DOCKETED	
Docket Number:	24-OPT-05
Project Title:	Corby Battery Energy Storage System Project
TN #:	259882
Document Title:	Volume 2 App 2 Project Description Appendices Part 2
Description:	N/A
Filer:	Doug Urry
Organization:	Tetra Tech
Submitter Role:	Applicant Consultant
Submission Date:	11/4/2024 1:41:17 PM
Docketed Date:	11/4/2024

APPENDIX 2-E: PRELIMINARY GEOTECHNICAL REPORT

CONFIDENTIAL

Draft Geotechnical Report

Corby BESS Project
Solano County, California

Prepared for

NEXTera ENERGY RESOURCES

Prepared By:

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RRC Project No. GE23006031
September 18, 2024

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NextEra Energy Resources, LLC
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Attn: Mr. Michael Benson

Re: Draft Geotechnical Report
Corby BESS Project
Solano County, California
RRC Project No. GE23006031

RRC Power & Energy, LLC (RRC) has completed the authorized subsurface exploration and geotechnical engineering evaluation for the proposed Corby BESS Project. The purpose of the geotechnical engineering study was to explore and evaluate the subsurface conditions at various locations on the site and develop geotechnical design and construction recommendations for the project. The attached report contains:

A description of our findings from the field exploration and laboratory-testing program;
Our engineering interpretation of the results with respect to the project characteristics; and
Our geotechnical site development and foundation design recommendations for the proposed project.

Should you have any questions concerning this Draft Geotechnical Report, please do not hesitate to contact us. Your business is always appreciated.

Sincerely,

RRC Power & Energy, LLC

Abbas Taghavi, Ph.D., P.E. (WA)
Geotechnical Engineer

REVISION HISTORY

Revision	Description	Prepared/Submitted by	Reviewed by	Issued Date
0	Draft Geotechnical Report	Abbas Taghavi	Hongbin Huo	11/03/2023
1	Updated Draft Geotechnical Report	Abbas Taghavi	Hongbin Huo	01/31/2024

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DRAFT GEOTECHNICAL REPORT

1.0 INTRODUCTION

RRC completed the authorized subsurface exploration and geotechnical engineering evaluation for the proposed Corby BESS Project. The site is located Northeast of the city of Vacaville in Solano County, California. The approximate boundaries of the site are shown in Figure 1 within Appendix A.

The purpose of this investigation and report is to:

- Explore subsurface soil, bedrock, and groundwater conditions;
- Conduct field and laboratory tests to characterize the subsurface soil and bedrock properties at selected locations across the site;
- Install piles and conduct load tests to assess geotechnical parameters; and
- Provide geotechnical engineering recommendations for the design and construction of foundation systems and access roadways.

The recommendations contained in this report are based upon our field and laboratory test results, engineering analyses, experience with similar soil conditions, and our understanding of the proposed project. We also reviewed published geological maps and groundwater level data obtained from published well logs.

2.0 PROPOSED CONSTRUCTION

This project will consist of construction of a power generation facility, a battery energy storage system, an underground collection system, associated equipment and private access roadways within the project site. We assume that the equipment in the substation and Battery Energy Storage System (BESS) including the inverters and containers will be supported on driven steel piles, most likely wide flange beams (W-beam) and direct embedment foundations.

We anticipate pile embedment depths of about 6 to 10 feet below existing ground surface. We assume the minimum center-to-center spacing to be 5 feet or more between adjacent steel piles. Private-access roadways will most likely be surfaced aggregate road base materials overlying compacted subgrade soils to support construction and vehicular traffic loads during and after construction. We also assume that there will be minimal site grading in the BESS area.

3.0 SITE EXPLORATION

RRC conducted a subsurface exploration program within the project site. RRC's surface exploration consisted of drilling geotechnical boreholes and excavating test pits within the project area. We conducted in-situ testing and collected undisturbed and disturbed samples for laboratory testing. Additionally, we installed and tested a total of 8 driven H-piles at 4 selected locations across the project site. The section 3.1 describes our site exploration program in detail.

RRC's subsurface exploration program consisted of:

- Drilled 9 borings within the proposed BESS area;
- Drilled one boring within the proposed Substation;

- Excavated one test pits within BESS and one test pit within Substation;
- Collected 4 undisturbed (tube) samples, 4 bulk samples for laboratory Thermal Resistivity (TR) testing;
- Conducted 4 Electrical Resistivity (ER) surveys including one test within Substation and 3 tests within BESS locations;
- Collected 4 samples from borings for corrosion testing;
- Collected one bulk sample for laboratory California Bearing Ratio (CBR) testing;
- Performed pile load testing on 8 installed test piles.

Figures 1 through 4 within Appendix A consist of maps for various boreholes and excavations, sample locations, in-situ testing, and Figure 9 in Appendix A shows geophysical measurement locations. A summary of subsurface exploration is provided within Table A1 within Appendix A.

The results of laboratory testing performed on soil samples obtained from the site during the site exploration are included in the boring logs in Appendix A and are thoroughly presented in Appendix B. The results of the field and laboratory geophysical tests are presented in Appendix C. The recommended design of deep foundations and results of pile load tests are presented in Appendices D and E, respectively.

3.1 Field Exploration and Testing

RRC geologist staked the borehole locations with a handheld GPS with accuracy of approximately 15 feet using the coordinates proposed by RRC and approved by NextEra. Drill crews under RRC's direction drilled the borings within 10 feet of each staked location.

RRC completed a total of 10 conventional geotechnical borings during the period of September 13, 2023 through September 14, 2023. We drilled borings B-01 through B-09 to depths ranging between 20.5 and 40.5 feet below ground surface within the BESS area, one substation boring B-10 was drilled to depth of 40 feet at the proposed substation location. Under the direction of RRC's field representative, the total of 2 test pits were excavated to the target depth of 10 feet, or to refusal depth on September 28, 2023. We collected bulk samples between 1 and 4 feet below ground surface.

We drilled boreholes with CME 75 truck mounted rigs. Drillers used continuous flight hollow-stem augers. During the field operations, the drill crews observed groundwater levels. Following the completion of the drilling operations, the drillers backfilled each bore with soil cuttings in accordance with state regulations. When needed, we added bentonite chips to supplement the soil cuttings. The test pits were excavated using a backhoe.

For thermal resistivity sampling locations, the drillers used augers and multiple shallow bores to collect bulk samples. We also collected relatively undisturbed tube samples at selected boreholes using tube samplers.

A summary of geographic latitude and longitude coordinates, and depth of each boring and test pit location drilled or excavated as part of the subsurface exploration program is presented in

Table A1 within Appendix A. Figure 2 within Appendix A shows the boring locations on a topographic map.

Standard Penetration Test (SPT) samplers obtain disturbed soil samples. RRC documented each penetration resistance value in accordance with *ASTM D1586: Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*. This test consists of driving the sampler into the ground with a 140-pound auto hammer free-falling 30 inches. The number of blows required to advance the SPT sampler 18 inches is counted and recorded, with the sum of the blows to drive the last 12 inches referred to as the standard penetration resistance value (N-value). Results of the field tests at select depth intervals are shown on the logs of boring under the “Field Data” column, where SPT N-values are preceded by the letter “N” (raw N value). The hammer efficiency is 60% or higher and accounted for in the design recommendations. Each soil sample from the SPT samplers collected in the field was visually classified, placed in plastic bags to preserve moisture content, and labeled as to location and depth. All SPT samples were arranged in core boxes and transported to the laboratory facility in Round Rock, Texas for further analysis.

RRC obtained relatively undisturbed samples in cohesive soils utilizing a hydraulically advanced 3-inch (OD) diameter stainless steel, thin-walled tube (Shelby) sampler in accordance with *ASTM D1587: Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes*. The soil samples obtained using Shelby tube were tested for consistency utilizing a pocket-sized penetrometer which provides an estimate of the undrained shear strength and unconfined compressive strength for fine-grained soils. The penetrometer reading is included on the logs of boring preceded by the letter “P.” Readings in excess of 4.5 tons per square feet (tsf), if any, indicate that the capacity of the device has been exceeded. Sufficient material from the lower end of the Shelby tube was removed for visual classification purposes. Both ends of the Shelby tube were sealed using plastic caps and secured with duct tape to prevent moisture loss in the sample. Sample location and depth were labeled on the outside surface of the tube. The Shelby tube samples were stored vertically onsite and transported to the laboratory facility in Round Rock, Texas for further analysis.

Modified California (MC) split-spoon samplers were used to obtain relatively undisturbed samples of both cohesive and non-cohesive soils. RRC’s field geologist and/or field engineer directed drill crews to obtain these samples at intermittent locations. The Modified California sampler is similar to the SPT sampler except that the Modified California sampler obtains 2.5-inch diameter samples in a steel sleeve. The Modified California sampler is driven like the SPT sampler and the blow counts are counted and recorded similarly. Results of the field tests are also shown on the Logs of Boring under the “Field Data” column and are preceded by the letter “N”. When conducting an analysis of the stratum, a correlation is needed to convert the Modified California N-value to an SPT N-value. For 3-inch outside diameter MC sampler, conversion factor is in the range of 0.45 (Lacroix and Horn, 1773 method) to 0.65 (Burmister, 1948 method). Based on the experience, RRC uses a conversion factor of 0.5 when performing soil density analysis of N-values for the Modified California sampler. A plastic cap was placed on each end of the Modified California sample to seal the sample’s moisture. The sample was labeled as to location and depth. All

Modified California samples were arranged in core boxes and transported to our laboratory facility in Round Rock, Texas for further analysis.

RRC classifies soils in general accordance with the Unified Soil Classification System (USCS); *ASTM D2488: Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The soil classification symbols appear on the boring logs and are briefly described in Appendix A. Bedrock materials were classified in general accordance with the general notes for rock classification included as part of *ASTM D5878: Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes.*

RRC's field geologist prepared field logs for each boring. The logs of boring contain classification of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples.

The project engineer/geologist reviewed all the field logs, soil samples, and laboratory test data to make appropriate modifications to the logs of boring as necessary. Final Logs of Boring and laboratory testing completed to date are provided in Appendix A. The logs of boring describe the strata encountered, their approximate thickness, SPT results, soil classifications, the various depths at which the samples were obtained, as well as the presence of groundwater.

3.1.1 Standpipe Piezometers

RRC installed seven temporary standpipe piezometers within the project area. We installed these to an approximate depth of 13 feet below the ground surface. Summary of these standpipe piezometer depth and screen depth are shown in Table A3 within Appendix A.

