A West Embankment, Clive, Tooele County, Utah. Job Number 10-817-05290, December 23, 2011.

Boore, D.M., and Atkinson, G.M., 2008, Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 s and 10.0 s, Earthquake Spectra, Vol. 24, No.1. p. 99-138.

Boulanger, R.W., and Idriss, I.M., 2004, Evaluating the Potential for Liquefaction and Cyclic Failure of Silts and Clays, Center for Geotechnical Modeling, University of California, Davis, Report No. *UCD/CGM-04/01.*

Boulanger, R.W., and Idriss, I.M., 2007, Evaluation of Cyclic Softening in Silts and Clays. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 133, No. 6, June 1, 2007 p. 641-652

Boulanger, R.W. and Idriss, I.M. 2008, Closure to "Liquefaction Susceptibility Criteria for Silts and Clays", Journal of Geotechnical and Geoenvironmental Engineering., ASCE, Vol. 134, No. 7, pp. 1027-1028.

Campbell, K.W., and Bozorgnia, Y., 2008, Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01s to 10.0 s, Earthquake Spectra, Vol. 24, No. 1. p. 139-171.

Chiou, B.S.J., and Youngs, R.R., 2008, An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra, Vol. 24, No.1. p. 173-215.

Geomatrix Consultants, Inc., 1999, Fault evaluation study and seismic hazard assessment, Private Fuel Storage Facility, Skull Valley, Utah: unpublished report prepared for Stone & Webster Engineering Corp., three volumes, variously paginated, Project 4790.

Ishihara, K., and Yoshimine, M., 1992, "Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes." Soils and Foundations, JSSMFE, Vol. 32 No. 1, March, p. 173-188.

Jefferies, M.G., and Davies, M.P., 1993, Use of CPTu to Estimate Equivalent SPT N_{60}, Geotechnical Testing Journal, ASTM, Vol. 16, No. 4, December 1993, p. 458-468.

Ladd, C.C., and Foott, R., 1974, New Design Procedures for Stability of Soft Clays, Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, pp. 763-786.

Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Y., Boyd, O.S., Perkins, D.M., Luco, N., Field, E.H., Wills, C.J., and Rukstales, K.S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008–1128, 61 p.

Risk Engineering Inc., 2010, EZ-FRISK™- Software for Earthquake Ground Motion Estimation, V7.62.

Robertson, P.K., and Wride, CE, 1998, Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test. Canadian Geotechnical Journal, Vol. 35, No. 3, p. 442-459.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M. Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris A. (2003). "Recent advances in soil liquefaction engineering: A unified and consistent framework." Keynote presentation, 26thAnnual ASCE Los Angeles Geotechnical Spring Seminar, H.M.S. Queen Mary, Long Beach, CA, USA, Report EERC-2003–06, Earthquake Engineering Research Institute, Berkeley, CA, USA.

Tokimatsu, T., and Seed, H.B., 1987, Evaluation of settlements in sands due to earthquake shaking, Journal of Geotechnical Engineering, ASCE, Vol.113, No. 8.

U.S. Geological Survey, 2008, Documentation of the 2008 Update of the United States National Seismic Hazard Maps, U.S. Department of the Interior, Open-File Report 2008-1128.

U.S. Geological Survey: Earthquake Hazards Program-Global V_s^{30} Map Server [Internet], Page Last Modified: May 25, 2011; Available from: http://earthquake.usgs.gov/hazards/apps/vs30/.

Utah Geological Survey, 2011, Working Group on Utah Earthquake Probabilities, Meeting # 4, February 16 and 17, 2011, http://geology.utah.gov/ghp/workgroups/wguep.htm (accessed Oct 2011).

Wells, D.L., and Coppersmith, K.J., 1994, New Empirical Relationships Among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement: Bulletin of the Seismological Society of America, Vol. 84, No. 4, p. 974-1002, August 1994.

Youd, T.L., DeDen, D.W., Bray, J.D., Sancio, R., Cetin, K.O., and Gerber, T.M. (2009). "Zero-displacement lateral spreads, 1999 Kocaeli, Turkey Earthquake." Journal of Geotechnical and Geoenvironmental Engineering., ASCE, Vol. 135, No. 1, pp. 46-61.

Youd, T. L., I. M. Idriss, R. D. Andrus, I. Arango, G. Castro, J. T. Christian, R. Dobry, W. D. L. Liam Finn, L. F. Harder, Jr., M. E. Hynes, K. Ishihara, J. P. Koester, S. S. C. Liao, W. F. Marcuson, III, G. R. Martin, J. K. Mitchell, Y. Moriwaki, M. S. Power, P. K. Robertson, R. B. Seed, and K. H. Stokoe, II (2001). Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, pp 817–833.

Appendix A

Round 3 Interrogatory CAW R313-25-8(4)-16/3
URS, 1/30/2012

The URS reference document has been modified so that only the text for Interrogatory CAW R313-25-8(4)-16/3 is presented.

UTAH DIVISION OF RADIATION CONTROL: ENERGY*SOLUTIONS'* CLIVE LLRW DISPOSAL FACILITY;

LICENSE AMENDMENT REQUEST; CLASS A WEST EMBANKMENT

ROUND 3 Interrogatories

This reference document has been modified so that only the following Interrogatory CAW R313-25-8(4)-16/3 is presented.

4. SEISMIC

INTERROGATORY CAW R313-25-8(4)-16/3: SEISMIC HAZARD EVALUATION / SEISMIC STABILITY ANALYSIS UPDATE

PRELIMINARY FINDING:

Refer to R313-25-8(5). Analyses of the long-term stability of the disposal site shall be based upon analyses of active natural processes including erosion, mass wasting, slope failure, settlement of wastes and backfill, infiltration through covers over disposal areas and adjacent soils, and surface drainage of the disposal site. The analyses shall provide reasonable assurance that there will not be a need for ongoing active maintenance of the disposal site following closure.

INTERROGATORY STATEMENT:

SEISMIC HAZARD ANALYSIS

The responses to Interrogatory CAW R313-25-8(4)-16/2A are inadequate and do not address the requests stated in the interrogatory. The Round 2 interrogatory is, therefore, restated and extended below:

1. *Perform and submit results of a probabilistic seismic hazard analysis (PSHA). Utilize a corrected version of EZ-FRISK or other accepted software for conducting PSHAs. Prepare the PSHA without using the Pankow and Pechmann (2004) ground motion model and giving full weight to the NGA models. Remove foreshocks and aftershocks from the earthquake history by de-clustering the earthquake catalog as done in standard probabilistic approaches*

and as was done by Pechmann and Arabasz (1995). Use the resulting earthquake catalog in any ground motion calculations.

2. *Correctly label the vertical axis in Figure 3 and correctly interpret the information presented in Figure 3 (cumulative frequency plot).*

SOIL LIQUEFACTION/CYCLIC SOFTENING

3. *The Applicant's response to Round 2 CAW interrogatories does not include a response to Interrogatory CAW R313-25-8(4)-16/2C. Please respond to Interrogatory CAW R313-25-8(4)-16/2C, repeated and extended below:*

   Please demonstrate that the potential effects of soil liquefaction and/or cyclic softening phenomena in native soils at the Clive Facility have been adequately accounted for in the geotechnical analyses supporting the design of the proposed CAW Embankment. In doing so, clearly justify the selection of soil parameters and any design assumptions by comparison of such with correlations, field test results, and/or laboratory test results (including cyclic shear testing) consistent with the guidance given by developers of current, published analytical methods. Also, evaluate and document the effects on previous liquefaction and embankment stability assessments and cyclic softening resulting from changes to the seismic hazard analysis addressed in Items 1 and 2 of this interrogatory.

BASIS FOR INTERROGATORY:

SEISMIC HAZARD ANALYSIS:

As requested in Interrogatory CAW R313-25-8(4)-16/2A, the Licensee did not justify the use of the semi-probabilistic approach for addressing the hazard from background earthquakes. The semi-probabilistic approach is not "standard practice for earth embankments" as stated in the response. For example, Utah Dam Safety and the Utah Division of Water Resources use a 5000-year return period probabilistic hazard map for the State produced by the U.S. Geological Survey to address background earthquakes in Utah. Utah Dam Safety recognized the deficiencies in semi-probabilistic approaches and decided to take a probabilistic seismic hazard analysis (PSHA) approach to address background earthquake hazard, as do the majority of seismic hazard evaluation practitioners currently. The Licensee did not address the criticisms of Pechmann and Arabasz (1995) of the semi-probabilistic approach as described in the Basis for Interrogatory. Although the Licensee did perform a PSHA in lieu of the semi-probabilistic approach as requested in the Interrogatory, the following comments must be addressed to produce an acceptable PSHA.

It appears that the Licensee used the semi-probabilistic approach to address the hazard from background earthquakes because they did not have a PSHA code that they thought was suitable for use for the Clive site. The Licensee should not constrain their response by their earlier seismic hazard evaluation (October 25, 2011 letter) simply because they lack a suitable PSHA

code. Also the Licensee is using a version of the proprietary code EZ-FRISK that contains an error in the hanging wall term of the Abrahamson and Silva (2008) NGA ground motion prediction model. The Licensee states that they requested Risk Engineering in 2009, the developer of EZ-FRISK, to make the correction but that they are still waiting for a corrected version of EZ-FRISK. It is suggested that the Licensee switch to another available PSHA code. Alternatively, Bob Youngs of AMEC Geomatrix is a nationally recognized practitioner in PSHAs who has developed his own codes.

The reviewer is puzzled why the ground motion prediction model by Pankow and Pechmann (2004) was used in the response (Addendum) to Interrogatory 2 when it was not used in their 25 October 2011 analysis, which only used the NGA models. The use of this model by the Licensee in the deterministic hazard analysis and PSHA described in the Addendum is not state-of-the-practice. The USGS did not use the Pankow and Pechmann (2004) model in the National Seismic Hazard Maps because the NGA models were available. Both Drs. Pankow and Pechmann at the University of Utah support the use of the NGA models in lieu of their own model. The Licensee's assertion that the NGA models are based mainly on California data and data from compressional tectonic regimes is incorrect. The NGA models are appropriate for normal faulting regimes (such as Utah), as was recognized by the USGS in their development of the National Seismic Hazard Maps. The database of normal faulting strong motion data used in the NGA models is superior to the database used in the original Spudich et al. (1999) model and subsequently by Pankow and Pechmann (2004). Hence the calculations performed by the Licensee should be repeated without the use of the Pankow and Pechmann (2004) ground motion model and the full weight should be given to the NGA models.

The current state-of-practice in seismic hazard evaluations is to not consider the hazard from foreshocks or aftershocks. In the calculation of historical seismicity recurrence, dependent events such as foreshocks and aftershocks are removed from the historical catalog through the process of declustering. In particular, it is important to remove dependent events when calculating seismicity rates for input into probabilistic seismic hazard analysis (PSHA) because PSHA assumes that earthquakes follow a Poisson process; that is they are independent events. As stated by McGuire (2004), "foreshocks and aftershocks are by definition smaller than the mainshock and thus the associated motions will generally be smaller. As a result, the standard procedure in PSHA is to include only mainshocks in deriving magnitude distributions". We note the Licensee is not following standard practice by including dependent events in their semi-deterministic approach; however, we agree that their calculated recurrence is conservative. The use of the 84th percentile ground motions is an accepted approach to include conservatism in a deterministic hazard analysis. Hence it should be noted that the Licensee is including additional conservatism over and above the use of 84th percentile ground motions in their hazard analyses.

The continued use of the terms "recurrence interval" and "return period" by the Licensee as interchangeable is incorrect. The USGS website cited by the Licensee in their response is incorrect and the USGS has been notified and are in the process of correcting this error (Mark Petersen, USGS, written communication, 2012). As stated by Robin McGuire, the author of the

PSHA code EZ-FRISK used by the Licensee, in his book on "Seismic Hazard and Risk Analysis" the two terms are defined as follows:

> *Return Period: the mean (average) time between occurrences of a seismic hazard, for example, a certain ground motion at a site, or a certain level of damage or loss.*
>
> *Recurrence Interval: the mean (average) time between occurrences of a given type of earthquake, for example, an earthquake of a specified magnitude, on a fault or in a region.*

SOIL LIQUEFACTION/CYCLIC SOFTENING:

In recent years, the geotechnical engineering profession's understanding of, and analysis methods for, liquefaction-related phenomena have evolved. In the past, liquefaction was treated as a phenomenon largely associated with the seismic loading of loose, clean sands which could result in significant loss of strength and large deformations. However, the 1999 earthquakes in Kocaeli, Turkey and Chi-Chi, Taiwan both highlighted the potential for significant strength loss and deformation of finer-grained soils – soils previously considered "non-liquefiable." Subsequent research (e.g., Andrews and Martin, 2000; Seed et al., 2003; Boulanger and Idriss, 2004, 2005, and 2006; Bray and Sancio, 2006; and Youd et al., 2009) has generally led to a distinction between "sand-like" soils which undergo liquefaction and "clay-like" soils which undergo cyclic-softening. Both phenomena are generally associated with generation of high pore pressures and strains during shear; however, the distinction between liquefaction and cyclic softening is important in that the methods of analysis and assessment are different for the different types of soil. Also important is that the resulting behaviors can vary.

In previous reports as well as "Geotechnical Update Report" dated February 15, 2011 (Attachment 5 to EnergySolutions, 2011), the Licensee addressed liquefaction susceptibility using site specific data and analyses (see Section 4.5.2, page 19, of referenced document). However, rather than presenting quantitative factors of safety and/or cyclic resistance and cyclic stress ratios, the Licensee qualitatively summarized the results of the analyses thusly:

> *"The 2005 study determined that for the design event, the majority of the soils in the upper 30 to 60 feet of the soil profile 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 would also be unlikely to liquefy under the design seismic event."*

A close reading of this statement reveals that the susceptibility of non-silty or "clean" sands (those which, if loose, are most prone to liquefaction) which may be at the site has not been addressed. Also, from the information provided, it is not clear how the finer-grained soils were treated in the analyses. Similarly, with respect to slope stability and other deformation-related assessments, it is unclear how the shear strengths of finer-grained soils subject to seismic loading conditions were assessed and quantified. Reported fines content, moisture content, and Atterberg limit data suggest that some of the loose/soft soils at the site are "marginal" soils which may or may not experience liquefaction and/or cyclic softening. Published guidance and criteria (e.g., Youd et al., 2001; Boulanger and Idriss, 2008, Bray and Sancio, 2008, Boulanger

and Idriss, 2011) currently referenced in the geotechnical engineering profession typically recommend that such soils be examined in greater detail and potentially be subjected to cyclic shear testing.

Stability and deformation calculations for existing embankments may be affected by the particular issues described in this interrogatory. Stability and deformations associated with the proposed CAW Embankment, particularly given the increase in embankment height and longer slopes of this embankment relative to other embankments at the Clive Facility, need to be assessed with consideration given to these issues.

Also, it should be noted that other current/recent interrogatories submitted for the proposed CAW Embankment License Amendment Request focus on further verifying the level of ground acceleration expected at the site. The effect of any revisions to such parameters on previous liquefaction and embankment stability assessments need to evaluated and documented.

REFERENCES

SEISMIC HAZARD ANALYSIS:

Abrahamson, N.A. and Silva, W.J., 2008, Summary of the Abrahamson and Silva NGA ground motion relations: Earthquake Spectra, v. 24, p. 67-97.

McGuire, R.K., 2004, Seismic hazard and risk analysis: Earthquake Engineering Research Institute, MNO-10, 240 p.

Pankow, K.L. and Pechmann, J.C., 2004, The SEA99 ground-motion predictive relations for extensional tectonic regimes: Revisions and a new peak ground velocity relation: Bulletin of the Seismological Society of America, v. 94, p. 341-348.

Pechmann, J.C. and Arabasz, W.J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, in W. R. Lund (ed.), Environmental and Engineering Geology of the Wasatch Front Region, 1995 Symposium and Field Conference: Utah Geological Association Publication 24, p. 77-93.

Spudich, P., Joyner, W.B., Lindh, A.G., Boore, D.M., Margaris, B.M., and Fletcher, J.B., 1999, SEA99 - A revised ground motion prediction relation for use in extensional tectonic regimes: Bulletin of the Seismological Society of America, v. 89, p. 1156-1170.

SOIL LIQUEFACTION/CYCLIC SOFTENING:

Andrews, D.C.A. and Martin, G.R. (2000).“Criteria for liquefaction of silty soils.” Proc. Twelfth World Conf. on Earthquake Eng., Auckland, New Zealand.

Boulanger, R.W. and Idriss, I.M. (2004). “Evaluating the potential for liquefaction or cyclic failure of silts and clays.” Rep.No.UCD/CGM-04/01, Center for Geotechnical Modeling, Dept. of Civil and Environmental Eng., Univ. of California, Davis ,CA, USA.

Boulanger, R.W. and Idriss, I.M. (2005). “New criteria for distinguishing between silts and clays that are susceptible to liquefaction versus cyclic failure.” Proc. Technologies to Enhance

Dam Safety and the Environment, 25th Annual United States Society on Dams Conf., USSD, Denver, pp. 357–366.

Boulanger, R.W. and Idriss, I.M. (2006). "Liquefaction susceptibility criteria for silts and clays." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 132, No. 11, pp. 1413-1426.

Boulanger, R.W. and Idriss, I.M. (2008). Closure to "Liquefaction susceptibility criteria for silts and clays." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 134, No. 7, pp. 1027-1028.

Boulanger, R.W. and Idriss, I.M. (2011). "Cyclic failure and liquefaction: Current issues." Proc. Fifth International Conf. of Earthquake Geotechnical Eng., Santiago, Chile.

Bray, J.D. and Sancio, R.B. (2006). "Assessment of the liquefaction susceptibility of fine grained soils." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 132, No. 9, pp. 1165-1177.

Bray, J.D. and Sancio, R.B. (2008). Closure to "Assessment of the liquefaction susceptibility of fine grained soils." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 134, No. 7, pp. 1031-1034.

EnergySolutions (2011). License Amendment Request: Class A West Embankment, with Attachments 1 through 7 and cover letter to Mr. Rusty Lundberg at Utah Division of Radiation Control dated May 2, 2011.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M. Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris A. (2003). "Recent advances in soil liquefaction engineering: A unified and consistent framework." Keynote presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, H.M.S. Queen Mary, Long Beach, CA, USA, Report EERC-2003–06, Earthquake Engineering Research Institute, Berkeley, CA, USA.

Youd, T.L., DeDen, D.W., Bray, J.D., Sancio, R., Cetin, O., and Gerber, T.M. (2009). "Zero-displacement lateral spreads, 1999 Kocaeli, Turkey Earthquake." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 135, No. 1, pp. 46-61.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Jr., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., III, Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., II. (2001). "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 127, No. 10, pp. 817-833.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Jr., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., III, Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., II. (2001). "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 127, No. 10, pp. 817-833.

Appendix B – **Figures and Tables for Seismic Hazard Evaluation**

Figure B-1.1 – Quaternary Faults and Historical Seismicity near Energy*Solutions* Site

Table B-1.1 – Summary of Earthquake Source Parameters and DSHA and PSHA Results

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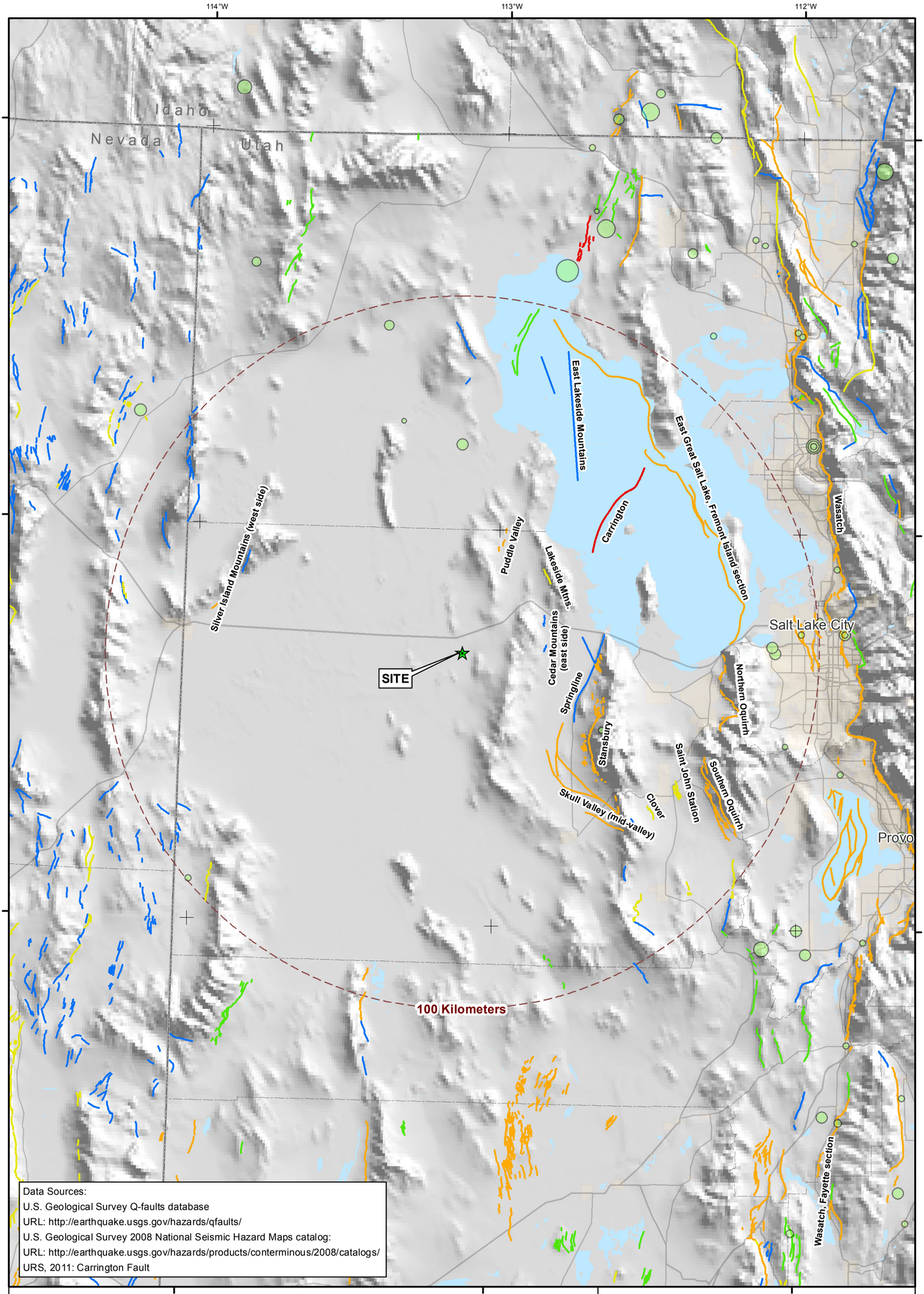


Explanation
Fault line color denotes age of most recent displacement

- Historic
- Holocene/Late Pleistocene
- Late Quaternary
- Mid Quaternary
- Quaternary

Independent Seismicity (1769-2006)
Magnitude (M)

- 4.0
- 5.0
- 6.0
- 7.0

0 10 20 Miles

N

QUATERNARY FAULTS AND HISTORICAL SEISMICITY NEAR ENERGY SOLUTIONS SITE
Clive, Tooele County, Utah

By: _____	Date: 04/04/2012	Project No. 10-817-05290.2
amec		Figure B-1.1

Table B-1.1 Summary of Earthquake Source Parameters and DSHA and PSHA Results
Response to Interrogatory: Seismic Hazard Evaluation
Energy*Solutions* Clive Facility
Clive, Utah

Fault Sources - DSHA	Fault Parameters							Distance (km)			PGA (g)	
	Type	M_{max}	HW	Dip	Rake	Length	Width	r_x	r_{jb}	r_{rup}	Median	84th
East Cedar Mountains	NM	6.6	No	55	-90	10	18.3	23.0	23.0	23.0	0.12	0.21
Puddle Valley	NM	6.5	Yes	55	-90	7	18.3	30.0	19.5	24.6	0.12	0.21
Lakeside Mountain	NM	6.5	Yes	55	-90	4	18.3	31.0	20.5	25.4	0.12	0.20
Skull Valley	NM	7.1	Yes	55	-90	32	18.3	34.5	24.0	28.3	0.14	0.24
Springline	NM	6.8	Yes	55	-90	18	18.3	35.0	24.5	28.7	0.12	0.21
Stansbury	NM	7.3	Yes	55	-90	69	18.3	37.0	26.5	30.4	0.16	0.24
Carrington	NM	7.1	Yes	55	-90	30	18.3	47.0	36.5	39.5	0.10	0.17
East Lakeside Mountain	NM	6.9	No	55	-90	36	18.3	57.0	57.0	57.0	0.06	0.11
Silver Island Mountains	NM	6.5	No	55	-90	6	18.3	65.0	65.0	65.0	0.04	0.07
Clover	NM	6.5	No	55	-90	4	18.3	68.0	68.0	68.0	0.04	0.07
Saint John Station	NM	6.5	Yes	55	-90	5	18.3	70.0	59.5	61.4	0.05	0.08
Oquirrh-East Great Salt Lake	NM	7.5	Yes	55	-90	61	18.3	75.0	64.5	66.2	0.08	0.14
Background Source - PSHA	**Type**	**M_{max}**									**Median**	
Western U.S. Extensional Gridded Background, 5000 year return period	SS & NM	7.0									0.24	

Notes:
1. Spectral accelerations are mean of four NGA relationships (Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008)).
2. Site shear wave velocity (V_{S30}) is 305 m/s; default (calculated) values of $Z_{1.0}$ and $Z_{2.5}$ are 417 m and 2.02 km, respectively.
3. Fault Type: NM – Normal, SS – Strike slip
4. M_{max} - Maximum magnitude from Utah Working Group on Earthquake Probabilities or maximum magnitude based on empirical relationships of Wells and Coppersmith (1994). For short faults of less than 10 km length, including Puddle Valley, Lakeside Mountain, Silver Island Mountains, Clover, and Saint John Station, a minimum value of maximum magnitude is assessed as 6.5. See text for additional discussion.
5. Fault geometry is based on the geometry of the closest fault segment to the site, not the average geometry of the combined rupture segment.
6. HW: Hanging wall condition; Yes = On Hanging Wall, No = Not On Hanging Wall
7. TOR = Depth to top of rupture (0 for all faults); DTR = Depth to bottom of rupture (15 km for all faults); DWR = Down dip width of rupture (18.3 km for 55º dip for all faults).
8. r_x = Horizontal distance from top of rupture measured perpendicular to fault strike; r_{jb} = Boore-Joyner distance; r_{rup} = Fault Rupture Distance

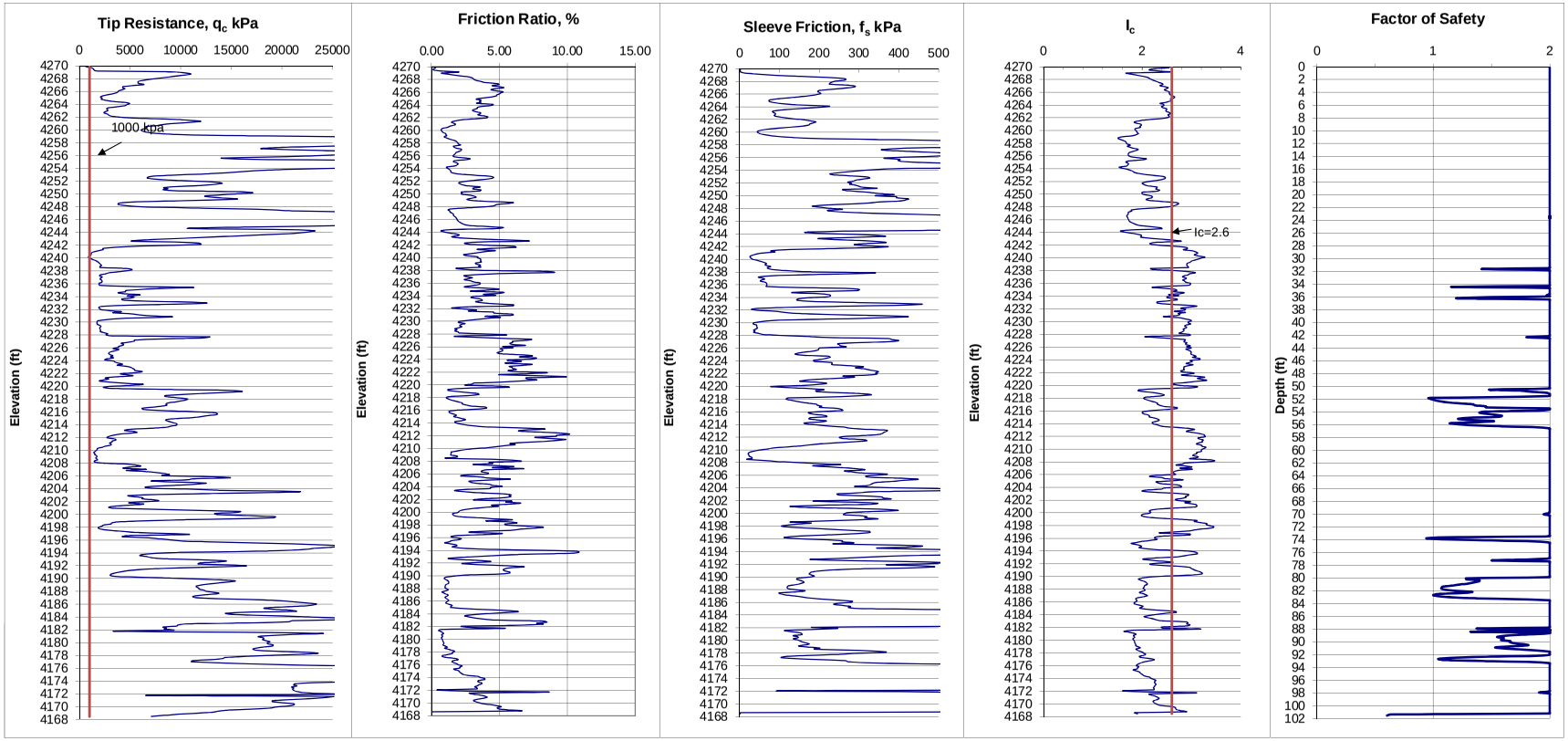
Appendix C – **Figures and Tables for Soil Liquefaction and Cyclic Softening Evaluation**