Each piezometer consisted of 1-inch diameter, schedule 40 PVC pipe. The piezometers include a bottom screen/slotted section that is 5 or 10-feet long and backfilled with sand that extends to about one-foot above the screened portion. The remainder of the piezometer is 10-feet of PVC riser pipe that extends approximately 2 feet above ground. We used bentonite to backfill the upper portion of the piezometer. Groundwater levels measured to date within the temporary piezometers are discussed in Section 4.4 of this report. Additional groundwater levels may be collected by RRC and/or Client's representative during subsequent field visits. RRC assumes that piezometer will be decommissioned by the Client during construction.

3.1.2 Percolation Test

RRC performed percolation tests at depths between 3 to 5 feet below ground surface near 5 selected locations on September 15, 2023. The test locations are plotted in Figure 4 within Appendix A and the testing results are provided within Appendix B. It should be noted that these infiltration tests were conducted on in-situ soils at indicated depths. Detailed design evaluation and implementation of drainage plan is the responsibility of the civil designer. Additional tests may be needed if the infiltration basin's bottom grade changes as a result of earthwork (cut and fill) or variable soil conditions are encountered during construction. Based on the results of percolation tests, the infiltration rates range between 0.01 and 0.03 inches per hour.

3.1.3 Geophysical Investigation

RRC completed a surface geophysical survey for this project between September 25 and September 27, 2023 at 4 selected locations, as shown within Appendix A. Electrical resistivity (ER) tests were performed in the field at selected locations. Thermal resistivity (TR) tests were performed in the laboratory on remolded and undisturbed samples. The detailed description of each of these geophysical methods is presented in section 4.6. The results of geophysical investigations were presented within Appendix C.

3.2 Laboratory Tests

The soil samples were returned to the laboratory, examined by the project engineer/geologist, and applicable laboratory testing was assigned on selected soil samples. RRC commissioned Beyond Engineering & Testing for laboratory tests. Beyond E&T performed laboratory tests in general accordance with ASTM standards and locally accepted practices. The following laboratory methods of analyses were generally utilized, where sample quality allowed:

- Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System): ASTM D2487 – for classifying mineral and organo-mineral soils for engineering purposes based on particle-size characteristics, liquid limit, and plasticity index;
- Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass: ASTM D2216 – for laboratory determination of the water content by mass of soil, rock and similar materials where loss of water results in the reduction in mass by drying;
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils: ASTM D4318 – for laboratory determination of the liquid limit, plastic limit, and the plasticity index of soils;
- Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-μm) Sieve: ASTM D1140 – for determination of the amount of material finer than the No. 200 (75-μm) sieve by washing;
- Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis: ASTM D6913 – for separation of particles into size ranges and quantitative determination of the mass of particles in each range;
- Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort: ASTM D698 – for determining the relationship between molding content and dry unit weight of soils compacted (compaction curve);
- Standard Test Methods for Unconfined Compressive Strength of Cohesive Soils: ASTM D2166 – for determining the unconfined compressive strength of cohesive soil in the undisturbed, remolded, or compacted condition, employing strain-controlled application of the axial load;
- Standard Test Methods for One Dimensional Swell or Collapse of Soils: ASTM D4546, Method C – for measuring load-induced compression subsequent to wetting-induced swell or collapse deformation;

- Standard Test Methods for CBR (California Bearing Ratio) of Laboratory-Compacted Soils: ASTM D1883 – for determining the CBR of pavement subgrade, subbase, and base course materials from laboratory-compacted soils;
- Standard Test Method for Measurement of Soil Resistivity using the Two-Electrode Soil Box Method: ASTM G187 – for measurement of soil resistivity for soil samples removed from the ground, for assessment and control of corrosion of buried structures;
- Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing: ASTM G51 – for measuring the pH of a soil in corrosion testing;
- Sulfate and Chlorides Content: ASTM D4327 – for determination of inorganic anions in reagent water, surface water, ground water, and finished drinking water, and
- Standard Test Method for Determination of Thermal Conductivity of Soil and Rock by Thermal Needle Probe Procedure: ASTM D5334 – for measurement of the thermal resistivity of undisturbed soil samples.

4.0 SUBSURFACE CONDITIONS

4.1 Geology

The geologic interpretations contained herein are based on available geological maps and literature, and review of the logs of excavation as part of this study. The Corby BESS project site is located within the California Trough Section of the Pacific Border Physiographic Province (Reference 1). The project site is in the southern portion of the Putah Plain, a relatively flat, broad area that stretches from the Coast Range, down to the Sacramento River Delta.

The surficial quaternary deposits in this site contain Holocene aged alluvial fan deposits. These were deposited in a river system environment about 11,000 to 3 million years ago and can range from 20 feet to 100 feet thick (Reference 2).

The Preliminary geologic map of the Lodi 30' X 60' quadrangle, California, California Geological Survey, Preliminary Geologic Maps PGM-09-04, scale 1:100,000 (Reference 3) indicates the surficial deposits of the site consisting of the following geologic units of the listed geologic time periods.

4.1.1 Quaternary Period

- **Alluvial Fan Deposits (Qpf):** Unconsolidated to semi-consolidated alluvium deposits containing of pale brown to gray-brown fine-grained silts and clays, inter-bedded with sand and gravel lenses.

4.1.2 Hazards

The project site is located 7 miles away from near 3 thrust faults striking westward. There is also a system of right lateral strike slips faults about 16 miles away. No large earthquakes (M<4) have been located near these faults, however, there have been some larger earthquakes with a magnitude between 4.0-6.0 in the surrounding area (Reference 2).

Figure 5 within Appendix A shows the project site plotted on the geologic map.

4.2 Subsurface Stratigraphy

As indicated on the boring and test pit logs, the soil stratigraphy at the site generally consisted of 6 to 11 inches of topsoil underlain by either clay or sand layers. Depth of topsoil may vary in other parts of the project area not evaluated in the logs. Bedrock was not encountered in any of the borings. The soil layers can generally be described in Table 4.2.1.

Table 4.2.1. Subsurface Profile

Approximate Depth (ft)	Soil Type	Layer Description
0 – 0.5	Topsoil	Soft to Medium stiff, with varying amounts of clay
0.5 – 3	Fat Clay/ Lean Clay	Soft to very stiff
3 – 9	Lean Clay	Medium stiff to hard, with varying amounts of sand
9 – 13	Poorly Graded Sand with Clay	Loose to medium dense, with varying amounts of clay
> 13	Lean Clay	With varying amounts of sand

The above descriptions are general and depth ranges are approximate because boundaries between different strata are seldom clear and abrupt in the field. Detailed Boring Logs for locations drilled as part of this study, which present stratum descriptions, soil classifications, types of sampling used, laboratory test data, and additional field data are presented in Appendix A. The Boring Log Key, defining the terms and descriptive symbols used on each log of boring, is also presented in Appendix A.

4.3 Laboratory Test Results

RRC obtained the service of Beyond Engineering & Testing, LLC to conduct laboratory tests. Laboratory tests were carried out to classify the soil, and measure the soil moisture-density relationship, soil strength, swell/collapse potential and thermal resistivity among others.

Laboratory test results indicate the native soils possess in-situ moisture contents in the range from about 16% to 30% with an average of 21%.

The native soils have Plasticity Indices (PI) ranging from 18 to 40 with an average PI of 29. Clay soils with a PI less than 15 are generally considered to exhibit a low expansive potential provided their moisture contents are stable. High plasticity clay soils with a PI greater than 25 are generally considered to exhibit a high expansive potential if their moisture contents are allowed to change significantly.

The in-situ dry unit weights of the native soils at the project site range from 95 to 111 pounds per cubic foot (pcf) with an average of 105 pcf. The in-situ total unit weights of the native soils range from 115 to 131 pcf with an average of 126 pcf.

The compressive strength of the soil ranges from 2,520 to 7,820 pounds per square foot (psf) with an average of 5,170 psf.

We performed standard Proctor tests (moisture/density relationships) to determine the maximum dry unit weight and optimum moisture content in accordance with ASTM D698. We also conducted Atterberg Limits on these samples to assess soil type. A summary of the test results is presented in Table 4.3.1.

Table 4.3.1. Summary of Standard Proctor and Atterberg Test Results

Sample Location	Depth (feet)	Material Type	Liquid Limit (%)	Plasticity Index	Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)
B-02	1 to 4	CL	40	24	110.8	14.5
B-04	1 to 4	CL	32	18	113.4	13.1
B-06	1 to 4	CL	38	25	108.3	14.3
B-10	1 to 4	CL	36	21	108.0	16.2

Notes: pcf = pounds per cubic foot; CL = Lean Clay.

Results of Unconfined Compressive Strength (UC) tests performed on soil samples are presented in Table 4.3.2. These tests were performed in accordance with ASTM D2166 on relatively undisturbed samples.

Table 4.3.2. Summary of Soil Unconfined Compressive Strength Test Results

Sample Location	Depth (feet)	Material Type	In-situ Dry Unit Weight (pcf)	In-situ Moisture Content (%)	Unconfined Compressive Strength, q_u (psf)
B-01	6	CL	111.1	18.6	7,820
B-07	2.5	CH	99.9	24.4	2,520

Notes: pcf = pounds per cubic foot; psf = pounds per square foot; CL = Lean Clay; CH = Fat Clay.

Results of swell testing performed in accordance with ASTM D4546, on selected undisturbed soil samples, are summarized in Table 4.3.3. It should be noted that laboratory test results are based on “as received” moisture content at the time of the geotechnical investigation.

Table 4.3.3. Summary of Swell/Collapse Tests

Sample Location	Depth (feet)	Material Type	Liquid Limit (%)	Plasticity Index	Dry Density (pcf)	Moisture Content (%)	Surcharge Load (psf)	Swell Strain (%)
B-03	3.5	CH	53	40	95.1	21.1	350	0.6
B-05	3.5	CL	40	25	105.6	18.3	350	0.2

Notes: psf = pounds per square foot; Negative swell strain indicates collapse during test; CH = Fat Clay; CL = Lean Clay.

We performed California Bearing Ratio (CBR) tests on select samples. The test specimens soaked for 96 hours prior to the CBR load test. A summary of the test results is summarized in Table 4.3.4. Design CBR values represent strength at 95% compaction relative to the maximum dry density as determined by ASTM D698.

Table 4.3.4. Summary of CBR Test Results

Sample Location	Depth (feet)	Material Type	Design Dry Unit Weight (pcf)	Design CBR (%)
B-02	1 to 4	CL	105	2.1

Notes: pcf = pounds per cubic foot; psi = pounds per square inch; CL = Lean Clay.

Results of water-soluble sulfate and chloride testing, minimum resistivity, and pH performed on shallow soil samples are summarized in Table 4.3.5.

Table 4.3.5. Summary of Sulfate and Chloride Contents, Minimum Resistivity, and pH of Soil

Sample Location	Depth (feet)	Material Type	Chloride Contents (% by weight)	Sulfate Contents (% by weight)	Minimum Resistivity (ohm-cm)	pH
B-01	1 to 3	CH	0.0002	0.0018	871	8.5
B-04	1 to 3	CL	0.0006	0.0023	2,077	8.2
B-07	1 to 3	CL	0.0006	0.0023	804	8.4
B-10	1 to 3	CL	0.0020	0.0037	1,273	8.0

Notes: CH = Fat Clay; CL = Lean Clay

Graphical test results of laboratory testing results along with a summary of laboratory testing are presented in Appendix B.

4.4 Groundwater Conditions

Based upon the information obtained from the borings drilled, test pits excavated as part of this study, and review of published well log records and the piezometer reading, it is RRC's opinion that static groundwater level may impact the design and construction of foundations 7 feet or deeper below the ground surface.

We encountered groundwater in all borings at depths ranging from 7.5 to 15 feet below grade during drilling or immediately after the completion of drilling. Measurements from shallow groundwater piezometers were also collected to provide an accurate estimate of long-term groundwater levels. Groundwater observations for each boring location are also presented on Table A1 in Appendix A. Upon completion of each borehole, we backfilled in accordance with applicable state and local regulations; therefore, subsequent groundwater measurements are not available.

We reviewed published water well logs for Solano County, State, available from California Department of Water Resources (Reference 4). Historical water well logs within the project site indicate that static groundwater levels were reported to be between 13 (Well ID No. WCR2015-010062 and WCR2015-009493 in year 2015) and 315 feet below the ground surface. The well locations presented on Table A3 in Appendix A are plotted on Figure 10 in Appendix A.

RRC installed temporary piezometers at one location as shown in Figure 12 within Appendix A. Piezometers water levels are summarized in Table A3. In the piezometer, groundwater was encountered at 9 to 11.1 feet below ground surface during site investigation in September and October 2023.

We note that the majority of the water wells were installed to deep aquifers below typical foundation depth and indicate piezometric or static groundwater level within those deep aquifers only. The static water levels from the deep wells do not always provide useful groundwater information for shallow aquifers or perched water tables near foundation depths that should be considered in foundation design.

It is imperative to note that the short-term groundwater level observations presented in this report are not an accurate evaluation of groundwater levels. This report is not to be interpreted as a comprehensive groundwater study. Groundwater levels are highly dependent on climatic and hydrologic conditions before and after construction, and site development including irrigation demands, drainage and other factors. The presence of water during construction can vary relative to this report. It is understood that the presence of groundwater may influence certain construction activities and long-term performance of foundations and pavements. If a detailed groundwater study is desired, a groundwater hydrologist should be retained to perform these services.

4.5 Geohazard Assessment

The following table represents geologic or physical hazards to the project. Within each subject, we address the level of risk associated with the particular hazard relative to this project.

Table 4.5.1. Geohazard Assessment

Geohazard	Site Risk (Low/Moderate/High)	Comment
Swell/Shrink Soils	Moderate to High	Based on swell test results and correlations from PI values of soil samples, surficial expansive soil has been encountered at some of borings and test pit locations. Detailed discussion presented in Section 6.1.
Collapsible Soils	Low	From laboratory test results, the collapse potential of samples was between 0.2 and 0.6%, which is considered low (Reference 5).
Shallow Bedrock	Low	Shallow bedrock was not encountered in any of the explored locations.
Frost Penetration Depth	Low	The frost penetration depth at the project site is approximately 6 inches (Reference 6).
Shallow Groundwater	Moderate to High	Shallow groundwater was encountered between 7.5 and 15 feet below groundwater. Foundation design considerations and dewatering systems may be required.
Corrosive Soils (Concrete)	Low	Based on publicly available maps, surficial soils are expected to exhibit a low to moderate potential for corrosion of concrete. However, laboratory test results show a low potential for corrosion of concrete. Refer to Section 6.3 on corrosivity of soil, the laboratory test results in Appendix B and publicly available data presented in Figure 6 within Appendix A.
Corrosive Soils (Steel)	Moderate to High	Although chloride content from laboratory testing shows a non-aggressive category of corrosion of steel (refere to Section 6.3), based on the resistivity from laboratory test results in Appendix B and publicly available data presented in Figure 7 within Appendix A, surficial soils are expected to exhibit a moderate to high potential for corrosion of unprotected steel.
Earthquake (Ground Rupture)	Low	Low hazard zone in the U.S.
Earthquake (Seismicity)	Low	Low hazard zone in the U.S.
Earthquake (Liquefaction)	Low	Low hazard zone in the U.S., and liquefaction analysis showed low risk.

Geohazard	Site Risk (Low/Moderate/High)	Comment
Flooding	Low	Proposed project area is generally within a minimal flood zone (Zone X) as shown in Figure 8 within Appendix A. The design of flood-proofing as well as scour/erosion protection is the responsibility of the project Civil Engineer or Hydrology Engineer
Settlement	Low to Moderate	Anticipated total settlement is estimated on the order of about 1.5 to 2.0 inches under normal operating loading condition.
Slope Stability	Low	The project area was generally flat to gently rolling with some steeper slopes near streams/drainages across the site. Final site grading plan should be reviewed to evaluate the large-scale slope instability issues for the site.
Subsidence (Caves/Karst)	Low	The risk of finding karstic features within the project area boundaries is low. No karst/void features reported in public data (Reference 7). RRC logs of boring did not encounter any voids or karst features.

4.6 Geophysical Properties

RRC also completed Earth Electrical Resistivity (ER) surveys at 4 locations across the site. We conducted ER surveys using the Wenner four-pin method. The ER locations are mapped on Figure 9 in Appendix A. The ER survey methodologies are presented in Section 4.6.1.

RRC obtained the services of Beyond Engineering & Testing (Beyond) to perform in-situ and laboratory thermal resistivity tests. These test locations were selected by RRC and approved by NextEra. Thermal resistivity test methodologies are presented in Section 4.6.2. These locations are mapped on Figure 9 in Appendix A.

Table A1 within Appendix A presents a summary of geographic latitude and longitude coordinates where the geophysical surveys were performed and the soil samples for laboratory thermal resistivity testing were collected.

4.6.1 Earth Electrical Resistivity Surveys

RRC performed a total of 4 field electrical resistivity (ER) surveys using a using a SuperStingTM R8/IP, DCMemory Earth Resistivity Meter using the Wenner 4-pin array method in accordance with the ASTM D 6431 and IEEE std-81. The ER surveys represent three surveys at BESS (B-3, B-5, B-10), and one survey at the substation (TP-SUB). The ER survey locations are mapped on Figure 9 included in Appendix A. Each survey consists of two perpendicular arrays, both centered at the same location. At BESS locations, we used a maximum ‘a’ spacing of 400 feet; at the substation location, we used a maximum ‘a’ spacing of 600 feet. Results of the ER surveys are presented in Appendix C.

Interpretation of the ER surveys is beyond the scope of this study and should be performed by the electrical design team.

4.6.2 Thermal Resistivity of Soils

We collected bulk samples from soil cuttings of boreholes between 1 to 4 feet below existing site grade. We also collected undisturbed tube samples from boreholes from 1 foot below existing site grade. The in-situ and laboratory thermal resistivity tests were performed in accordance with ASTM D5334.

We conducted thermal resistivity tests on remolded samples targeting 90% compaction of the maximum dry density as determined by ASTM D698 at optimum moisture content. Thermal resistivity samples were then tested at a series of moisture contents from as-received moisture content to 0% moisture content. The thermal resistivity testing on tube samples were tested at a series of moisture contents from "as-received" moisture content to 0% moisture content. The result is a thermal resistivity dry-out curve. Thermal resistivity test results are presented within Appendix C.

Interpretation of the thermal resistivity tests results is beyond the scope of this study and should be performed by the design team.

5.0 STATIC PILE LOAD TESTING

5.1 Pile Information

After reviewing boring logs, evaluating the available test data, and considering the soil variability across the project site, RRC selected 4 locations for pile load tests. The approximate locations of static pile load testing are shown in Figure 13 included in Appendix A. The purpose of the static load testing program was to obtain site-specific performance data for design of pile foundations.

5.2 Test Pile Driving

RRC installed a total of 8 test piles at 4 selected locations, using a PD-10 pile driver between October 3, 2023 and October 9, 2023. We installed additional piles and repeated some of the tests, particularly compression testing, in locations B-5, B-8, and B-10 to ensure the quality of the collected data is acceptable. We installed two piles with different embedment depths at each of 4 selected locations. The test piles are spaced about 10 feet apart from each other, to reduce interference. The proposed pile installation and testing locations are based on the anticipated project layout. If the current plan changes, it is recommended that RRC be notified to perform further geotechnical investigations and provide recommendations.

The test piles consisted of wide flange sections W6X9 steel piles. Table E1 within Appendix E presents a *Summary Table of Test Pile Locations and Installation Records*. RRC assumes the production piles will be installed using the same or similar model of driving machine with similar energy output. Note that the difficult/refusal pile driving condition in this project is defined as less than 1-inch of pile movement over one minute of drive time using a PD-10 Pile Driver. During pile installation in native soil, RRC pile installation refusal was not encountered.

RRC recorded the time that was used to advance each pile to its final embedment depth. The summary of pile installation data is presented in the *Summary Table of Test Pile Locations and Installation Records* within Appendix E. Following installation of the piles, RRC performed axial compression/tension, and lateral load testing of the test piles between October 3 and 10, 2023.

5.3 Pile Load Test Procedures and Equipment

Following pile installation, RRC conducted pile load tests on each test pile. Axial and lateral loads were applied to the test piles using an excavator. We made connections to the test piles using shackles with a 12-ton load capacity and flange clamps with 5-ton to 15-ton load capacity. RRC used a calibrated Motionics Bluetooth digital dial indicator to measure deflections. We also used calibrated tension load cells to measure loads. RRC personnel managed these tests and documented the load cells and gauges.

We applied horizontal loads for the lateral load tests at approximately 2 feet above ground surface of each test pile in the strong-axis direction of the piles. RRC used two dial gauges with 2-inch travel to measure deflections. One dial gauge was placed 3 inches above the ground surface and another dial gauge placed 2 feet above ground surface to measure the lateral deflection.

For axial compression/uplift testing, an Enerpac pull cylinder and a telehandler were utilized to generate the specified compression/uplift loads. Loading was applied at the top center of each pile. RRC measured axial deflections using two dial gauges, attached to the two sides of the flanges at approximately 6 inches above the ground surface.

RRC proposed the magnitude and sequence of test load steps, based on the anticipated pile design loads and deflection requirements, which was approved by NextEra. Table 5.3.1 summarized the maximum applied test load as well as the test pile information.