Figures C-1.1 through C-1.6 – Factor of Safety vs. Depth
Figures C-1.7 & C-1.8 – Slope Stability Calculations (static undrained strengths)

Table C-1.1 -- Cyclic Softening of "Clay-like" Soils using M = 7.3 and PGA = 0.24 g
Table C-1.2 -- Cyclic Softening of "Clay-like" Soils using M = 7.3 and PGA = 0.28 g

EnergySolutions -CAW Embankment
No.:10-817-05290
April 3, 2012

Cone Log #99-01



Values used in Analysis: M=7.3, amax=0.28, Ground elev=4270, Water table elev=4248.00

Figure C-1.1

EnergySolutions - CAW Embankment
Project No.: 10-817-05290
April 3, 2012

Cone Log #99-02

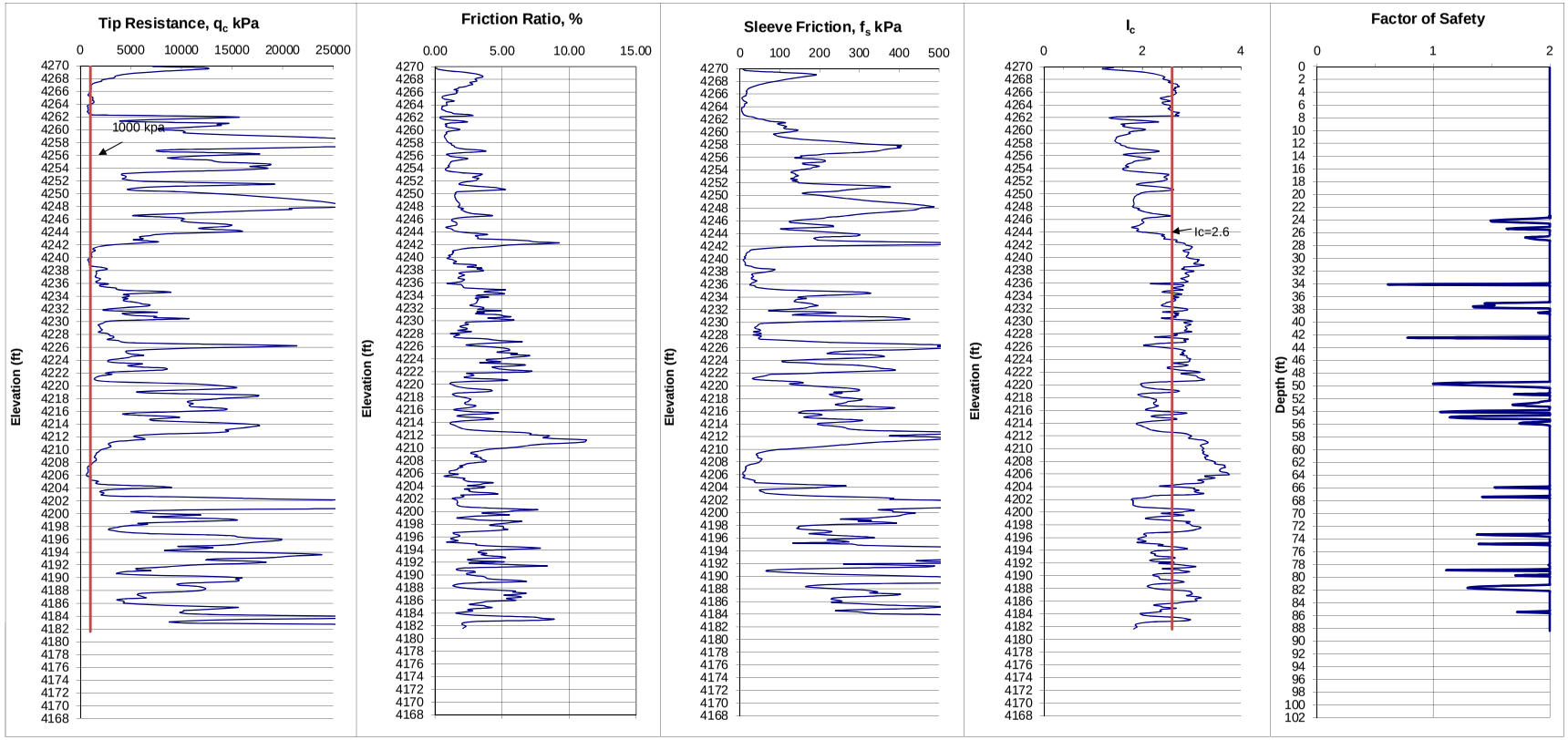


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Figure C-1.2

Energy Solutions -CAW Embankment
Project No. 10-817-05290
April 3, 2012

Cone Log #99-03

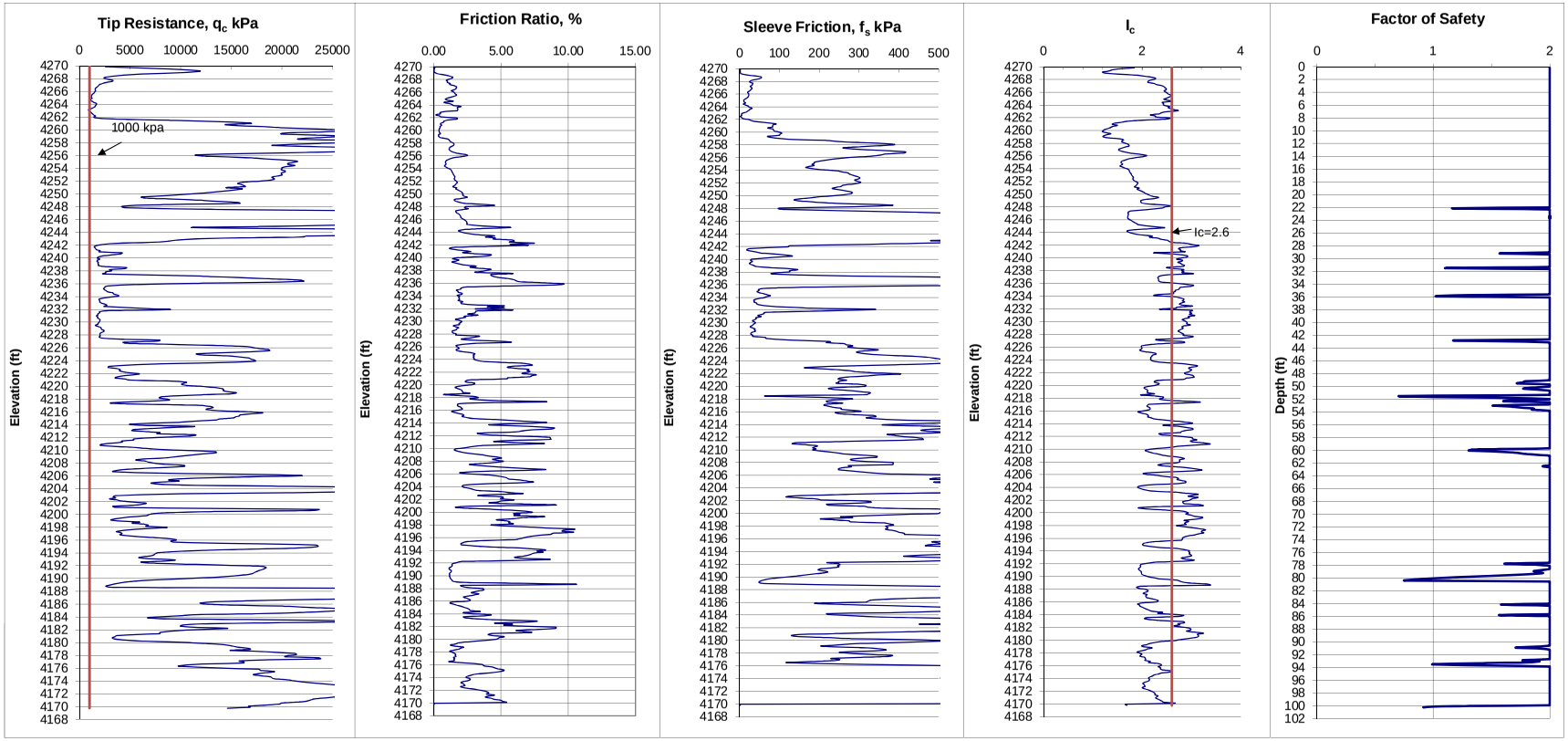

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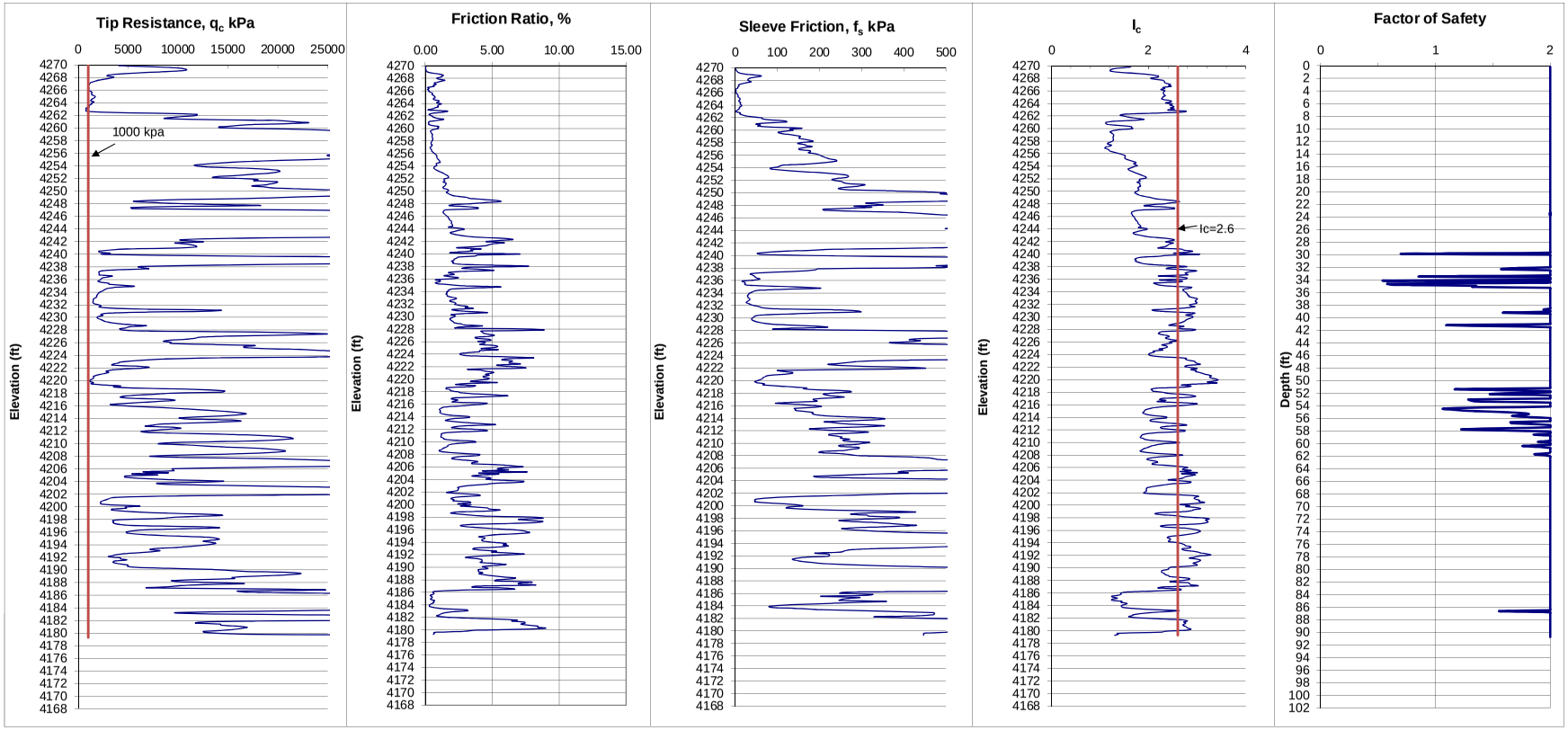
Figure C-1.3

Energy Solutions - CAW Embankment
Project No.: 10-817-05290
April 3, 2012

Cone Log #04-04

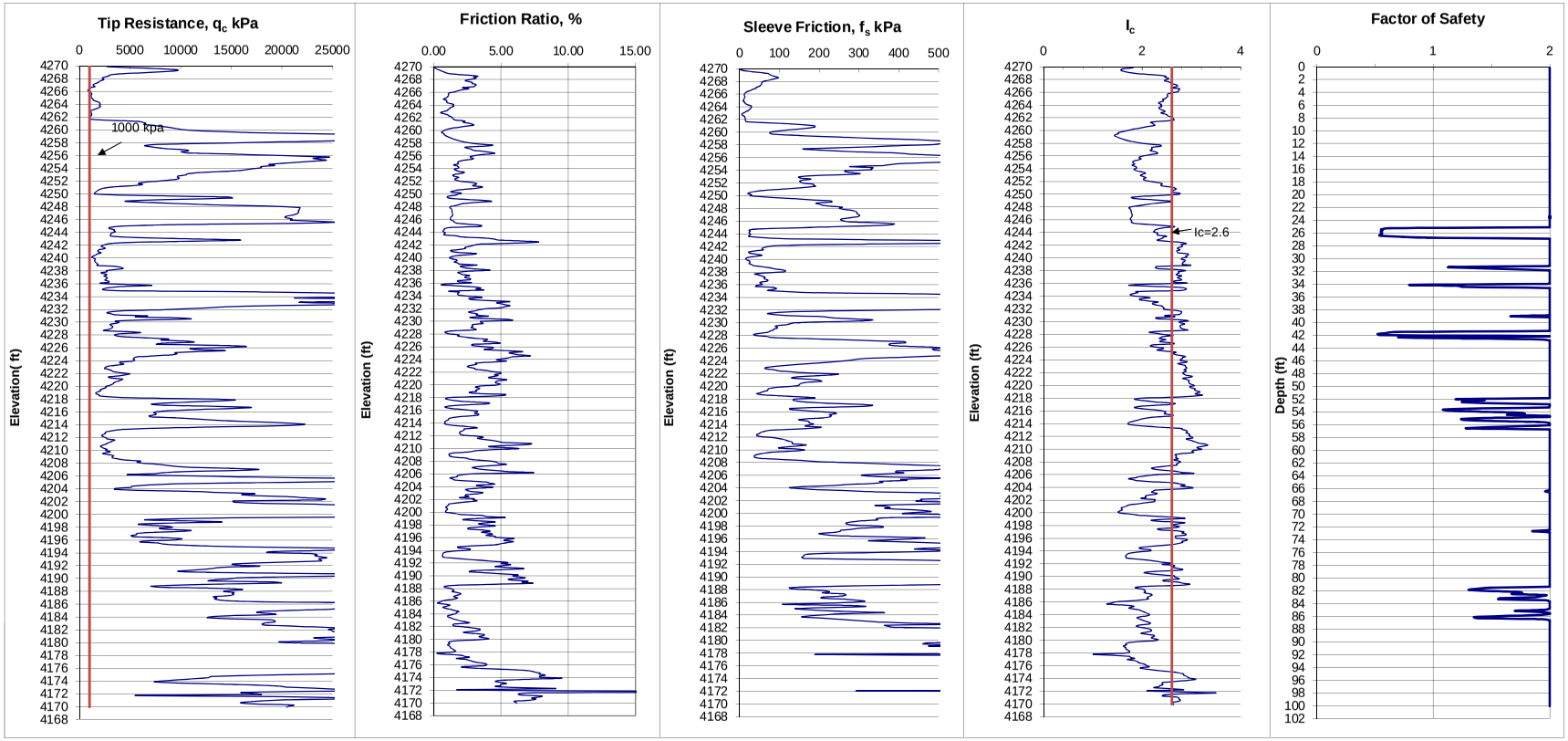


Values used in Analysis: M=7.3, amax=0.28, Ground elev=4270, Water table elev=4248.00