Table 5.3.1. Static pile load testing summary

Testing Type	Axial Compression	Axial Tension	Lateral Test
Max. applied load (lbs)	25,000	10,000	10,000
Height of applied load (in)	--	--	48
Pile embedment (ft)	6, 7, 8, 10	6, 7, 8, 10	6, 7, 8, 10
Number of tested piles	8	8	8

5.4 Pile Load Test Results

The results of the 8 test piles were used to evaluate the vertical and lateral support for the driven piles. The lateral and axial load test data, including graphical plots of the load tests, are presented in Appendix E.

- When determining the uplift and compressive pile skin friction values, RRC used a 0.5-inch deflection of the pile as the criteria for failure.
- For lateral design, a calibration model using LPILE Version 2022-12.007 was used to simulate the lateral load test results and determine LPILE parameters.

The recommended design parameters for pile foundation design are presented in Appendix D.

6.0 GEOTECHNICAL RECOMMENDATIONS

We assume that the battery energy storage systems such as containers, inverter, and generators will be supported on steel driven pile foundation systems.

The extent and location of site grading is currently unknown. RRC anticipates that the proposed foundations will bear on/in native soils with finish grade near current site grade with minimum slope stability impacts. RRC should be retained to review the civil drawings and cross-sections for BESS areas and other critical areas along the proposed roadways. This will allow us to evaluate the need for additional studies such as slope stability analyses. If site grade is significantly changed at structure locations, we can assess whether our original recommendations apply. RRC's geotechnical recommendations presented in this report should be verified when information on the foundation design and site grading become available.

6.1 Expansive Soil Consideration

A critical consideration for design of shallow foundation systems in the project location is mitigation of high-plasticity clay soils. Potential Vertical Rise (PVR) is an estimate of the potential of an expansive soil to swell from its current state if the clay is allowed to absorb additional moisture. This method is based upon empirical correlation using Atterberg Limits test results and, laboratory swell/collapse potential test, and as such should only be considered as providing estimates of potential movements/heave and not precise prediction of the movements that may occur.

The surficial clay soils at this site exhibit a moderate to high expansion potential when subjected to variation in moisture contents. The depth of moisture variation is estimated to be about 3 feet for the site (active zone). Shrink/swell movement for the proposed foundations at the project is calculated to be approximately 1.5 inches using the Texas Department of Transportation (TxDOT) method Tex-124-E, assuming the in-situ moisture content based on available testing results. The estimated PVR of about 1.5 inches is above the commonly accepted value of 1 inch by structural engineers and designers for a shallow foundation system, floor slab-on-grade system or stiffened beam and slab foundation. For this project, we recommend limiting the PVR to 1 inch or less to prevent structural damage and/or operational distress that can be caused by volume changes in clay soil.

Estimated PVR values are based upon anticipated typical changes in soil moisture content from a dry to wet condition; however, soil movements in the field depend on the actual changes in moisture content. Thus, actual soil movements could be less than that calculated if little soil moisture variations occur or exceed the estimated values if actual soil moisture content changes are greater than anticipated. This condition is often the result of excessive droughts, flooding, "perched" groundwater infiltration, poor surface-drainage, excessive irrigation adjacent to building foundations, and/or leaking irrigation lines or plumbing. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reducing potential shrink-swell movement. It is imperative that proper drainage be maintained during construction and throughout the life of the structure for adequate shallow foundation performance.

As stated above, the PVR values are only indications of the order of magnitude of shrink/swell movement potential. To reduce the PVR values to about 1 inch, removal of expansive fat clay materials to a minimum depth of 2 feet should be performed within the entire building footprint and critical structures with low PVR tolerance. These materials should be replaced with non-expansive select fill materials meeting the criteria outlined in sections 7.3 and 7.4 of this report. The over-excavation area should extend a minimum lateral distance of about 1 foot beyond the edge of the footings.

The placement of fill material in areas where competent bedrock is exposed after cut is not needed. However, exposed shale bedrock shall be preserved from being disturbed, freezing, swelling and/or desiccation. Any degraded shale material directly below foundation footprint shall be over-excavated to the competent layer and replaced with non-expansive fill material.

In general, foundations at the project site are expected to undergo some degree of vertical movement that can potentially cause minor distress. The design and construction of the proposed structures should take such factors into consideration.

6.2 Foundation Considerations and Recommendations

The proposed project site appears suitable for the proposed BESS project construction. Driven pile foundations can be used to support containers, inverters and transformers and other heavily loaded (axial or lateral) structures. Spread footings and/or mat foundations are acceptable for lightly loaded structures such as equipment pads. A summary of anticipated conditions that will require particular attention in the design and construction is presented below:

- The project is located within a low seismic region. The results of liquefaction analysis showed that the risk of settlements due to liquefaction is low.
- During this phase of site investigation, expansive soils (high PI clays) were encountered at borings and test pits below existing ground surface across the project site. The subgrade includes expansive soil that will exhibit shrink and swell behavior, which shall be considered by the foundation designer. Shrinkage and swell of soils would be anticipated seasonally as we assume BESS development does not significantly change the moisture conditions on the project site. Based on testing results and experience from previous nearby projects, the depth of moisture variation is estimated to be about 3 feet (active zone).
- RRC's boreholes and historical published well logs indicate that groundwater will likely effect foundations within 7 feet of the existing ground.
- Excavation contractors and/or underground utility installers should consider performing test pits or probing tests to evaluate proper means and methods for advancing excavations. Potential caving/sloughing of loose/soft and dry soils within narrow and shallow utility trenches may require sidewalls of trenches to be sloped in order to properly install underground utilities. Excavated trench bottoms should be thoroughly cleaned prior to bedding materials and cable placement and backfilling. For shallow spread footings bearing on native soil, it is anticipated that excavations may be advanced with conventional earth moving equipment to cover the potential scour depth and avoid bearing on unsuitable topsoil.
- We assume minimal cut and fill for the proposed project. We expect that driven piles will bear on native soils with minimal slope stability impacts. The final grading plan is recommended to be provided to review in the final design.

- We recommend the design engineers to take site flooding/scour into proper account during civil, structural and electrical design.

Detailed foundation design and construction recommendations are outlined in subsequent sections of this report.

6.2.1 Driven Pile Foundation

The BESS area may use steel pile foundations. Pile lengths will be dictated by compression and overturning resistance. The structural engineer should determine the length of the steel piles to meet axial and lateral loading requirements.

Lean Clay, Fat Clay, and Sandy Lean Clay soils are present within the anticipated embedment depths of the proposed H-piles. We did not encounter difficult pile driving conditions within 10 feet below existing ground surface. We caution that we did not encounter boulders, areas of high gravel content, or intermittent zones of cementation. Such materials can cause refusal or misdirect piles while driving.

Due to soil disturbance during construction, potential scour, seasonal moisture change of clay soil and soil deformation after lateral loading, we recommend the upper 3 feet or scour depth, whichever is deeper, to be ignored during the axial load analysis and the soil properties in the upper 12 inches to be reduced during the lateral load analysis. A foundation monitoring program can be established and maintained throughout the project life. Particular attention and maintenance should be performed after storm events.

The axial compression and uplift capacity of driven piles included the skin friction developed along the perimeter of the pile. The perimeter of a wide flange pile was taken as twice the sum of the flange width and web depth (i.e., the "box" perimeter). The ultimate uplift and compression unit skin friction is based on the results of the axial load testing. RRC applied a factor of safety of 1.5 to calculate the H-pile allowable skin friction. RRC also applied a factor of safety of 2.0 in the allowable end bearing capacity value. RRC provides design parameters for general soil condition encountered across the proposed BESS area. Based on SPT data from geotechnical investigation and the results of pile load testing, the soil near B-03 had shown a lower load bearing capacity than the rest of the site. Therefore, the area close to B-03 (within 300 feet) was identified as Weak Zone, and the rest of the site is called General Zone. The summary of allowable uplift skin friction, allowable compressive skin friction and allowable end bearing capacity for H-piles within the proposed BESS site are presented in Table D1.1 within Appendix D.

RRC's opinion that it is appropriate to include H-pile end bearing capacity for this project for partial soil plugging condition, to enhance pile axial capacity under compression. The allowable pile end bearing under compression can be applied to a 50% of the box area of the H-pile provided that the pile lengths are at least 6 feet embedded into the subsurface materials.

In order to calculate the lateral load response of pile foundations utilizing LPILE program, input parameters are evaluated using modeled lateral response of the testing piles. LPILE analyses are performed by applying the test loads that resulted in significant deflection near ground surface for piles with different embedment depths to calibrate the LPILE input parameters to match the lateral pile load test results. For lateral pile analysis, we recommend the soil within the upper 12 inches of pile embedment, or within the anticipated scour depth, whichever is deeper, be modeled as soft/loose soil with low strength. The summary of recommended LPILE parameters for lateral analysis of driven H-piles are presented in Table D1.2 within Appendix D.

Based on the soils encountered in our study, this site has a moderate to high risk for the swell/shrink. Expansive soil (fat clay) was encountered during this site investigation in most boring locations, particularly at shallow depths and can be influencing the pile capacity due to soil movements. As noted in section 6.1, the depth of moisture variation is estimated to be about 3 feet for the site (active zone). Based upon pile load testing and laboratory swell/expansion testing performed as part of this study, uplift forces on steel piles due to expansive clay can be estimated using a skin friction value of 400 psf (no factor of safety is applied) acting on the "Box" area of the pile for the pile length in fat clays within the active zone which is about 3 feet below existing ground surface. The estimated uplift force induced by soil swelling is calculated based on empirical correlations and pile load testing results of test piles installed within "active zone". It is RRC's opinion that the above uplift force due to clay swell/shrink movement should not be combined with the external superstructural uplift force on the pile. The uplift forces could be reduced if potentially expansive clay soils are removed and replaced with structural fill or non- to low-expansive soils. Another alternative to designing for the uplift loads due to clay expansion on piles/piers is placing friction reducing material (i.e. epoxy or bitumen coating) to the portion of the pile embedded in the expansive clays within the moisture change zone, i.e., upper 4 feet below grade. If friction reducer is used, additional pile load testing should be conducted to verify the performance of the friction reducing material.

We note that swelling can occur if water is allowed to pond on clay soils with high plasticity. Conversely, soil shrinkage can occur if clay soil desiccates. High plasticity clay may experience shrinkage during periods of dry weather as moisture evaporation occurs at the ground surface and the groundwater table drops. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reducing potential shrink-swell movement.

Further guideline on the design and construction of driven piles are presented in Reference 8.

6.2.2 Drilled Shafts

Structure elements with heavy axial loads and/or large overturning moments may utilize drilled pier foundations. Pier lengths will likely be dictated by overturning resistance. Allowable end bearing pressures and allowable skin friction values at the substation locations are presented in Appendix D.