Figure C-1.4

EnergySolutions - CAW Embankment
Project No.: 10-817-05290
April 3, 2012

Cone Log #04-05

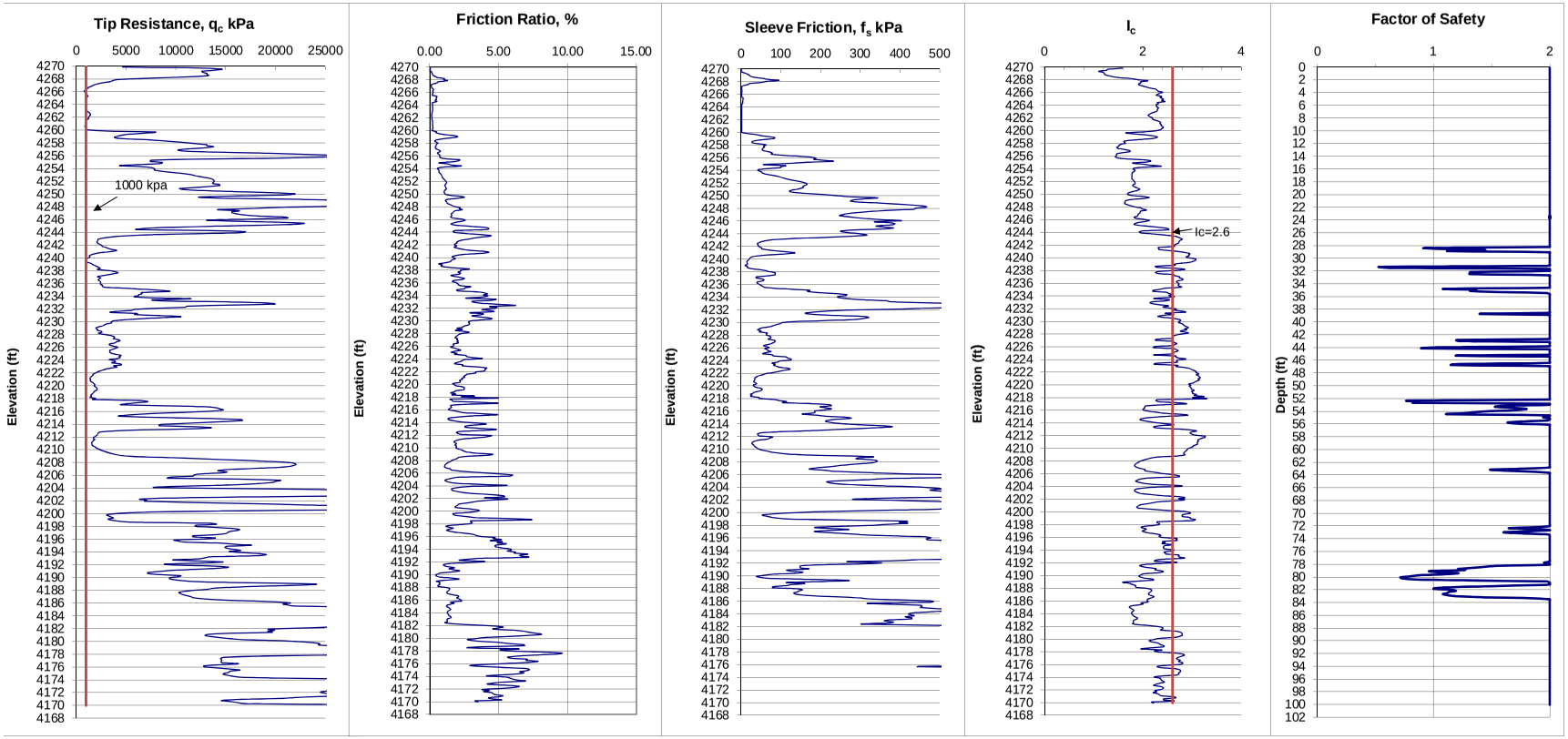


Values used in Analysis: M=7.3, amax=0.28, Ground elev=4270, Water table elev=4248

Figure C-1.5

Energy Solutions - CAW Embankment
Project No.:10-817-05290
April 3, 2012

Cone Log #04-06



Values used in Analysis: M=7.3, amax=0.28, Ground elev=4270, Water table elev=4248.00

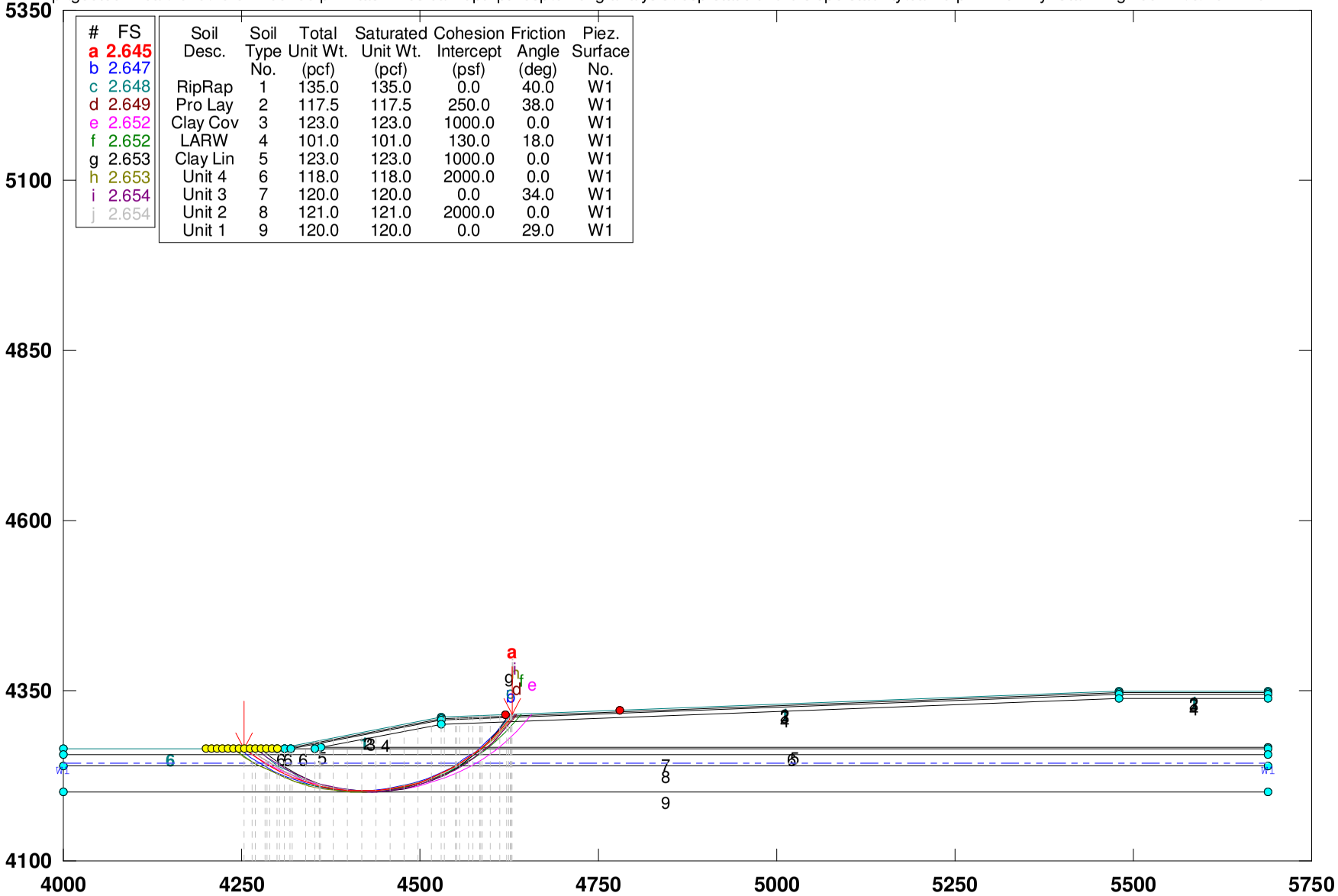
Figure C.1-6

CAW Emb 38' 5(H):1(V) side-slopes, static - Su, Deep Surf 020212 CAWFU.DTA

p:\geotechnical\2010\10-817-05290 ph 1 task 2 es caw upt rpt-resp\04 eng analysis\resp static undrd slope stability\cawfu.pl2 Run By: Staff Engineer 4/5/2012 02:47PM

#	FS
a	2.645
b	2.647
c	2.648
d	2.649
e	2.652
f	2.652
g	2.653
h	2.653
i	2.654
j	2.654

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
RipRap	1	135.0	135.0	0.0	40.0	W1
Pro Lay	2	117.5	117.5	250.0	38.0	W1
Clay Cov	3	123.0	123.0	1000.0	0.0	W1
LARW	4	101.0	101.0	130.0	18.0	W1
Clay Lin	5	123.0	123.0	1000.0	0.0	W1
Unit 4	6	118.0	118.0	2000.0	0.0	W1
Unit 3	7	120.0	120.0	0.0	34.0	W1
Unit 2	8	121.0	121.0	2000.0	0.0	W1
Unit 1	9	120.0	120.0	0.0	29.0	W1



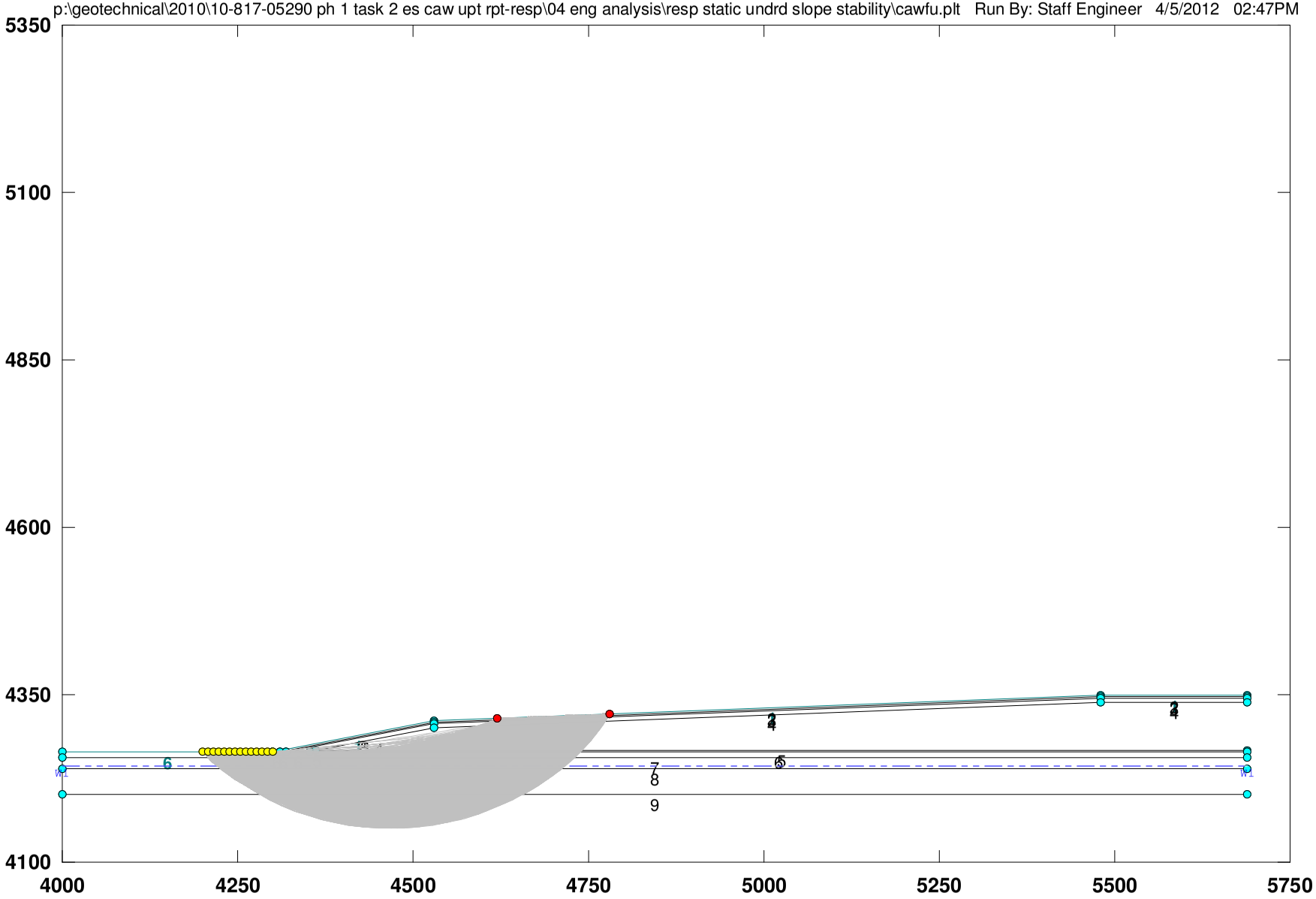
GSTABL7 v.2 FSmin=2.645

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

GSTABL7

CAW Emb 38' 5(H):1(V) side-slopes, static - Su, Deep Surf 020212 CAWFU.DTA

p:\geotechnical\2010\10-817-05290 ph 1 task 2 es caw upt rpt-resp\04 eng analysis\resp static undrd slope stability\cawfu.plt Run By: Staff Engineer 4/5/2012 02:47PM