Allowable end bearing pressures and allowable skin frictions utilize a factor of safety of 3 and 2.5, respectively. Skin friction values should be reduced by 25% when calculating pull-out resistance, where applicable. Settlement associated with drilled piers is anticipated to be on the order of about ½ to 1 inch. Piers should have a minimum diameter of 1½ feet. The structural engineer should determine the length of the drilled piers to satisfy axial and lateral loading.

Based upon our experience with similar clay soils and results of laboratory swell/expansion testing performed as part of this study, uplift forces on drilled piers can be computed using the following equation:

U = 8*d (kips), where d is pier diameter in feet

This equation is based on the assumption that expansive clay soils within the upper 3 feet (the active zone) of the existing ground surface contribute to uplift forces on the drilled piers. The 3 feet zone represents the depth to which naturally occurring seasonal changes in clay soil water content would be expected to occur. The uplift forces could be reduced if potentially expansive clay soils are removed and replaced with structural engineered fill or non- to low expansive soils. Additional changes in the moisture content of the clay soils surrounding drilled piers caused by poor drainage may cause an increase in the active zone depth leading to larger uplift forces. Therefore, it is imperative that the design provides for proper drainage around the structures during construction and throughout the life of the structure. Uplift forces should be resisted by a combination of dead-load and skin friction contribution of the clay soils below a depth of 3 feet.

It is RRC's opinion that the above uplift force due to clay swell/shrink movement should not be combined with the external superstructural uplift force on the pile. An alternative to designing for the uplift loads due to clay expansion on piles/piers is placing friction reducing material such as bitumen coating and plastic wrap. Friction reducing material should only be applied to the portion of the pile which will be embedded in the expansive clays within the moisture change zone, i.e., upper 5 feet below grade.

Lateral load analysis may be performed using the LPILE computer program. LPILE uses a p-y curve finite difference technique for predicting the soil-structure interaction and response. RRC provides detailed LPILE parameters within Appendix D to evaluate drilled piers under lateral loads.

LPILE parameters are calculated based on soil profiles shown on the logs of borings. Cohesion (C) for clay soils is correlated to blow counts (i.e., SPT N values). For this, RRC has used the relationship of q_u (tsf) and SPT N proposed by Terzaghi & Peck, as referenced in Naval Facilities Engineering Command (NAVFAC 7.01). Strain factor ε_{50} is estimated based on the cohesion. ε_{50} can also be used as a default value in the LPILE program. Internal friction angle (ϕ) for sand and/or silt soils is calculated based on its soil types and several correlation equations recommended by NAVFAC and Federal Highway Administration (FHWA) and a minimum value is selected for the ϕ.

Vertical steel reinforcement to resist tensile loads caused by uplift forces should extend the full length of the pier shaft. Additional reinforcement required by structural demands for axial compressive loads, lateral loads, or minimum reinforcement required by design codes should also be satisfied.

Further guideline on the design and construction of drilled shafts are presented in Reference 9.

6.2.3 Helical Pile and Ground Screw Foundations

An alternative deep foundation system that could be considered for support of the Substation and BESS structures is a helical pile and ground screw foundation. Helical piles and ground screws come in a multitude of diameters with varying helices amounts and sizes. Since helical piles and ground screws vary greatly from manufacturer to manufacturer, determining a pile design capacity without knowing what will be installed is difficult.

The actual design of the piles including the pile capacity, spacing, helix diameter(s), shaft length, bracket attachment and configuration, and shaft diameter should be performed by an experienced helical pile contractor or structural engineer. An experienced helical pile contractor should review the data from this report to assess the equipment required to achieve the minimum length and capacity. We recommend load testing be conducted at the site to confirm anticipated capacities and to finalize design information. The load tests should be performed at the minimum pile length designed.

RRC does not recommend using vertically installed helical piles to resist lateral loads without approved lateral load test data, as these types of foundations are typically designed to resist axial loads. Helical piles installed at a batter may be used to resist lateral loads. Typically, helical piles can be installed to a batter of up to 45 degrees from the horizontal. Only the horizontal component of the allowable axial load should be considered to resist the lateral loading and only in the direction of the batter.

RRC should be consulted to review load test data, and a representative of the geotechnical engineer should be present to observe test and production helical pile installation to verify that proper bearing materials have been encountered during installation. For any piles that encounter refusal conditions prior to the recommended minimum length, predrilling may be required to achieve the recommended depth. However, this site is considered suitable for helical piles with no anticipated pile refusal based on the information obtained from the Substation and BESS borings. We recommend a load test be performed to confirm pile capacity.

6.2.4 Shallow Foundation Systems

Lightly loaded structures, including inverter/transformer skids within proposed BESS area may use shallow foundations. We assume that these structures will bear near the existing ground surface. Continuous footings should be 12 inches or greater in width. Anticipated settlement of the foundations will be on the order of about 1 inch when the foundation uses the full allowable bearing capacity.

In areas where very soft to soft clay soils are encountered beneath shallow foundation bearing elevation, small lightly loaded structures within these facilities may utilize continuous or pad footings bearing on a minimum of 2-feet structural fill materials. We recommend structural fill materials be compacted to a minimum of 98% of the maximum dry density as outlined by ASTM D698, moisture conditioned within 2% of optimum moisture content. The over-excavation area should extend a minimum lateral distance of about 1 foot beyond the edge of the footings.

If the equipment is supported by earthen pads, the side slopes for the earthen pad should be no steeper than 2H:1V (Horizontal to Vertical) slope. The edge of the earthen equipment pad shall have a minimum of 15 feet clearance from surrounding structures or construction loads.

For reinforced concrete slabs bearing at finished grade, we recommend over-excavation of foundation subgrade soils to a minimum of 3 feet below the finished grade and replacement of with non-expansive structural fill material or flowable fill (controlled low strength material) having compressive strength of at least 150 psi. The non-expansive structural fill material should be in accordance with Section 7.3 of this report, and contain 6% or less of finer material passing No. 200 sieve. All structural fill materials below the slab-on-grades should be moisture conditioned within 2% of optimum moisture content and compacted to a minimum of 98% of the maximum dry density as determined by ASTM D698. When bearing on this earthen pad, the mat foundation may use a net allowable bearing pressure 2,500 psf.

For shallow foundation systems, net allowable bearing pressures, which include a factor of safety of 3, outlined in Table 6.2.1 can be used for the substation locations, provided the above design guidelines are followed.

Table 6.2.1. Recommended Soil Parameters for Structural Design of Footing and Mat Foundations

Parameter	Design Value
Design Groundwater Level, ft.	7
Average Unit Weight, pcf	115
Modulus of Subgrade Reaction of Select Fill Pad, pci	50*
Undrained Shear Strength**, psf	1,200
Angle of Internal Friction, degrees	--
Friction Coefficient at Foundation Base	0.35
Net allowable bearing pressure for Strip or Continuous Footings (psf), for foundation width of 1 foot or greater, at a min. bearing depth of 1 foot	2,000
Net allowable bearing pressure for Square or Pad Footings (psf), for foundation width of 2.0 feet or greater, at a min. bearing depth of 1 foot	2,500

Notes: pcf = pounds per cubic foot; psf = pounds per square foot; pci = pounds per cubic inch.
* For a 1 ft. x 1 ft. Plate.
** Average design S_u for soils below 1 foot (foundation bearing elevation).

RRC recommends that a qualified representative of a geotechnical engineer observe shallow foundation excavations in this area to assess the need for any over-excavation and re-compaction and/or replacement.

Shallow foundations should be adequately reinforced and proportioned to resist swell/uplift forces associated with the near surface clay soils. For shallow foundation systems founded on compacted fill material at project site, net allowable bearing pressures, which include a factor of safety of 3, outlined in Table 6.2.1 can be used.

Other design and construction recommendations outlined in the ACI Design Manual should be followed. It is imperative that proper drainage be maintained during construction and throughout the life of Substation structures to provide for adequate shallow foundation performance.

6.3 Corrosivity of Soils

RRC commissioned Project X Corrosion Engineering to conduct chemical tests on selected soil samples. Water-soluble sulfate and chloride test results are presented in Appendix B and summarized in Table 4.3.5. The test results indicate the soils in the upper 3 feet possess a risk for sulfate exposure and corrosion potential to concrete ranging from "Negligible". Foundation concrete should be designed in accordance with *ACI 318: Building Code Requirements for Structural Concrete and Commentary*.

Minimum Soil Box Electrical Resistivity and pH testing results are presented in Table 4.3.5 of this report. Soil Box Electrical Resistivity results indicate soils within the upper 10 feet exhibit "Moderately Corrosive" to "Corrosive" electrical characteristics with regards to galvanic corrosion of steel, according to Table 6.3.1. For chlorides, the test results indicate "Non-Aggressive" corrosion potentials to steel. Cathodic protection for buried metal pipe should be designed by a qualified corrosion engineer.

Table 6.3.1. Effect of Soil Box Electrical Resistivity on Corrosion (Reference 10)

Aggressiveness	Resistivity in ohm-cm
Very Corrosive	< 700
Corrosive	700 – 2,000
Moderately Corrosive	2,000 – 5,000
Mildly Corrosive	5,000 – 10,000
Non-Corrosive	> 10,000

6.4 Lateral Earth Pressures

Lateral earth pressures will apply in soil strata. The proposed foundations will be designed to resist all lateral movements; therefore, the "at rest" lateral earth pressure will apply. The following "at rest" equivalent fluid pressures are recommended in Table 6.4.1. The lightweight range is more conservative and necessary for the "at rest" and "passive" conditions. The heavier weights are more conservative for the "active" condition.

Table 6.4.1. Equivalent Fluid Pressures

Soil Type	Condition		Equivalent Fluid Pressure (psf/ft)
Clay Soils ϕ=20°, γ_t=110 to 115 pcf	At Rest	k_o= 0.66	72
	Active	k_a= 0.49	56
	Passive	k_p= 2.04	224
Sand Soils ϕ=30°, γ_t=115 to 120 pcf	At Rest	k_o= 0.48	56
	Active	k_a= 0.32	38
	Passive	k_p= 3.12	359

We note that for clay soils, the angle of internal friction is adjusted to correct for shear versus a lower friction angle. Theoretical correlation for the at-rest coefficient of clays is a property of Plasticity Index. For large variations in clay plasticity, RRC can provide alternate k_o coefficients. Because we expect that there will only be one foundation design, we expect that the lateral earth pressure will be dictated by the sandy condition.

Passive and active earth pressure resistance will only mobilize after significant movement of the foundation. The passive case occurs where a structural element tends to move into the soil mass. The active case occurs when the element tends to move away from the soil mass. Both cases are applicable for unrestrained foundation elements.

The earth pressure values listed in the previous table do not include safety factors. We recommend a minimum safety factor of 2.0 be applied when using passive earth pressure for lateral load resistance. Surcharge loads should also be considered where appropriate. The values apply only to cases where the ground surface is level. We should be contacted to provide suitable values for cases where the ground surface is sloped. Similarly, if a structure is submerged below water, then the earth pressures change dramatically and require a different analysis.

6.5 Seismic Design

RRC provides seismic design using 2022 California Building Code (CBC) (Reference 11). Based on Boring Logs data, we recommend using Site Class D for stiff soil. The seismic design parameters were computed using the Applied Technology Council *Seismic Design Maps,* which is a web-based application program (Reference 12). Table 6.5.1 summarizes recommended seismic parameters to be used in the design:

Table 6.5.1. Recommended Seismic Parameters

Parameter	Recommended Value
Site Soil Classification	D
S_S – Mapped Spectral Response Acceleration at Short Period (0.2-Second)	1.190 g
S_1 – Mapped Spectral Response Acceleration at 1-Second Period	0.428 g
S_{DS} – Design Spectral Response Acceleration Parameter at Short Periods (0.2-Second)	0.812 g
S_{D1} – Design Spectral Response Acceleration Parameter at 1-Second Period	N/D*
SDC – Seismic Design Category	N/D*
F_a – Site Amplification Factor at 0.2-Second	1.024
F_v – Site Amplification Factor at 1.0-Second	N/D*
PGA_M – Site Modified Peak Ground Acceleration	0.547 g

* N/D: Not determined-(S1>0.2): A site-specific response analysis should be performed by the designer to determine this parameter.

7.0 FOUNDATION CONSTRUCTION CRITERIA

7.1 Site Preparation

Prior to construction, we recommend adequate positive drainage be provided to maintain a relatively dry condition in the area of proposed construction. This will be very important if any work is attempted during periods of prolonged rainfall or heavy snowfall followed by warmer days. Ponding of water in the areas of construction should be avoided. Winter conditions can also impact the construction process. Newly placed fill should not be placed on frozen subgrade and frozen material should not be used for fill.

Site preparation should begin by removing surface vegetation and major root systems within the foundation areas. Topsoil or organics shall not be allowed underneath proposed facilities, structures or permanent pavement. Deleterious materials should be placed in non-structural areas or removed from the sites. Proper slopes meeting federal and state OSHA requirements should be maintained.

During excavation and other earthwork for foundation, every effort should be made to avoid disturbing subgrade materials at the planned foundation bearing elevation. Soil disturbance means not damaging or disturbing native soils unless prescribed, such as over-ex, scarification, compaction, etc. When the subgrade is disturbed, the resulting surface should be re-compacted to achieve a minimum compaction of 98% of the maximum dry density as determined by ASTM D698 and moisture conditioned within 2% of optimum moisture content or drier. A qualified representative of a geotechnical engineer should verify the bearing capacity and stability of the subgrade prior to foundation installation.

7.2 General Site Grading Fill Specifications

Native sand and clay material will be used as general site grading fill. Materials with significant organics should not be used under any structures. After site clearing and grubbing, the general fill should be placed in loose lifts not exceeding 12 inches in thickness and compacted to a minimum of 90% of the ASTM D698 maximum dry unit weight. If the general site grading is located below proposed pavement, foundations or equipment pads, then other compaction requirements apply. See following sections for details.

Both cut and fill slopes shall be no steeper than 3 horizontal to 1 vertical. Fill areas shall be cleared of all vegetation and debris, recompacted to a minimum of 90% of the ASTM D698 maximum dry unit weight, proof-rolled and inspected by the grading inspector and geotechnical engineer prior to the placing of fill. The proof-rolling can be conducted with a fully loaded water truck or dump truck to assess the presence of soft areas and the need for remedial measures, if any. Proof-rolling acceptance standards include no rutting or pumping greater than 1.5 inches. Typically, 8-inch-thick compacted lifts are a maximum, but if a contractor can complete thicker lifts and it can be verified that full densification occurs throughout the lift, then lifts up to 12-inches are possible.

7.3 Structural Fill Specifications

Structural fill material beneath foundations, where required, should consist of a non-expansive, well-graded material with sufficient binder for compaction purposes and meet the requirements of the standard specification of California Department of Transportation (Caltrans) Aggregate Base

Class 2 or better (Reference 13). RRC's intent is to make Structural Fill interchangeable with flexible road base, where convenient.

Structural fill should be compacted to at least 95% of maximum dry density from ASTM D1557 or 98% of maximum dry density from ASTM D698. The structural fill should be moisture conditioned within 2% of optimum moisture content. Typically, 8-inch lifts are a maximum, but if a contractor can complete thicker lifts and it can be verified that full densification occurs throughout the lift, then lifts to 12-inches are possible.

7.4 Native Soils as Select Fill below Foundations

RRC understands the importance of using native soils whenever feasible. The following specifications allow reasonable native soil reuse while maintaining structural requirements for end bearing capacity and settlement. Modification of unsuitable foundation soils shall consist of over-excavation and replacement with any of the following materials:

All soils that possess the following properties qualify as Select Fill that may be used under foundations: maximum plasticity index of 15 and a maximum liquid limit of 40, and classify as SC-SM, SC, Sandy CL, well-graded GC, and well-graded GM.

Select Fill placement under foundations should be limited to two-feet thick. Deeper replacement must be approved by a Geotechnical Engineer in order to assess settlement potentials for that specific location. Otherwise, use Structural Fill.

When dealing with subgrade pumping, rutting, or moisture, and the remediation has a maximum thickness of 12-inches, then the excavated soils may be scarified and reused to complete the remediation. Deeper remediation requires either Select Fill or Structural Fill.

All reused and Select Fill soils used under foundations shall be compacted to a minimum of 98% of the maximum dry density as determined by ASTM D698 and shall be moisture conditioned within 3% of optimum moisture content.

7.5 Substation Structures

This section provides construction recommendations and specifications related to shallow and deep foundations for structures. This section is intended to apply for all electrical substation and transmission line structures. If future, more specific geotechnical studies for those facilities are conducted, then disregard this section and refer to the recommendations in those more specific studies.

7.5.1 Shallow Foundation Construction

The following construction criteria and general guidance should be observed during foundation construction:

- The engineer's qualified representative should observe all foundation excavations to assess proper bearing materials are present at foundation bearing elevation in accordance with the recommendations given herein, and to assess the need for densification of the subgrade materials.

- Care should be taken to protect the exposed soils from being disturbed, freezing or desiccation.
- The foundation excavation should be sloped sufficiently to create internal sumps for runoff collection and removal. Foundation excavations subject to rainfall and possible deterioration from accumulated water should be protected using a protective "mud-slab" (lean concrete). If surface runoff water or groundwater seepage accumulates at the bottom of the foundation excavation, it should be collected and removed and not allowed to adversely affect the quality of the bearing surface.
- The foundation excavations should be checked for size and cleaned prior to the placement of reinforcing steel. Take precautions during the placement of reinforcement and concrete to prevent the loose material from falling into the excavation.
- If the equipment is supported by earthen pads, the side slopes for the earthen pad should be no steeper than 2H:1V (Horizontal to Vertical) slope. The edge of the earthen equipment pad shall have a minimum of 15 feet clearance from surrounding structures or construction loads.

7.5.2 Drilled Shaft Foundation Construction

The following items are important for the successful completion of drilled shaft foundations:

- The engineer's representative should observe all drilled shaft excavations. This inspection is to verify proper depth, bearing stratum, cleanliness, verticality (plumbness) and to record other observations regarding the drilled shaft construction.
- If water is present within the shaft, it is imperative that the contractor use proper construction methods to account for the water. Either the water must be removed, or the contractor must use tremies or pumps to allow concrete placement under water.
- Prompt placement of concrete in the excavation as it is completed, cleaned, and inspected is strongly recommended. Under no circumstances should a shaft be drilled that cannot be filled with concrete before the end of the workday.
- The reinforcement steel cage placed in the shaft should be designed to be stable and centered during the placement of concrete.
- The use of a casing or liner may be required in areas where shaft excavations extend into areas of caving sand soils. The drilling contractor should be prepared to provide means and methods to properly construct drilled shafts. We recommend that the construction contract include a budget for temporary casing and/or slurry drilling in case the sloughing of sands or entry of water prevents the proper construction of piers.
- Varying subsurface soil conditions may be encountered at a distance from a boring location or some interval between boring locations along the transmission line alignment. A Geotechnical Engineer or his representative should observe subsurface conditions during installation of any intermediate poles or ancillary structures such as anchors to verify subsurface conditions match the design criteria.
- Drilled shaft construction should follow applicable industry standard. Means and methods of construction shall be determined by the contractor.

7.5.3 Driven Pile Foundation Construction

The following items are important for the successful completion of driven pile foundations:

- The Project Engineer or his/her representative should observe pile driving. Steel W-piles shall be of the cross section, size, and weight per foot (mass per meter) specified in the contract documents. All piles which have been improperly driven, broken, or are otherwise defective shall be removed and replaced or otherwise corrected, as directed by the Project Engineer or his/her representative.
- Pile driving equipment furnished by the Contractor shall be approved by the pile design Engineer or his/her representative. All pile driving equipment shall be sized so that the project piles can be driven with reasonable effort to the required lengths without damage.
- Upon completion of driving, inspection, and approval, the pile (if required) shall be neatly cut on a horizontal plane at the elevation specified in the contract documents.
- Consider protecting piles against corrosion, abrasion or other detrimental factors.
- We recommend that pile load tests for production piles to verify pile capacities. Qualified geotechnical personnel should conduct the pile load tests and present the test results to the design engineer of record for further evaluation. Load tests should be performed in general accordance with ASTM standards. Piles driving time shall be recorded for all test and production piles and submitted to the design engineer of record for review.
- Pile driving can affect existing structures in the vicinity. Structures located close to the proposed pile foundations should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded.

7.6 Open Excavations

With all excavations in soil, sloped excavations and trench shields are required for excavations greater than four feet in depth. The contractor's "Competent Person" (as defined by OSHA) must inspect each trench wall to determine the type of bench or slope that is required. With all excavations, only a "Competent Person" shall determine whether sloped, benched, or trench shields can be used. OSHA and applicable state and local standards should be observed and followed. Site safety is the responsibility of the contractor. For general planning purposes, RRC offers the following:

- The surficial cohesive clay soils across this site are generally soft to medium stiff. This soil type classifies as an OSHA Type A material that requires the excavation's sidewall be sloped at 3/4H:1V (or flatter). Soft clay with cohesion less than 500 psf does NOT quality as type A.
- The sandy soils at the site possess low to zero cohesion. This soil type classifies as an OSHA Type B material that requires the excavation's sidewall be sloped at a 1H:1V slope (or flatter). The silt content may give the appearance of cohesion when first excavated, but this is not correct.
- The presence of water within any excavation automatically creates a Type C classification. Also, soft clays with cohesion less than 500 psf also require a Type C class. All Type C class excavations require a 1.5H:1V slope or bench.