GSTABL7

```
                                   ***  GSTABL7  ***

                          ** GSTABL7 by Garry H. Gregory, P.E. **

             ** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **
                         (All Rights Reserved-Unauthorized Use Prohibited)


          ******************************************************************************
                              SLOPE STABILITY ANALYSIS SYSTEM
                     Modified Bishop, Simplified Janbu, or GLE Method of Slices.
                     (Includes Spencer & Morgenstern-Price Type Analysis)
                     Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
                     Nonlinear Undrained Shear Strength, Curved Phi Envelope,
                     Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
                     Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
          ******************************************************************************


          Analysis Run Date:        4/5/2012
          Time of Run:              02:47PM
          Run By:                   Staff Engineer
          Input Data Filename:      P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawfu.in
          Output Filename:          P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawfu.OUT
          Unit System:              English

          Plotted Output Filename:  P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawfu.PLT




          PROBLEM DESCRIPTION:  CAW Emb 38' 5(H):1(V) side-slopes,
                                static - Su, Deep Surf 020212 CAWFU.DTA



          BOUNDARY COORDINATES

              4 Top   Boundaries
             22 Total Boundaries


          Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
             No.        (ft)       (ft)       (ft)       (ft)     Below Bnd

              1       4000.00    4265.00    4300.00    4265.00        6
              2       4300.00    4265.00    4530.00    4311.00        1
              3       4530.00    4311.00    5480.00    4349.00        1
              4       5480.00    4349.00    5689.00    4349.00        1
              5       4300.00    4265.00    4310.00    4265.00        6
              6       4310.00    4265.00    4530.00    4309.00        2
              7       4530.00    4309.00    5480.00    4347.00        2
              8       5480.00    4347.00    5689.00    4347.00        2
              9       4310.00    4265.00    4318.00    4265.00        6
             10       4318.00    4265.00    4530.00    4307.50        3
             11       4530.00    4307.50    5480.00    4345.50        3
             12       5480.00    4345.50    5689.00    4345.50        3
             13       4318.00    4265.00    4353.00    4265.00        6
             14       4353.00    4265.00    4361.00    4267.00        5
             15       4361.00    4267.00    4530.00    4300.50        4
             16       4530.00    4300.50    5480.00    4338.50        4
             17       5480.00    4338.50    5689.00    4338.50        4
             18       4361.00    4267.00    5689.00    4267.00        5
             19       4353.00    4265.00    5689.00    4265.00        6
             20       4000.00    4256.00    5689.00    4256.00        7
             21       4000.00    4239.00    5689.00    4239.00        8
             22       4000.00    4201.00    5689.00    4201.00        9

          User Specified Y-Origin =      4100.00(ft)

          Default X-Plus Value = 0.00(ft)

          Default Y-Plus Value = 0.00(ft)
1


         ISOTROPIC SOIL PARAMETERS


           9 Type(s) of Soil


          Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
          Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
           No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.

             1   135.0    135.0       0.0     40.0    0.00       0.0      1
             2   117.5    117.5     250.0     38.0    0.00       0.0      1
```

3	123.0	123.0	1000.0	0.0	0.00	0.0	1
4	101.0	101.0	130.0	18.0	0.00	0.0	1
5	123.0	123.0	1000.0	0.0	0.00	0.0	1
6	118.0	118.0	2000.0	0.0	0.00	0.0	1
7	120.0	120.0	0.0	34.0	0.00	0.0	1
8	121.0	121.0	2000.0	0.0	0.00	0.0	1
9	120.0	120.0	0.0	29.0	0.00	0.0	1

1

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	4000.00	4243.00
2	5689.00	4243.00

EARTHQUAKE DATA HAS BEEN SUPPRESSED

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

4200 Trial Surfaces Have Been Generated.

300 Surface(s) Initiate(s) From Each Of 14 Points Equally Spaced
Along The Ground Surface Between X =4200.00(ft)
and X =4300.00(ft)

Each Surface Terminates Between X =4620.00(ft)
and X =4780.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)

20.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1) * *

Selected ki function = Constant (1.0)

Selected Lambda Coefficient = 1.00

Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.

Specified Tension Crack Water Force Factor = 0.000

Total Number of Trial Surfaces Attempted = 4200

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 3

Number of Trial Surfaces with Misleading FS = 1

Number of Trial Surfaces With Valid FS = 4196

Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Attempted = 0.1 %

Statistical Data On All Valid FS Values:
FS Max = 5.907 FS Min = 2.645 FS Ave = 4.024
Standard Deviation = 0.726 Coefficient of Variation = 18.05 %

((Modified Bishop FS for Critical Surface = 2.649))

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4253.847	4265.000
2	4269.415	4252.445
3	4285.930	4241.165
4	4303.288	4231.229
5	4321.378	4222.701
6	4340.088	4215.635
7	4359.300	4210.074
8	4378.892	4206.054
9	4398.741	4203.601
10	4418.722	4202.729
11	4438.709	4203.444
12	4458.577	4205.742
13	4478.200	4209.607
14	4497.454	4215.017
15	4516.219	4221.937
16	4534.376	4230.322
17	4551.811	4240.121
18	4568.414	4251.271
19	4584.081	4263.704
20	4598.712	4277.339
21	4612.216	4292.091
22	4624.508	4307.868
23	4629.185	4314.967

Circle Center At X = 4419.711 ; Y = 4454.752 ; and Radius = 252.026

*** FOS = 2.645 Theta (ki=1.0) = 4.79 ***
Lambda = 0.084

Individual data on the 39 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	11.2	5926.0	0.0	0.0	0.	0.	0.0	0.0	0.0
2	4.4	5621.7	0.0	0.0	0.	0.	0.0	0.0	0.0
3	13.8	28420.7	0.0	0.0	0.	0.	0.0	0.0	0.0
4	2.7	7341.3	0.0	186.4	0.	0.	0.0	0.0	0.0
5	3.8	11238.2	0.0	793.2	0.	0.	0.0	0.0	0.0
6	10.3	35580.6	0.0	5137.0	0.	0.	0.0	0.0	0.0
7	3.3	13061.0	0.0	2560.0	0.	0.	0.0	0.0	0.0
8	6.7	29622.4	0.0	6183.2	0.	0.	0.0	0.0	0.0
9	8.0	40138.4	0.0	9283.1	0.	0.	0.0	0.0	0.0
10	3.4	18504.9	0.0	4545.3	0.	0.	0.0	0.0	0.0
11	18.7	117366.2	0.0	29742.3	0.	0.	0.0	0.0	0.0
12	12.9	94455.4	0.0	24520.1	0.	0.	0.0	0.0	0.0
13	6.3	49694.8	0.0	13101.6	0.	0.	0.0	0.0	0.0
14	1.7	13802.3	0.0	3584.8	0.	0.	0.0	0.0	0.0
15	17.9	153212.0	0.0	40015.1	0.	0.	0.0	0.0	0.0
16	19.8	184911.8	0.0	47640.2	0.	0.	0.0	0.0	0.0
17	20.0	198214.0	0.0	49714.6	0.	0.	0.0	0.0	0.0
18	20.0	206552.7	0.0	49813.0	0.	0.	0.0	0.0	0.0
19	19.9	209707.5	0.0	47932.5	0.	0.	0.0	0.0	0.0
20	19.6	207650.7	0.0	44086.3	0.	0.	0.0	0.0	0.0
21	19.3	200520.9	0.0	38297.8	0.	0.	0.0	0.0	0.0
22	18.8	188645.0	0.0	30604.9	0.	0.	0.0	0.0	0.0
23	13.8	132009.7	0.0	16937.8	0.	0.	0.0	0.0	0.0
24	4.4	40346.0	0.0	4117.2	0.	0.	0.0	0.0	0.0
25	15.4	132982.6	0.0	9216.3	0.	0.	0.0	0.0	0.0
26	2.0	16067.9	0.0	491.1	0.	0.	0.0	0.0	0.0
27	4.3	33556.7	0.0	463.8	0.	0.	0.0	0.0	0.0
28	12.3	88588.9	0.0	0.0	0.	0.	0.0	0.0	0.0
29	6.0	38430.5	0.0	0.0	0.	0.	0.0	0.0	0.0
30	9.7	55754.6	0.0	0.0	0.	0.	0.0	0.0	0.0
31	1.4	7281.8	0.0	0.0	0.	0.	0.0	0.0	0.0
32	2.1	10820.5	0.0	0.0	0.	0.	0.0	0.0	0.0
33	11.1	49078.7	0.0	0.0	0.	0.	0.0	0.0	0.0
34	13.5	43300.8	0.0	0.0	0.	0.	0.0	0.0	0.0
35	9.4	17853.2	0.0	0.0	0.	0.	0.0	0.0	0.0
36	2.9	3135.3	0.0	0.0	0.	0.	0.0	0.0	0.0

37	2.3	1514.8	0.0	0.0	0.	0.	0.0	0.0	0.0
38	1.0	363.6	0.0	0.0	0.	0.	0.0	0.0	0.0
39	1.4	182.6	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4246.154	4265.000
2	4261.792	4252.532
3	4278.352	4241.316
4	4295.733	4231.422
5	4313.831	4222.909
6	4332.535	4215.828
7	4351.733	4210.222
8	4371.309	4206.125
9	4391.144	4203.562
10	4411.118	4202.549
11	4431.111	4203.091
12	4451.001	4205.185
13	4470.668	4208.819
14	4489.993	4213.970
15	4508.859	4220.608
16	4527.153	4228.692
17	4544.763	4238.173
18	4561.582	4248.995
19	4577.510	4261.090
20	4592.449	4274.388
21	4606.310	4288.806
22	4619.008	4304.257
23	4626.417	4314.857

Circle Center At X = 4414.152 ; Y = 4459.664 ; and Radius = 257.133

*** FOS = 2.647 Theta (ki=1.0) = 4.75 ***
Lambda = 0.083

1

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4269.231	4265.000
2	4284.375	4251.936
3	4300.589	4240.227
4	4317.752	4229.960
5	4335.738	4221.212
6	4354.411	4214.049
7	4373.633	4208.523
8	4393.259	4204.676
9	4413.145	4202.536
10	4433.140	4202.120
11	4453.097	4203.431
12	4472.867	4206.458
13	4492.301	4211.180
14	4511.256	4217.560
15	4529.590	4225.552
16	4547.167	4235.096
17	4563.854	4246.121
18	4579.527	4258.543
19	4594.071	4272.272
20	4607.376	4287.205
21	4619.343	4303.229
22	4626.557	4314.862

Circle Center At X = 4427.871 ; Y = 4433.422 ; and Radius = 231.371

*** FOS = 2.648 Theta (ki=1.0) = 4.84 ***
Lambda = 0.085

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4261.539	4265.000
2	4277.084	4252.417
3	4293.584	4241.114
4	4310.934	4231.165
5	4329.022	4222.632

6	4347.734	4215.570
7	4366.950	4210.025
8	4386.547	4206.031
9	4406.401	4203.614
10	4426.384	4202.789
11	4446.369	4203.562
12	4466.228	4205.928
13	4485.835	4209.871
14	4505.065	4215.367
15	4523.795	4222.381
16	4541.906	4230.867
17	4559.281	4240.772
18	4575.810	4252.032
19	4591.388	4264.575
20	4605.915	4278.322
21	4619.297	4293.185
22	4631.451	4309.068
23	4635.421	4315.217

Circle Center At X = 4426.706 ; Y = 4453.148 ; and Radius = 250.359

*** FOS = 2.649 Theta (ki=1.0) = 4.80 ***
Lambda = 0.084

1

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4253.847	4265.000
2	4269.793	4252.930
3	4286.559	4242.024
4	4304.057	4232.339
5	4322.200	4223.923
6	4340.896	4216.819
7	4360.050	4211.063
8	4379.564	4206.685
9	4399.341	4203.705
10	4419.280	4202.140
11	4439.279	4201.997
12	4459.238	4203.276
13	4479.056	4205.973
14	4498.631	4210.072
15	4517.865	4215.553
16	4536.661	4222.389
17	4554.922	4230.544
18	4572.558	4239.978
19	4589.477	4250.643
20	4605.595	4262.483
21	4620.830	4275.441
22	4635.104	4289.449
23	4648.347	4304.438
24	4657.250	4316.090

Circle Center At X = 4431.220 ; Y = 4482.637 ; and Radius = 280.762

*** FOS = 2.652 Theta (ki=1.0) = 4.62 ***
Lambda = 0.081

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4238.462	4265.000
2	4254.411	4252.932
3	4271.178	4242.030
4	4288.678	4232.348
5	4306.823	4223.936
6	4325.521	4216.836
7	4344.676	4211.085
8	4364.192	4206.712
9	4383.970	4203.738
10	4403.909	4202.179
11	4423.908	4202.043
12	4443.867	4203.330
13	4463.683	4206.034
14	4483.257	4210.142
15	4502.489	4215.631
16	4521.281	4222.476
17	4539.539	4230.640
18	4557.170	4240.082

19	4574.084	4250.755
20	4590.196	4262.604
21	4605.424	4275.570
22	4619.690	4289.587
23	4632.923	4304.583
24	4641.214	4315.449

Circle Center At X = 4415.745 ; Y = 4482.571 ; and Radius = 280.653

*** FOS = 2.652 Theta (ki=1.0) = 4.61 ***
Lambda = 0.081

1

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4276.924	4265.000
2	4291.709	4251.531
3	4307.659	4239.465
4	4324.642	4228.902
5	4342.517	4219.929
6	4361.134	4212.622
7	4380.339	4207.040
8	4399.973	4203.231
9	4419.873	4201.226
10	4439.872	4201.041
11	4459.804	4202.679
12	4479.505	4206.125
13	4498.810	4211.351
14	4517.559	4218.314
15	4535.596	4226.956
16	4552.771	4237.204
17	4568.940	4248.974
18	4583.971	4262.167
19	4597.738	4276.675
20	4610.126	4292.376
21	4621.033	4309.140
22	4623.999	4314.760

Circle Center At X = 4431.805 ; Y = 4420.006 ; and Radius = 219.124

*** FOS = 2.653 Theta (ki=1.0) = 4.81 ***
Lambda = 0.084

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4238.462	4265.000
2	4254.313	4252.804
3	4271.014	4241.800
4	4288.474	4232.047
5	4306.601	4223.597
6	4325.298	4216.495
7	4344.464	4210.779
8	4363.997	4206.481
9	4383.791	4203.623
10	4403.742	4202.219
11	4423.742	4202.279
12	4443.684	4203.801
13	4463.461	4206.777
14	4482.968	4211.192
15	4502.099	4217.022
16	4520.753	4224.235
17	4538.830	4232.793
18	4556.232	4242.650
19	4572.867	4253.753
20	4588.645	4266.043
21	4603.482	4279.455
22	4617.298	4293.915
23	4630.021	4309.347
24	4634.142	4315.166

Circle Center At X = 4412.847 ; Y = 4475.083 ; and Radius = 273.030

*** FOS = 2.653 Theta (ki=1.0) = 4.65 ***
Lambda = 0.081

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4269.231	4265.000
2	4284.558	4252.151
3	4300.902	4240.624
4	4318.149	4230.498
5	4336.180	4221.844
6	4354.869	4214.723
7	4374.087	4209.183
8	4393.699	4205.264
9	4413.570	4202.993
10	4433.561	4202.385
11	4453.532	4203.444
12	4473.346	4206.164
13	4492.865	4210.525
14	4511.952	4216.498
15	4530.476	4224.040
16	4548.306	4233.099
17	4565.320	4243.612
18	4581.399	4255.506
19	4596.432	4268.698
20	4610.312	4283.097
21	4622.945	4298.603
22	4634.243	4315.106
23	4634.279	4315.171

Circle Center At X = 4430.850 ; Y = 4442.213 ; and Radius = 239.844

*** FOS = 2.654 Theta (ki=1.0) = 4.82 ***
Lambda = 0.084

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4261.539	4265.000
2	4276.996	4252.308
3	4293.441	4240.925
4	4310.764	4230.930
5	4328.849	4222.389
6	4347.573	4215.359
7	4366.810	4209.889
8	4386.431	4206.014
9	4406.304	4203.761
10	4426.294	4203.145
11	4446.268	4204.170
12	4466.090	4206.830
13	4485.628	4211.106
14	4504.749	4216.970
15	4523.325	4224.382
16	4541.230	4233.292
17	4558.345	4243.641
18	4574.553	4255.358
19	4589.746	4268.365
20	4603.821	4282.573
21	4616.684	4297.888
22	4628.248	4314.206
23	4628.687	4314.947