Protect construction slopes and permanent embankment slopes from surface runoff water. Design site grading to deter surface water from flowing down unprotected slopes. The contractor should avoid surcharge loads, either static or dynamic, adjacent to an excavation slope. Prevent construction equipment from traveling along or near the top of the excavation slope. The contractor's "Competent Person" must monitor temporary slopes, trenches, and dewatering during construction in order to detect early warnings of movement. Site safety is the responsibility of the contractor.

7.7 Drainage and Construction Dewatering

Proper drainage should be provided away from the foundation elements during all phases of construction and post-construction grading. Proper drainage is essential to the long-term stability of the structures. Ponding of water near the foundation elements from improper drainage should not be permitted.

Based on the available groundwater information, shallow groundwater should not be a concern for the proposed foundation excavations. If rain causes perched groundwater conditions, we anticipate the groundwater re-charge rate should be slow enough to conduct excavation dewatering with conventional sumps and pumps.

7.8 Earthwork During Winter Weather or Wet Condition

The near-surface soft subgrade soils can be moisture sensitive and become extremely wet with exposure to ponding water due to precipitation events. To the extent practical, we recommend that the earthwork be completed during extended periods of warm and dry weather to reduce the amount of necessary subgrade remedial measures for soft and unsuitable conditions beneath access roadways, equipment pads, etc.

If earthwork is planned to be performed during wet season, it is important to maintain the site in a well-drained condition during construction including not allowing water to stand or pond on areas of the exposed earthwork. The diversion of surface runoff around exposed soils and draining of ponded water on the site are recommended. To help direct surface water over the earthwork, we suggest surface slopes of 2% to 3% be constructed and maintained. The use of berms, ditches, and similar means may be used to prevent stormwater from entering the earthwork area and to convey any water off-site efficiently. Surface drainage should be directed away from the pavement areas, and no ponding of water should be allowed on the paved surface or adjacent to the edges of the pavement areas. Temporarily recompact loose subgrade soils if rain is forecast to promote site drainage and reduce moisture infiltration.

Moisture control during wet weather months can be difficult since the moisture content of in-situ soil could be in excess of optimum moisture content, thereby making it very difficult to achieve specified compaction. If the subgrade cannot be adequately compacted to the minimum densities specified, earthwork may require additional mitigative measures including:

- removal of water and drying or aeration,
- chemical treatment of the soil, or
- removal and replacement with select fill.

If schedule allows, pumping out the ponding water, aeration and scarifying the surface of earthwork to accelerate drying by natural means is one of the cost-effective to reduce moisture in the soil after heavy rainfall. Construction equipment should not be operated on the site during this drying time. As an alternative, the contractor may elect to dry the soil using lime or fly ash worked into the wet soils or use geosynthetics as a stabilization technique. RRC should be contacted for additional recommendations if chemical treatment is needed due to soft and wet subgrade. If the saturated soil cannot achieve the specified compaction, it should be removed and replaced with Structural Fill. The removed soil may be used in non-structural areas where significant post construction settlement is acceptable. The contractor is ultimately responsible for moisture conditioning of fill/backfill materials to achieve proper compaction.

If earthwork is performed during the winter months when freezing may occur, no grading fill, structural fill, or other fill should be placed on frosted or frozen ground. Also, the frozen material should not be placed as fill material. Frozen ground should be allowed to thaw or be completely removed prior to placement of fill. A good practice is to cover the compacted fill with a “blanket” of loose fill to help prevent the compacted fill from freezing.

Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic. On-site clay soils may pump and unstable subgrade conditions could develop during general construction operations, particularly if the soils are exposed to high moisture levels and/or subjected to repetitive construction traffic. The use of light construction equipment would aid in reducing subgrade disturbance.

8.0 ACCESS ROADWAYS

It is our understanding that private access roadways will be built for construction and maintenance purposes and these roadways will consist of compacted earth or gravel. Traffic volumes during construction are anticipated to be frequent with medium to heavy equipment utilizing the access roadways. Following the construction period, the traffic volumes will be light and vehicles accessing the roadways will generally consist of pickup trucks and occasional single and multi-axle truck traffic. The section thickness design should be based upon the methodology outlined by the American Association of State Highways and Transportation Officials (AASHTO) for design of aggregate-surfaced roadways (Reference 14).

8.1 Pavement Section Thickness Recommendations Based on AASHTO 1993

Organic topsoil about 6 inches thick was encountered at borings and test pits performed across the site. However, depth of topsoil may vary in other parts of the project area. The contractor shall be prepared to perform the earthwork taking into consideration the site variability. Engineers and the project owner shall determine whether site grading for road subgrade is intended to meet elevations below topsoil or at topsoil elevations. This issue will also affect area grading and drainage. All road design options provided herein are based on topsoil removal.

The surficial soils encountered within a majority of the boreholes indicated native soils consisting mostly of clay with varying amounts of sand and silts. These materials are generally considered to be poor to good in terms of supporting vehicular and construction traffic as defined by AASHTO when used for support of pavement structures. If the civil engineering team suggests different

ESAL from the values below, RRC may be contacted to reevaluate the aggregate base thickness. But overall, the Civil Engineer of Record is responsible for the final roadway section design.

The estimated aggregate base thickness is presented in Table 8.1.1 according to AASHTO 1993 (Reference 14) are based on the anticipated ESAL values of different road sections within a typical wind project. Based on the results of laboratory CBR testing, the CBR value of surficial clay soils is about 2% (resilient modulus of about 3,980 psi) at majority of project site. If actual pavement design is based on wet subgrade without subgrade improvement, additional CBR tests are recommended to verity surficial materials encountered near road sections with higher required ESAL. If the actual ESAL and CBR are different from the values below, RRC may be contacted to reevaluate the aggregate base thickness. But overall, the Civil Engineer of Record is responsible for the final roadway section design.

Table 8.1.1. Estimated Aggregate Base Thickness for Access Roadway

Anticipated Minimum ESAL for different road sections	Aggregate Base Thickness (inches) With Allowable Rut Depth of 2-inch	
	Wet Subgrade Design CBR=2 (without Subgrade Improvement)	With Subgrade Improvement Using Soil-Cement/Lime Mix Assumed CBR=15*
1,000	4.0	4.0
5,000	6.0	4.0
10,000	7.5	4.0

Notes: * A formal cement or lime mix design should be performed prior to construction to determine design unconfined compressive strengths, CBR values and aggregate base thicknesses.

We caution that 2-inch rut is the AASHTO criteria for determining the limits to a failing pavement. In practice, 2 inches of rut is very poor performance. Therefore, RRC recommends a 1-inch proof-roll test limit to assess “acceptable” performance.

Prior to the placement of the aggregate base materials along access roadway alignments, stripping and removal of existing vegetation and other deleterious materials from the proposed roadway alignment should be performed. Topsoil and organics could be up to about 24 inches or more in thickness in some areas and should not be allowed for use in structural areas or along roadway alignments. Compact the resulting subgrade to provide a stable surface.

The exposed subgrade should then be proof-rolled with a tandem axle dump truck/water truck with a total rear axle load of no less than 40,000 lbs to assess the presence of soft areas and the need for remedial measures. A proof-roll test should include at least two passes over each area. Ruts and deflections during the proof-roll shall be ½-inch or less. Remove unsuitable soils and process the soils or replace with suitable soils and compact to a minimum compaction of 98% relative to ASTM D698. Moisture is not a criterion for road subgrades.

As an alternative to thick road base sections, the team can consider using geogrid (Tensar Biaxial Type 2 or equivalent) on top of geotextile (Mirafi HP 570 or equivalent) in areas where excessive “pumping” is observed.

Consideration could also be given to performing a cement or lime stabilization for the subgrade soils supporting pavements. Near surface soils may have higher moisture content during the wet season than those presented in the current report, especially in the area with poor drainage. Chemical stabilization may also help “dry” saturated subgrade soils with improved constructability. A stabilized subgrade will achieve a higher CBR value that in turn allows a reduction in the flexible base course. The flexible base course thickness for a properly stabilized subgrade section could be reduced to about four inches. A formal cement or lime mix design should be performed to determine the lime/cement content, compaction targets, design CBR values and aggregate base thicknesses.

Crushed aggregate road base should be in general accordance with California Department of Transportation Class 2 or better. Aggregate base materials should be compacted to at least 95% of the maximum dry density determined from ASTM D1557 and near optimum moisture content (moisture is not a pass/fail criteria).

It is imperative that proper drainage be provided in the construction of the roadways to enhance their performance. Post-construction proof rolling of the access roads should be performed prior to re-opening the roadways for traffic after periods of heavy rainfall/snow melt to assess stability of the roadway and the need for remedial measures. Areas where remedial measures are required should be re-worked and corrected prior to acceptance. It is also imperative that periodic inspection of the access roadways be performed following periods of rainfall or snowmelt to assess the condition of the roads. Additionally, compacted earth and gravel road design methodology presented herein this section assumes that on-going maintenance during and after construction will be performed to keep roads in serviceable condition.

9.0 LIMITATIONS

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is likely soil conditions will vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified immediately in order to provide supplemental recommendations (if needed). If the scope of the proposed construction, including the proposed loads or structural locations, changes from those described in this report, our data should also be reviewed.

We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. The recommendations provided in this report assume that an adequate program of tests and observations will be performed during the construction phase in order to evaluate compliance with our recommendations. RRC should be contacted if field conditions differ from the findings listed in the report to reevaluate our conclusions and recommendations.

This report may be used only by the client and only for the purposes stated, within three years of its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client, or the related design team members for this project, who wishes to use this report shall notify RRC of such intended use. Based on the intended use of the report, RRC may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release RRC from any liability resulting from the use of this report by any unauthorized party.

Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference," as that latter term is used relative to contracts or other matters of law.