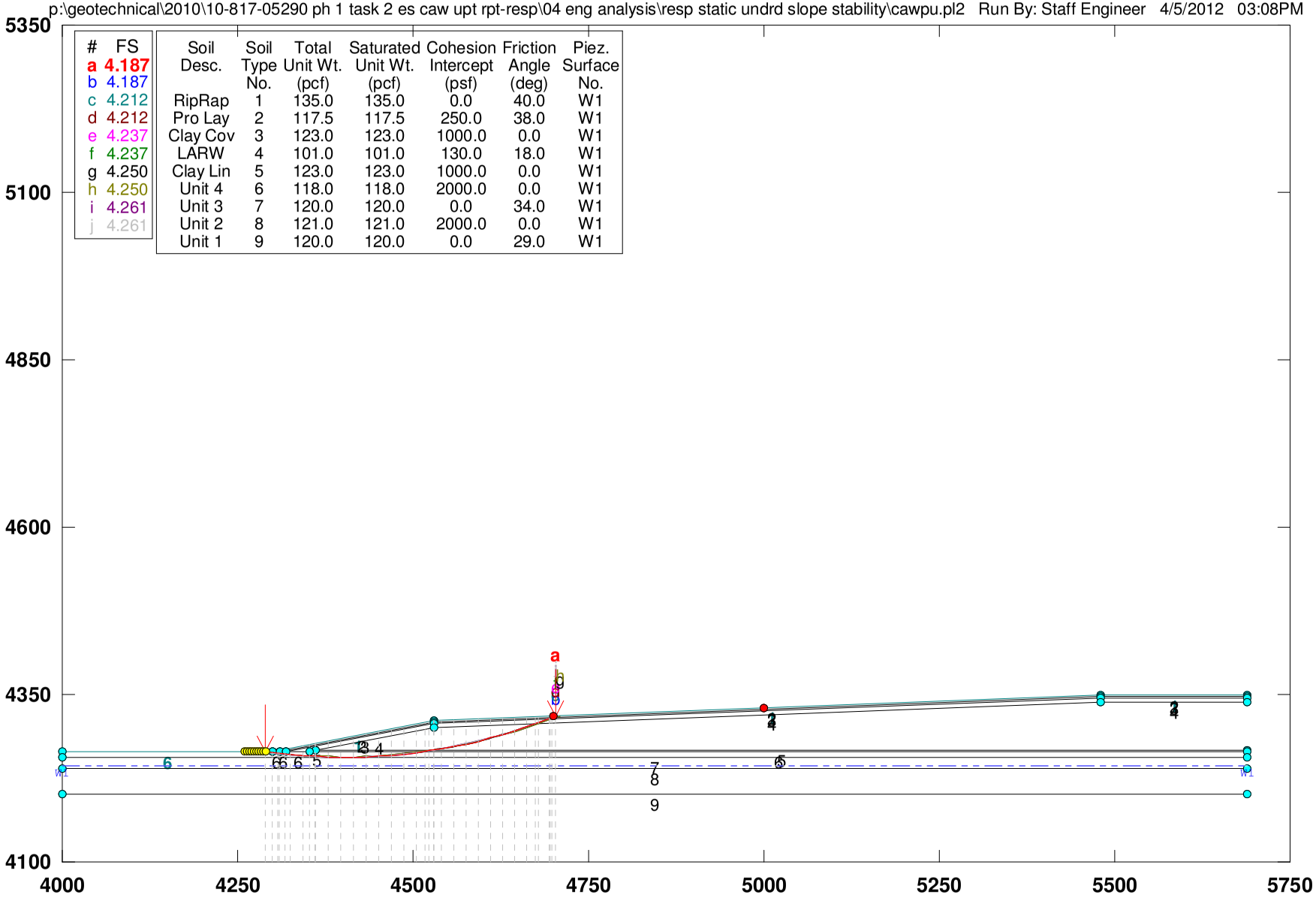
Circle Center At X = 4423.796 ; Y = 4446.835 ; and Radius = 243.703

*** FOS = 2.654 Theta (ki=1.0) = 4.86 ***
Lambda = 0.085

**** END OF GSTABL7 OUTPUT ****

CAW Emb 38' 5(H):1(V) side-slopes, static - Su, ShalClay unit 020212 CAWPU

p:\geotechnical\2010\10-817-05290 ph 1 task 2 es caw upt rpt-resp\04 eng analysis\resp static undrd slope stability\cawpu.pl2 Run By: Staff Engineer 4/5/2012 03:08PM



#	FS
a	4.187
b	4.187
c	4.212
d	4.212
e	4.237
f	4.237
g	4.250
h	4.250
i	4.261
j	4.261

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
RipRap	1	135.0	135.0	0.0	40.0	W1
Pro Lay	2	117.5	117.5	250.0	38.0	W1
Clay Cov	3	123.0	123.0	1000.0	0.0	W1
LARW	4	101.0	101.0	130.0	18.0	W1
Clay Lin	5	123.0	123.0	1000.0	0.0	W1
Unit 4	6	118.0	118.0	2000.0	0.0	W1
Unit 3	7	120.0	120.0	0.0	34.0	W1
Unit 2	8	121.0	121.0	2000.0	0.0	W1
Unit 1	9	120.0	120.0	0.0	29.0	W1

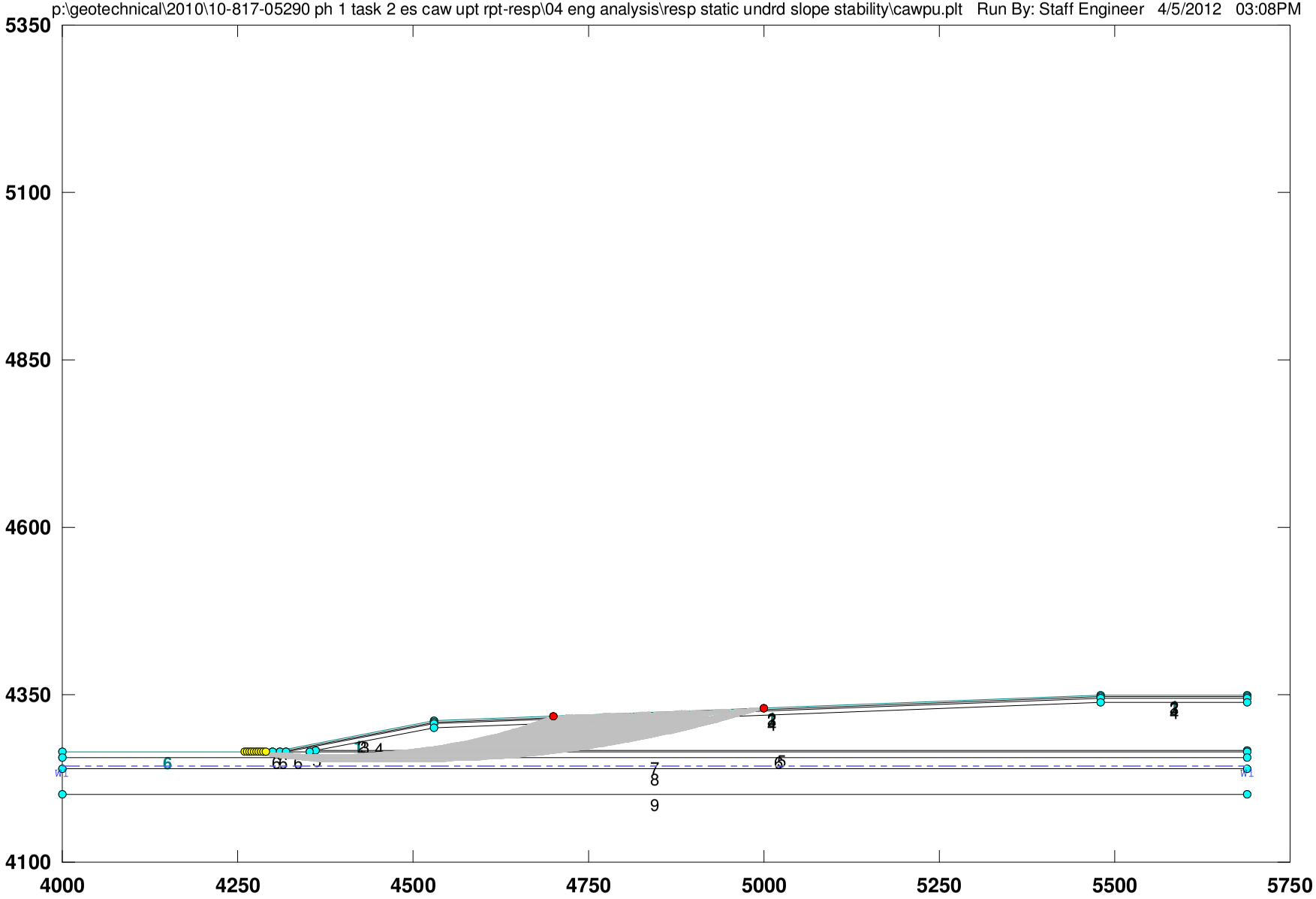
GSTABL7 v.2 FSmin=4.187

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

GSTABL7

CAW Emb 38' 5(H):1(V) side-slopes, static - Su, ShalClay unit 020212 CAWPU

p:\geotechnical\2010\10-817-05290 ph 1 task 2 es caw upt rpt-resp\04 eng analysis\resp static undrd slope stability\cawpu.plt Run By: Staff Engineer 4/5/2012 03:08PM



GSTABL7

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                                   ***  GSTABL7  ***

                          ** GSTABL7 by Garry H. Gregory, P.E. **

             ** Original Version 1.0, January 1996; Current Version 2.004, June 2003 **
                         (All Rights Reserved-Unauthorized Use Prohibited)


          ******************************************************************************
                              SLOPE STABILITY ANALYSIS SYSTEM
                     Modified Bishop, Simplified Janbu, or GLE Method of Slices.
                     (Includes Spencer & Morgenstern-Price Type Analysis)
                     Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
                     Nonlinear Undrained Shear Strength, Curved Phi Envelope,
                     Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
                     Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
          ******************************************************************************


          Analysis Run Date:        4/5/2012
          Time of Run:              03:08PM
          Run By:                   Staff Engineer
          Input Data Filename:      P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawpu.in
          Output Filename:          P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawpu.OUT
          Unit System:              English

          Plotted Output Filename:  P:\Geotechnical\2010\10-817-05290 Ph 1 Task 2 ES CAW Upt Rpt-Resp\04 ENG ANALYSIS\Resp
Static Undrd Slope Stability\cawpu.PLT




          PROBLEM DESCRIPTION:  CAW Emb 38' 5(H):1(V) side-slopes,
                                static - Su, ShalClay unit 020212 CAWPU



          BOUNDARY COORDINATES

              4 Top   Boundaries
             22 Total Boundaries


          Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
             No.        (ft)       (ft)       (ft)       (ft)     Below Bnd

              1       4000.00    4265.00    4300.00    4265.00        6
              2       4300.00    4265.00    4530.00    4311.00        1
              3       4530.00    4311.00    5480.00    4349.00        1
              4       5480.00    4349.00    5689.00    4349.00        1
              5       4300.00    4265.00    4310.00    4265.00        6
              6       4310.00    4265.00    4530.00    4309.00        2
              7       4530.00    4309.00    5480.00    4347.00        2
              8       5480.00    4347.00    5689.00    4347.00        2
              9       4310.00    4265.00    4318.00    4265.00        6
             10       4318.00    4265.00    4530.00    4307.50        3
             11       4530.00    4307.50    5480.00    4345.50        3
             12       5480.00    4345.50    5689.00    4345.50        3
             13       4318.00    4265.00    4353.00    4265.00        6
             14       4353.00    4265.00    4361.00    4267.00        5
             15       4361.00    4267.00    4530.00    4300.50        4
             16       4530.00    4300.50    5480.00    4338.50        4
             17       5480.00    4338.50    5689.00    4338.50        4
             18       4361.00    4267.00    5689.00    4267.00        5
             19       4353.00    4265.00    5689.00    4265.00        6
             20       4000.00    4256.00    5689.00    4256.00        7
             21       4000.00    4239.00    5689.00    4239.00        8
             22       4000.00    4201.00    5689.00    4201.00        9

          User Specified Y-Origin =      4100.00(ft)

          Default X-Plus Value = 0.00(ft)

          Default Y-Plus Value = 0.00(ft)
1


         ISOTROPIC SOIL PARAMETERS


           9 Type(s) of Soil


          Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
          Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
           No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
```

1	135.0	135.0	0.0	40.0	0.00	0.0	1
2	117.5	117.5	250.0	38.0	0.00	0.0	1
3	123.0	123.0	1000.0	0.0	0.00	0.0	1
4	101.0	101.0	130.0	18.0	0.00	0.0	1
5	123.0	123.0	1000.0	0.0	0.00	0.0	1
6	118.0	118.0	2000.0	0.0	0.00	0.0	1
7	120.0	120.0	0.0	34.0	0.00	0.0	1
8	121.0	121.0	2000.0	0.0	0.00	0.0	1
9	120.0	120.0	0.0	29.0	0.00	0.0	1

1

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	4000.00	4243.00
2	5689.00	4243.00

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

3800 Trial Surfaces Have Been Generated.

380 Surface(s) Initiate(s) From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X =4260.00(ft)
and X =4290.00(ft)

Each Surface Terminates Between X =4700.00(ft)
and X =5000.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 4250.00(ft)

18.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1) * *

Selected ki function = Constant (1.0)

Selected Lambda Coefficient = 1.00

Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.

Specified Tension Crack Water Force Factor = 0.000

Total Number of Trial Surfaces Attempted = 3800

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 95

Number of Trial Surfaces with Misleading FS = 12

Number of Trial Surfaces With Valid FS = 3693

Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Attempted = 2.8 %

Statistical Data On All Valid FS Values:
FS Max = 30.528 FS Min = 4.187 FS Ave = 5.070
Standard Deviation = 0.731 Coefficient of Variation = 14.42 %

((Modified Bishop FS for Critical Surface = 4.204))

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4290.001	4265.000
2	4307.829	4262.515
3	4325.710	4260.452
4	4343.636	4258.812
5	4361.595	4257.598
6	4379.578	4256.808
7	4397.574	4256.444
8	4415.574	4256.506
9	4433.567	4256.994
10	4451.543	4257.908
11	4469.494	4259.247
12	4487.407	4261.010
13	4505.274	4263.196
14	4523.084	4265.805
15	4540.827	4268.834
16	4558.494	4272.282
17	4576.074	4276.147
18	4593.558	4280.428
19	4610.935	4285.120
20	4628.197	4290.223
21	4645.333	4295.732
22	4662.334	4301.646
23	4679.190	4307.959
24	4695.893	4314.670
25	4703.504	4317.940

Circle Center At X = 4403.947 ; Y = 5017.113 ; and Radius = 760.695

*** FOS = 4.187 Theta (ki=1.0) = 5.89 ***
Lambda = 0.103

Individual data on the 35 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	10.0	822.1	0.0	0.0	0.	0.	0.0	0.0	0.0
2	7.8	2619.0	0.0	0.0	0.	0.	0.0	0.0	0.0
3	2.2	1191.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	8.0	5930.0	0.0	0.0	0.	0.	0.0	0.0	0.0
5	7.7	7996.3	0.0	0.0	0.	0.	0.0	0.0	0.0
6	17.9	26916.3	0.0	0.0	0.	0.	0.0	0.0	0.0
7	9.4	18460.8	0.0	0.0	0.	0.	0.0	0.0	0.0
8	8.0	18034.1	0.0	0.0	0.	0.	0.0	0.0	0.0
9	0.6	1423.2	0.0	0.0	0.	0.	0.0	0.0	0.0
10	18.0	47294.6	0.0	0.0	0.	0.	0.0	0.0	0.0
11	18.0	55108.4	0.0	0.0	0.	0.	0.0	0.0	0.0
12	18.0	61997.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	18.0	67945.5	0.0	0.0	0.	0.	0.0	0.0	0.0
14	18.0	72939.9	0.0	0.0	0.	0.	0.0	0.0	0.0
15	18.0	76974.1	0.0	0.0	0.	0.	0.0	0.0	0.0
16	17.9	80041.2	0.0	0.0	0.	0.	0.0	0.0	0.0
17	17.9	82137.9	0.0	0.0	0.	0.	0.0	0.0	0.0
18	12.3	57482.0	0.0	0.0	0.	0.	0.0	0.0	0.0
19	5.5	25776.7	0.0	0.0	0.	0.	0.0	0.0	0.0
20	6.9	32472.0	0.0	0.0	0.	0.	0.0	0.0	0.0
21	0.1	398.6	0.0	0.0	0.	0.	0.0	0.0	0.0
22	10.7	49630.8	0.0	0.0	0.	0.	0.0	0.0	0.0
23	17.7	77923.9	0.0	0.0	0.	0.	0.0	0.0	0.0
24	17.6	72300.9	0.0	0.0	0.	0.	0.0	0.0	0.0
25	17.5	65951.6	0.0	0.0	0.	0.	0.0	0.0	0.0
26	17.4	58899.8	0.0	0.0	0.	0.	0.0	0.0	0.0
27	17.3	51177.1	0.0	0.0	0.	0.	0.0	0.0	0.0
28	17.1	42813.0	0.0	0.0	0.	0.	0.0	0.0	0.0
29	17.0	33840.1	0.0	0.0	0.	0.	0.0	0.0	0.0
30	12.4	18803.8	0.0	0.0	0.	0.	0.0	0.0	0.0
31	4.5	5419.8	0.0	0.0	0.	0.	0.0	0.0	0.0
32	15.2	11950.6	0.0	0.0	0.	0.	0.0	0.0	0.0

33	1.5	613.1	0.0	0.0	0.	0.	0.0	0.0	0.0
34	2.5	809.5	0.0	0.0	0.	0.	0.0	0.0	0.0
35	5.1	693.1	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4290.001	4265.000
2	4307.829	4262.515
3	4325.710	4260.452
4	4343.636	4258.812
5	4361.595	4257.598
6	4379.578	4256.808
7	4397.574	4256.444
8	4415.574	4256.506
9	4433.567	4256.994
10	4451.543	4257.908
11	4469.494	4259.247
12	4487.407	4261.010
13	4505.274	4263.196
14	4523.084	4265.805
15	4540.827	4268.834
16	4558.494	4272.282
17	4576.074	4276.147
18	4593.558	4280.428
19	4610.935	4285.120
20	4628.197	4290.223
21	4645.333	4295.732
22	4662.334	4301.646
23	4679.190	4307.959
24	4695.893	4314.670
25	4703.504	4317.940

Circle Center At X = 4403.947 ; Y = 5017.113 ; and Radius = 760.695

*** FOS = 4.187 Theta (ki=1.0) = 5.89 ***
Lambda = 0.103

1

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4286.668	4265.000
2	4304.496	4262.515
3	4322.376	4260.447
4	4340.301	4258.797
5	4358.259	4257.567
6	4376.241	4256.758
7	4394.236	4256.369
8	4412.236	4256.400
9	4430.230	4256.853
10	4448.209	4257.726
11	4466.163	4259.019
12	4484.082	4260.731
13	4501.955	4262.862
14	4519.774	4265.411
15	4537.528	4268.375
16	4555.208	4271.753
17	4572.805	4275.544
18	4590.308	4279.746
19	4607.708	4284.355
20	4624.995	4289.370
21	4642.160	4294.787
22	4659.194	4300.604
23	4676.088	4306.818
24	4692.832	4313.426
25	4703.534	4317.941

Circle Center At X = 4401.880 ; Y = 5026.207 ; and Radius = 769.877

*** FOS = 4.212 Theta (ki=1.0) = 5.87 ***
Lambda = 0.103

Failure Surface Specified By 25 Coordinate Points

Point	X-Surf	Y-Surf

No.	(ft)	(ft)
1	4286.668	4265.000
2	4304.496	4262.515
3	4322.376	4260.447
4	4340.301	4258.797
5	4358.259	4257.567
6	4376.241	4256.758
7	4394.236	4256.369
8	4412.236	4256.400
9	4430.230	4256.853
10	4448.209	4257.726
11	4466.163	4259.019
12	4484.082	4260.731
13	4501.955	4262.862
14	4519.774	4265.411
15	4537.528	4268.375
16	4555.208	4271.753
17	4572.805	4275.544
18	4590.308	4279.746
19	4607.708	4284.355
20	4624.995	4289.370
21	4642.160	4294.787
22	4659.194	4300.604
23	4676.088	4306.818
24	4692.832	4313.426
25	4703.534	4317.941

Circle Center At X = 4401.880 ; Y = 5026.207 ; and Radius = 769.877

*** FOS = 4.212 Theta (ki=1.0) = 5.87 ***
Lambda = 0.103

1

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4283.334	4265.000
2	4301.162	4262.515
3	4319.042	4260.441
4	4336.966	4258.782
5	4354.923	4257.538
6	4372.904	4256.708
7	4390.899	4256.294
8	4408.899	4256.296
9	4426.894	4256.713
10	4444.875	4257.547
11	4462.831	4258.795
12	4480.754	4260.458
13	4498.634	4262.535
14	4516.461	4265.024
15	4534.226	4267.925
16	4551.918	4271.235
17	4569.530	4274.953
18	4587.052	4279.077
19	4604.473	4283.604
20	4621.785	4288.533
21	4638.979	4293.860
22	4656.044	4299.583
23	4672.974	4305.700
24	4689.757	4312.205
25	4703.604	4317.944

Circle Center At X = 4399.817 ; Y = 5035.312 ; and Radius = 779.070

*** FOS = 4.237 Theta (ki=1.0) = 5.84 ***
Lambda = 0.102

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4283.334	4265.000
2	4301.162	4262.515
3	4319.042	4260.441
4	4336.966	4258.782
5	4354.923	4257.538
6	4372.904	4256.708
7	4390.899	4256.294
8	4408.899	4256.296

9	4426.894	4256.713
10	4444.875	4257.547
11	4462.831	4258.795
12	4480.754	4260.458
13	4498.634	4262.535
14	4516.461	4265.024
15	4534.226	4267.925
16	4551.918	4271.235
17	4569.530	4274.953
18	4587.052	4279.077
19	4604.473	4283.604
20	4621.785	4288.533
21	4638.979	4293.860
22	4656.044	4299.583
23	4672.974	4305.700
24	4689.757	4312.205
25	4703.604	4317.944

Circle Center At X = 4399.817 ; Y = 5035.312 ; and Radius = 779.070

*** FOS = 4.237 Theta (ki=1.0) = 5.84 ***
Lambda = 0.102

1

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4290.001	4265.000
2	4307.851	4262.673
3	4325.748	4260.749
4	4343.683	4259.228
5	4361.648	4258.111
6	4379.634	4257.399
7	4397.632	4257.093
8	4415.631	4257.192
9	4433.625	4257.697
10	4451.602	4258.606
11	4469.554	4259.921
12	4487.472	4261.639
13	4505.346	4263.761
14	4523.168	4266.284
15	4540.929	4269.208
16	4558.620	4272.532
17	4576.231	4276.253
18	4593.754	4280.370
19	4611.180	4284.880
20	4628.500	4289.782
21	4645.705	4295.073
22	4662.786	4300.750
23	4679.735	4306.810
24	4696.544	4313.250
25	4708.493	4318.140

Circle Center At X = 4402.231 ; Y = 5056.216 ; and Radius = 799.136

*** FOS = 4.250 Theta (ki=1.0) = 5.84 ***
Lambda = 0.102

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4290.001	4265.000
2	4307.851	4262.673
3	4325.748	4260.749
4	4343.683	4259.228
5	4361.648	4258.111
6	4379.634	4257.399
7	4397.632	4257.093
8	4415.631	4257.192
9	4433.625	4257.697
10	4451.602	4258.606
11	4469.554	4259.921
12	4487.472	4261.639
13	4505.346	4263.761
14	4523.168	4266.284
15	4540.929	4269.208
16	4558.620	4272.532
17	4576.231	4276.253
18	4593.754	4280.370
19	4611.180	4284.880
20	4628.500	4289.782

21	4645.705	4295.073
22	4662.786	4300.750
23	4679.735	4306.810
24	4696.544	4313.250
25	4708.493	4318.140

Circle Center At X = 4402.231 ; Y = 5056.216 ; and Radius = 799.136

*** FOS = 4.250 Theta (ki=1.0) = 5.84 ***
Lambda = 0.102

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4280.001	4265.000
2	4297.829	4262.515
3	4315.708	4260.437
4	4333.631	4258.768
5	4351.587	4257.509
6	4369.567	4256.660
7	4387.562	4256.221
8	4405.562	4256.194
9	4423.558	4256.578
10	4441.540	4257.372
11	4459.500	4258.577
12	4477.427	4260.192
13	4495.312	4262.215
14	4513.147	4264.646
15	4530.922	4267.484
16	4548.628	4270.728
17	4566.255	4274.374
18	4583.794	4278.422
19	4601.236	4282.869
20	4618.572	4287.714
21	4635.792	4292.953
22	4652.889	4298.584
23	4669.853	4304.604
24	4686.674	4311.010
25	4703.345	4317.798
26	4703.690	4317.948

Circle Center At X = 4397.755 ; Y = 5044.392 ; and Radius = 788.237

*** FOS = 4.261 Theta (ki=1.0) = 5.81 ***
Lambda = 0.102

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4280.001	4265.000
2	4297.829	4262.515
3	4315.708	4260.437
4	4333.631	4258.768
5	4351.587	4257.509
6	4369.567	4256.660
7	4387.562	4256.221
8	4405.562	4256.194
9	4423.558	4256.578
10	4441.540	4257.372
11	4459.500	4258.577
12	4477.427	4260.192
13	4495.312	4262.215
14	4513.147	4264.646
15	4530.922	4267.484
16	4548.628	4270.728
17	4566.255	4274.374
18	4583.794	4278.422
19	4601.236	4282.869
20	4618.572	4287.714
21	4635.792	4292.953
22	4652.889	4298.584
23	4669.853	4304.604
24	4686.674	4311.010
25	4703.345	4317.798
26	4703.690	4317.948

Circle Center At X = 4397.755 ; Y = 5044.392 ; and Radius = 788.237

```
***  FOS =     4.261   Theta (ki=1.0) =     5.81  ***
                 Lambda =   0.102


          **** END OF GSTABL7 OUTPUT ****
```

Curernt GWT =	26	ft
Design GWT =	22	ft
Mw =	7.3	
PGA =	0.24	g

Table C-1.1: Cyclic Softening of "Clay-like" Soils using M = 7.3 and PGA = 0.24g

Geologic Unit No.	Predominant Soil Type	Depth (ft)	Moist Unit Weight (pcf)	Current Total Stress (psf)	Current Eff. Stress (psf)	OCR (at the time of investigation)	Preconsolidation Stress (psf)	Free-Field Su (psf)	Embank stress (psf)	Post-construction Total Stress (psf)	Post-construction Eff Stress (psf)	OCR (post-construction)	Post-construction Su (psf)	Su/eff stress	Static FS	Ts/S_u (based on FS)	K.alpha	MSF	$CRR_{M=7.5}$	rd	CSR	FS	Average FS
UNIT 4	Clay-Like	0																					
		2	118	236	236	5.3	1,251	229	6,720	6,956	6,284	1.0	1,634	0.26	2.64	0.38	0.88	1.01	0.18	1.00	0.17	1.06	1.1
		4	118	472	472	5.3	2,502	458	6,720	7,192	6,520	1.0	1,695	0.26	2.64	0.38	0.88	1.01	0.18	1.00	0.17	1.07	
		6	118	708	708	5.3	3,752	687	6,720	7,428	6,756	1.0	1,757	0.26	2.64	0.38	0.88	1.01	0.18	0.99	0.17	1.08	
		8	118	944	944	5.3	5,003	916	6,720	7,664	6,992	1.0	1,818	0.26	2.64	0.38	0.88	1.01	0.18	0.98	0.17	1.09	
		9	118	1,062	1,062	5.3	5,629	1,031	6,720	7,782	7,110	1.0	1,849	0.26	2.64	0.38	0.88	1.01	0.18	0.98	0.17	1.09	
UNIT 3	Sand-Like	10	120	1,182	1,182				6,720	7,902	7,230	1.0			2.64					0.98			
		12	120	1,422	1,422				6,720	8,142	7,470	1.0			2.64					0.97			
		14	120	1,662	1,662				6,720	8,382	7,710	1.0			2.64					0.96			
		16	120	1,902	1,902				6,720	8,622	7,950	1.0			2.64					0.96			
		18	120	2,142	2,142				6,720	8,862	8,190	1.0			2.64					0.95			
		20	120	2,382	2,382				6,720	9,102	8,430	1.0			2.64					0.94			
		22	120	2,622	2,622				6,720	9,342	8,670	1.0			2.64					0.93			
		24	120	2,862	2,862				6,720	9,582	8,910	1.0			2.64					0.92			
UNIT 2	Clay-Like	26	120	3,102	3,102	1.2	3,722	931	6,720	9,822	9,150	1.0	2,379	0.26	4.19	0.24	0.93	1.01	0.20	0.91	0.15	1.28	1.3
		28	120	3,342	3,217	1.2	3,861	966	6,720	10,062	9,265	1.0	2,409	0.26	4.19	0.24	0.93	1.01	0.20	0.91	0.15	1.27	
		30	120	3,582	3,332	1.2	3,999	1,001	6,720	10,302	9,380	1.0	2,439	0.26	4.19	0.24	0.93	1.01	0.20	0.90	0.15	1.27	
		32	120	3,822	3,448	1.2	4,137	1,035	6,720	10,542	9,496	1.0	2,469	0.26	4.19	0.24	0.93	1.01	0.20	0.89	0.15	1.27	
		34	120	4,062	3,563	1.2	4,275	1,070	6,720	10,782	9,611	1.0	2,499	0.26	4.19	0.24	0.93	1.01	0.20	0.88	0.15	1.27	
		36	120	4,302	3,678	1.2	4,414	1,104	6,720	11,022	9,726	1.0	2,529	0.26	4.19	0.24	0.93	1.01	0.20	0.87	0.15	1.27	
		38	120	4,542	3,793	1.2	4,552	1,139	6,720	11,262	9,841	1.0	2,559	0.26	4.19	0.24	0.93	1.01	0.20	0.86	0.15	1.28	
		40	120	4,782	3,908	1.2	4,690	1,174	6,720	11,502	9,956	1.0	2,589	0.26	4.19	0.24	0.93	1.01	0.20	0.85	0.15	1.28	
		42	120	5,022	4,024	1.2	4,828	1,208	6,720	11,742	10,072	1.0	2,619	0.26	4.19	0.24	0.93	1.01	0.20	0.84	0.15	1.28	
		44	120	5,262	4,139	1.2	4,967	1,243	6,720	11,982	10,187	1.0	2,649	0.26	4.19	0.24	0.93	1.01	0.20	0.83	0.15	1.29	
		46	120	5,502	4,254	1.2	5,105	1,277	6,720	12,222	10,302	1.0	2,679	0.26	4.19	0.24	0.93	1.01	0.20	0.82	0.15	1.29	
		48	120	5,742	4,369	1.2	5,243	1,312	6,720	12,462	10,417	1.0	2,708	0.26	4.19	0.24	0.93	1.01	0.20	0.81	0.15	1.30	
		50	120	5,982	4,484	1.2	5,381	1,347	6,720	12,702	10,532	1.0	2,738	0.26	4.19	0.24	0.93	1.01	0.20	0.80	0.15	1.30	
		52	120	6,222	4,600	1.2	5,520	1,381	6,720	12,942	10,648	1.0	2,768	0.26	4.19	0.24	0.93	1.01	0.20	0.79	0.15	1.31	
		54	120	6,462	4,715	1.2	5,658	1,416	6,720	13,182	10,763	1.0	2,798	0.26	4.19	0.24	0.93	1.01	0.20	0.78	0.15	1.31	
		56	120	6,702	4,830	1.2	5,796	1,450	6,720	13,422	10,878	1.0	2,828	0.26	4.19	0.24	0.93	1.01	0.20	0.77	0.15	1.32	
		58	120	6,942	4,945	1.2	5,934	1,485	6,720	13,662	10,993	1.0	2,858	0.26	4.19	0.24	0.93	1.01	0.20	0.76	0.15	1.33	
		60	120	7,182	5,060	1.2	6,072	1,520	6,720	13,902	11,108	1.0	2,888	0.26	4.19	0.24	0.93	1.01	0.20	0.75	0.15	1.33	
		62	120	7,422	5,176	1.2	6,211	1,554	6,720	14,142	11,224	1.0	2,918	0.26	4.19	0.24	0.93	1.01	0.20	0.74	0.15	1.34	
		64	121	7,664	5,293	1.2	6,351	1,589	6,720	14,384	11,341	1.0	2,949	0.26	4.19	0.24	0.93	1.01	0.20	0.73	0.14	1.35	
UNIT 1	Sand-Lke	66	121	7,906	5,410				6,720	14,626	11,458	1.0			4.19					0.72			
		68	121	8,148	5,527				6,720	14,868	11,575	1.0			4.19					0.71			
		70	121	8,390	5,644				6,720	15,110	11,692	1.0			4.19					0.71			

Notes :

1 Groundwater: Current = 26 ft bgs, Design = 22 ft bgs (see Section 2.2.3, February 15, 2011 Report)
2 Design Earthquake: Mw = 7.3 and PGA = 0.24g for Stansbury Fault, based on PSHA for 5,000-yr return period (See page 4 of this report)
3 Generalized Geologic Profile: Based on CPT99-01 through CPT99-03 and CPT-04 through CPT-06 (See Figures 5a and 5b of February 15, 2011 Report). Units 1 and 3 are primarily "sand-like" materials and were not analyzed for cyclic softening
4 Unit Weights: See Table 3.1 of February 15, 2011 Report, Unit weight of embankment =112 pcf
5 Overconsolidation ratio (OCR): Unit 4 - varies from 2 to 8 (used 5.3), Unit 2 - varies 1 to 2 (used 1.2). These OCR values were previously used in other calculations, as presented in February 15, 2011 report
6 Preconsolidation Pressure: Effective Stress x OCR
7 Free-Field Undrained Shear Strength: Use SHANSEP Model with m = 0.26 and n = 0.79 (see Section 3.2 and page 15 of December 13, 2005 Report)
8 Embankment Stress: Embankment height varies from 45 to 85 feet, Used 60 ft as average
9 Postconstruction Total Stress = Current Total Stress + Embankment Stress (plane-strain condition, no reduction with depth)
10 Postconstruction Effective Stress = Current Effective Stress + 90% of Embankment Stress (assumed 90% consolidation in all Units at the end of 2 years after construction)
11 Postconstruction OCR: 1.0 (assumed for all Units)
12 Postconstruction Undrained Shear Strength: Use SHANSEP Model with m = 0.26 and n = 0.79 and Postconstruction Effective Stress and respective OCR
13 Static Factor of Safety: min FS = 2.64 through Unit 4 and 4.19 through Unit 2 (using undrained strengths), see Appendix C of this report
14 Average Static Shear Stress Due to Embankment Loading/Undrained Shear Strength (Ts/Su) = (1/FS) for static limit equilibrium analysis
15 Static Shear Stress Correction Factor (K. alpha): Computed using Equation 9 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
16 Magnitude Scaling Factor (MSF): Computed using Equation 6 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
17 Cyclic Resistance Ratio (CRR): Computed using Equation 12 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
18 Stress Reduction Coefficient (rd): Computed using Idriss (1999) and Idriss-Boulanger (2004) - see Page 3-1 and 3-2 of UC-Davis Report No. UCD/CGM-04/01 dated December 2004
19 Cyclic Stress Ratio (CSR) = 0.65 * PGA * (postconstruction total stress / postconstruction effective stress) * rd
20 Factor of Safety for Cyclic Softening = CRR/ CSR

Curernt GWT =	26	ft
Design GWT =	22	ft
Mw =	7.3	
PGA =	0.28	g

Table C-1.2: Cyclic Softening of "Clay-like" Soils usign M = 7.3 and PGA = 0.28g

Geologic Unit No.	Predominant Soil Type	Depth (ft)	Moist Unit Weight (pcf)	Current Total Stress (psf)	Current Eff. Stress (psf)	OCR (at the time of investigation)	Preconsolidation Stress (psf)	Free-Field Su (psf)	Embank stress (psf)	Post-construction Total Stress (psf)	Post-construction Eff Stress (psf)	OCR (post-construction)	Post-construction Su (psf)	Su/eff stress	Static FS	Ts/S_u (based on FS)	K.alpha	MSF	$CRR_{M=7.5}$	rd	CSR	FS	Average FS
UNIT 4	Clay-Like	0																					
		2	118	236	236	5.3	1,251	229	6,720	6,956	6,284	1.0	1,634	0.26	2.64	0.38	0.88	1.01	0.18	1.00	0.20	0.91	0.92
		4	118	472	472	5.3	2,502	458	6,720	7,192	6,520	1.0	1,695	0.26	2.64	0.38	0.88	1.01	0.18	1.00	0.20	0.92	
		6	118	708	708	5.3	3,752	687	6,720	7,428	6,756	1.0	1,757	0.26	2.64	0.38	0.88	1.01	0.18	0.99	0.20	0.93	
		8	118	944	944	5.3	5,003	916	6,720	7,664	6,992	1.0	1,818	0.26	2.64	0.38	0.88	1.01	0.18	0.98	0.20	0.93	
		9	118	1,062	1,062	5.3	5,629	1,031	6,720	7,782	7,110	1.0	1,849	0.26	2.64	0.38	0.88	1.01	0.18	0.98	0.20	0.94	
UNIT 3	Sand-Like	10	120	1,182	1,182				6,720	7,902	7,230	1.0			2.64					0.98			
		12	120	1,422	1,422				6,720	8,142	7,470	1.0			2.64					0.97			
		14	120	1,662	1,662				6,720	8,382	7,710	1.0			2.64					0.96			
		16	120	1,902	1,902				6,720	8,622	7,950	1.0			2.64					0.96			
		18	120	2,142	2,142				6,720	8,862	8,190	1.0			2.64					0.95			
		20	120	2,382	2,382				6,720	9,102	8,430	1.0			2.64					0.94			
		22	120	2,622	2,622				6,720	9,342	8,670	1.0			2.64					0.93			
		24	120	2,862	2,862				6,720	9,582	8,910	1.0			2.64					0.92			
UNIT 2	Clay-Like	26	120	3,102	3,102	1.2	3,722	931	6,720	9,822	9,150	1.0	2,379	0.26	4.19	0.24	0.93	1.01	0.20	0.91	0.18	1.09	1.1
		28	120	3,342	3,217	1.2	3,861	966	6,720	10,062	9,265	1.0	2,409	0.26	4.19	0.24	0.93	1.01	0.20	0.91	0.18	1.09	
		30	120	3,582	3,332	1.2	3,999	1,001	6,720	10,302	9,380	1.0	2,439	0.26	4.19	0.24	0.93	1.01	0.20	0.90	0.18	1.09	
		32	120	3,822	3,448	1.2	4,137	1,035	6,720	10,542	9,496	1.0	2,469	0.26	4.19	0.24	0.93	1.01	0.20	0.89	0.18	1.09	
		34	120	4,062	3,563	1.2	4,275	1,070	6,720	10,782	9,611	1.0	2,499	0.26	4.19	0.24	0.93	1.01	0.20	0.88	0.18	1.09	
		36	120	4,302	3,678	1.2	4,414	1,104	6,720	11,022	9,726	1.0	2,529	0.26	4.19	0.24	0.93	1.01	0.20	0.87	0.18	1.09	
		38	120	4,542	3,793	1.2	4,552	1,139	6,720	11,262	9,841	1.0	2,559	0.26	4.19	0.24	0.93	1.01	0.20	0.86	0.18	1.09	
		40	120	4,782	3,908	1.2	4,690	1,174	6,720	11,502	9,956	1.0	2,589	0.26	4.19	0.24	0.93	1.01	0.20	0.85	0.18	1.10	
		42	120	5,022	4,024	1.2	4,828	1,208	6,720	11,742	10,072	1.0	2,619	0.26	4.19	0.24	0.93	1.01	0.20	0.84	0.18	1.10	
		44	120	5,262	4,139	1.2	4,967	1,243	6,720	11,982	10,187	1.0	2,649	0.26	4.19	0.24	0.93	1.01	0.20	0.83	0.18	1.10	
		46	120	5,502	4,254	1.2	5,105	1,277	6,720	12,222	10,302	1.0	2,679	0.26	4.19	0.24	0.93	1.01	0.20	0.82	0.18	1.11	
		48	120	5,742	4,369	1.2	5,243	1,312	6,720	12,462	10,417	1.0	2,708	0.26	4.19	0.24	0.93	1.01	0.20	0.81	0.18	1.11	
		50	120	5,982	4,484	1.2	5,381	1,347	6,720	12,702	10,532	1.0	2,738	0.26	4.19	0.24	0.93	1.01	0.20	0.80	0.18	1.12	
		52	120	6,222	4,600	1.2	5,520	1,381	6,720	12,942	10,648	1.0	2,768	0.26	4.19	0.24	0.93	1.01	0.20	0.79	0.17	1.12	
		54	120	6,462	4,715	1.2	5,658	1,416	6,720	13,182	10,763	1.0	2,798	0.26	4.19	0.24	0.93	1.01	0.20	0.78	0.17	1.13	
		56	120	6,702	4,830	1.2	5,796	1,450	6,720	13,422	10,878	1.0	2,828	0.26	4.19	0.24	0.93	1.01	0.20	0.77	0.17	1.13	
		58	120	6,942	4,945	1.2	5,934	1,485	6,720	13,662	10,993	1.0	2,858	0.26	4.19	0.24	0.93	1.01	0.20	0.76	0.17	1.14	
		60	120	7,182	5,060	1.2	6,072	1,520	6,720	13,902	11,108	1.0	2,888	0.26	4.19	0.24	0.93	1.01	0.20	0.75	0.17	1.14	
		62	120	7,422	5,176	1.2	6,211	1,554	6,720	14,142	11,224	1.0	2,918	0.26	4.19	0.24	0.93	1.01	0.20	0.74	0.17	1.15	
		64	121	7,664	5,293	1.2	6,351	1,589	6,720	14,384	11,341	1.0	2,949	0.26	4.19	0.24	0.93	1.01	0.20	0.73	0.17	1.16	
UNIT 1	Sand-Lke	66	121	7,906	5,410				6,720	14,626	11,458	1.0			4.19					0.72			
		68	121	8,148	5,527				6,720	14,868	11,575	1.0			4.19					0.71			
		70	121	8,390	5,644				6,720	15,110	11,692	1.0			4.19					0.71			

Notes :

1 Groundwater: Current = 26 ft bgs, Design = 22 ft bgs (see Section 2.2.3, February 15, 2011 Report)
2 Design Earthquake: Mw = 7.3 and PGA = 0.24g for Stansbury Fault, based on PSHA for 5,000-yr return period (See page 4 of this report)
3 Generalized Geologic Profile: Based on CPT99-01 through CPT99-03 and CPT-04 through CPT-06 (See Figures 5a and 5b of February 15, 2011 Report). Units 1 and 3 are primarily "sand-like" materials and were not analyzed for cyclic softening
4 Unit Weights: See Table 3.1 of February 15, 2011 Report, Unit weight of embankment =112 pcf
5 Overconsolidation ratio (OCR): Unit 4 - varies from 2 to 8 (used 5.3), Unit 2 - varies 1 to 2 (used 1.2). These OCR values were previously used in other calculations, as presented in February 15, 2011 report
6 Preconsolidation Pressure: Effective Stress x OCR
7 Free-Field Undrained Shear Strength: Use SHANSEP Model with m = 0.26 and n = 0.79 (see Section 3.2 and page 15 of December 13, 2005 Report)
8 Embankment Stress: Embankment height varies from 45 to 85 feet, Used 60 ft as average
9 Postconstruction Total Stress = Current Total Stress + Embankment Stress (plane-strain condition, no reduction with depth)
10 Postconstruction Effective Stress = Current Effective Stress + 90% of Embankment Stress (assumed 90% consolidation in all Units at the end of 2 years after construction)
11 Postconstruction OCR: 1.0 (assumed for all Units)
12 Postconstruction Undrained Shear Strength: Use SHANSEP Model with m = 0.26 and n = 0.79 and Postconstruction Effective Stress and respective OCR
13 Static Factor of Safety: min FS = 2.64 through Unit 4 and 4.19 through Unit 2 (using undrained strengths), see Appendix C of this report
14 Average Static Shear Stress Due to Embankment Loading/Undrained Shear Strength (Ts/Su) = (1/FS) for static limit equilibrium analysis
15 Static Shear Stress Correction Factor (K. alpha): Computed using Equation 9 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
16 Magnitude Scaling Factor (MSF): Computed using Equation 6 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
17 Cyclic Resistance Ratio (CRR): Computed using Equation 12 in Boulanger-Idriss (2007), ASCE Geotechnical Journal, Vol. 133, No. 6, p. 645
18 Stress Reduction Coefficient (rd): Computed using Idriss (1999) and Idriss-Boulanger (2004) - see Page 3-1 and 3-2 of UC-Davis Report No. UCD/CGM-04/01 dated December 2004
19 Cyclic Stress Ratio (CSR) = 0.65 * PGA * (postconstruction total stress / postconstruction effective stress) * rd
20 Factor of Safety for Cyclic Softening = CRR/ CSR