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APPENDIX A

RRC

Table A1 - Summary of Subsurface Exploration and Geographic Coordinates

Boring ID	Latitude	Longitude	Electrical Resistivity Testing Date	Thermal Resistivity Sampling Date	CBR Sampling Date	Chemical Sampling Date	Infiltration Date	PLT Testing Date	Piezometer Location?	Piezometer Installation Date	Drilling/ Excavation Date	Auger (ft)	Air/Mud Rotary (ft)	Rock Core (ft.)	Total Depth (ft)	Groundwater During Drilling (ft)	Groundwater Immediately After Drilling (ft)	Remarks
B-1	38.39050985	-121.9066864				9/13/2023	9/15/2023				9/13/2023	40.5			40.5	14	11	Drilling location was moved about 80 ft.toward north due to wet soil condition and high rutting
B-2	38.39110343	-121.908087		9/14/2023	9/14/2023		9/15/2023				9/14/2023	20.5			20.5	9	10	
B-3	38.39149962	-121.9067698	9/25/2023					10/4/2023			9/13/2023	40.5			40.5	12	12	
B-4	38.39204075	-121.9072708		9/14/2023		9/14/2023					9/14/2023	20.5			20.5	9	15	
B-5	38.39262045	-121.9066633	9/26/2023				9/15/2023	10/4/2023			9/14/2023	20.5			20.5	13	11	
B-6	38.39281334	-121.9085437		9/14/2023							9/14/2023	20.5			20.5	9	9	Drilling location was moved about 20 ft. toward north due to wet soil condition and possible high rutting
B-7	38.39362688	-121.9067287				9/14/2023					9/14/2023	20.5			20.5	9	9	
B-8	38.39383999	-121.907111						10/4/2023			9/12/2023	20.5			20.5	9	7.5	
B-9	38.39453657	-121.9069315					9/15/2023		Yes	9/12/2023	9/12/2023	20.5			20.5	9	9	
B-9A	38.39453657	-121.9069315									9/14/2023	15.5			15.5	10	9	Drilled an offset borehole B-9A to verify the low blow counts.
B-10	38.39455202	-121.9084521	9/26/2023	9/14/2023		9/14/2023	9/15/2023	10/4/2023			9/14/2023	40.5			40.5	10	8	
TP-1	38.39087433	-121.907887									9/28/2023	10.0			10.0	NE	NE	
TP-SUB	38.39405128	-121.9085067	9/27/2023								9/28/2023	10.0			10.0	NE	NE	

Notes: NE = Not Encountered; NA = Not Available

Table A2: Well Log Information Obtained from the California Department of Water Resources

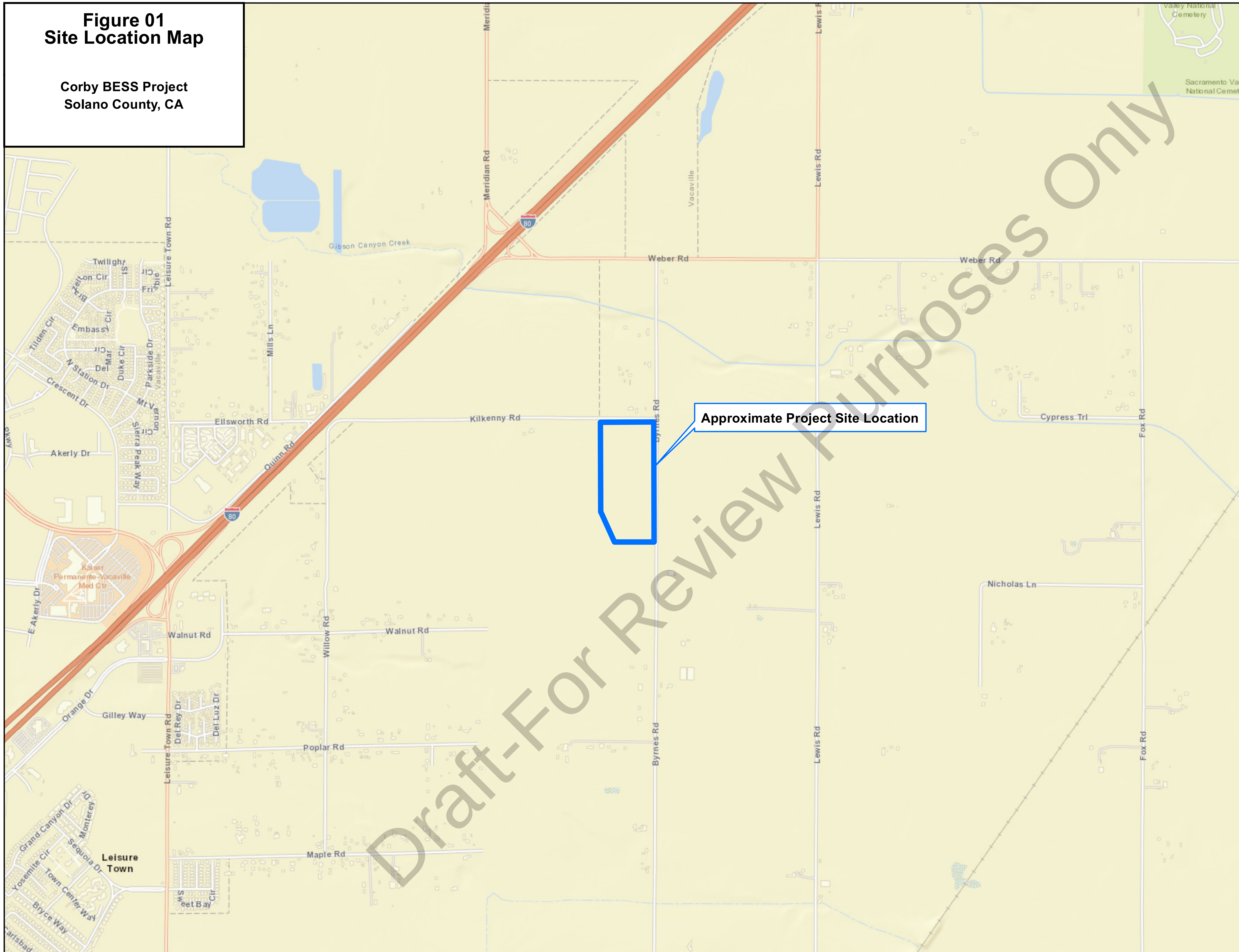
Well Report No. (WCR)	**Latitude**	**Longitude**	**Elevation** (feet above sea level)	**Well Depth** (feet below land surface)	**Water Level Record** (feet below land surface)	**Date of Record** (MM/DD/YYYY)
WCR2015-010080	38.370162	-121.954201		33	16	07/01/2015
WCR2015-010061	38.369795	-121.954574		30	14	07/10/2015
WCR2015-010072	38.370125	-121.954361		30.5	15	07/08/2015
WCR2015-010082	38.370247	-121.953885		36	15	07/01/2015
WCR2015-009495	38.370211	-121.954288		32	16	07/01/2015
WCR2015-010062	38.370454	-121.954646		31	13	07/10/2015
WCR2015-010083	38.370483	-121.954115		36	15	06/30/2015
WCR2015-010065	38.371142	-121.953651		30	14	07/09/2015
WCR2015-009493	38.369742	-121.954083		36	13	06/30/2015
WCR2017-004680	38.409282	-121.873790		860	98.6	08/12/2017
WCR2015-010088	38.369857	-121.954495		32	16	07/02/2015
WCR2015-010075	38.3699	-121.954244		32	17	07/08/2015
WCR2020-011895	38.3807	-121.846		870	65	08/31/2020
WCR2015-010076	38.369922	-121.954384		32.5	16	07/07/2015
WCR2015-010084	38.369899	-121.954348		33.5	16	07/01/2015
WCR2015-010081	38.369973	-121.954461		31.5	16	07/01/2015
WCR2015-009494	38.370046	-121.954124		31	17	07/02/2015
WCR2015-010078	38.370192	-121.954488		36	14	07/07/2015
WCR2015-009492	38.371129	-121.953659		60	14	07/09/2015
WCR2017-004723	38.3875548	-121.8614093		379	35.2	08/22/2017
WCR2019-016933	38.40635	-121.98158		400	315	12/24/2018
WCR2015-010063	38.370666	-121.954416		60	14	07/09/2015
WCR2015-010079	38.369909	-121.954659		30.5	15	07/02/2015
WCR2015-010064	38.370689	-121.954396		30	14	07/09/2015
N/D = No Data Available	https://dwr.maps.arcgis.com/apps/webappviewer/index.html?id=181078580a214c0986e2da28f8623b37					

Table A3: Summary of Groundwater Level Measurements

Boring No.	Latitude	Longitude	Installation Date	Pipe Stick Out Length (ft)	Pipe Length (ft)	Groundwater Immediately After Installation (ft)	Groundwater 24 hours After Installation (ft)	Groundwater 48 hours After Installation (ft)	Groundwater One Month After Installation (ft)
B-9	38.394527	-121.906933	09/13/23	2	15	9.0	10.0	10.1	11.1

Figure 01
Site Location Map

Corby BESS Project
Solano County, CA



0 0.25 0.5 Miles

N

Legend

Project Boundary

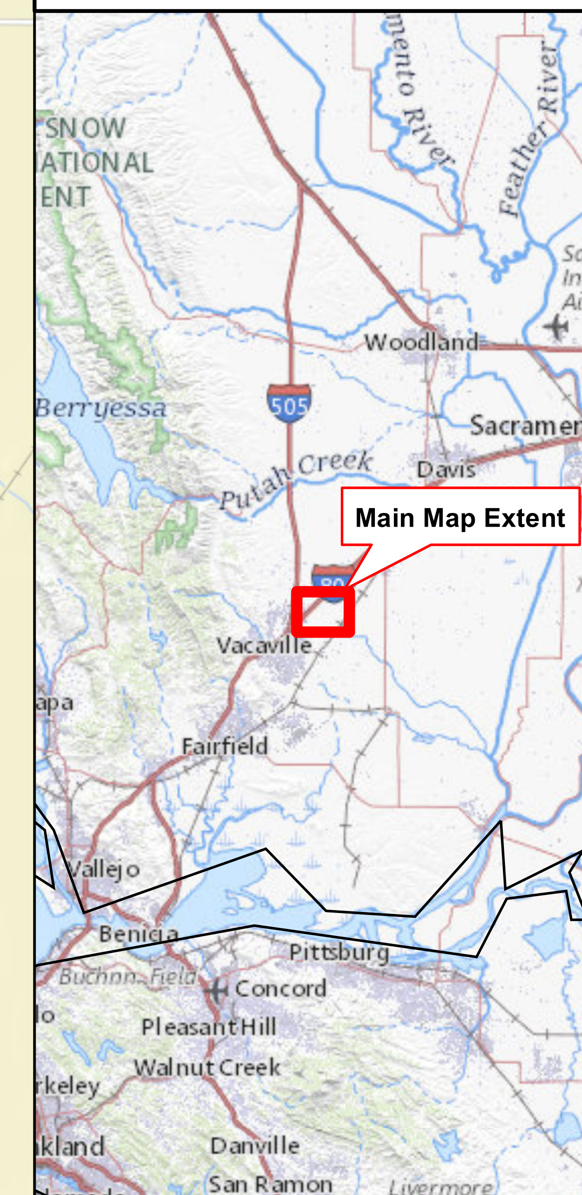


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Figure 02
BESS Boring Locations on a Topographic Map

Corby BESS Project
Solano County, CA

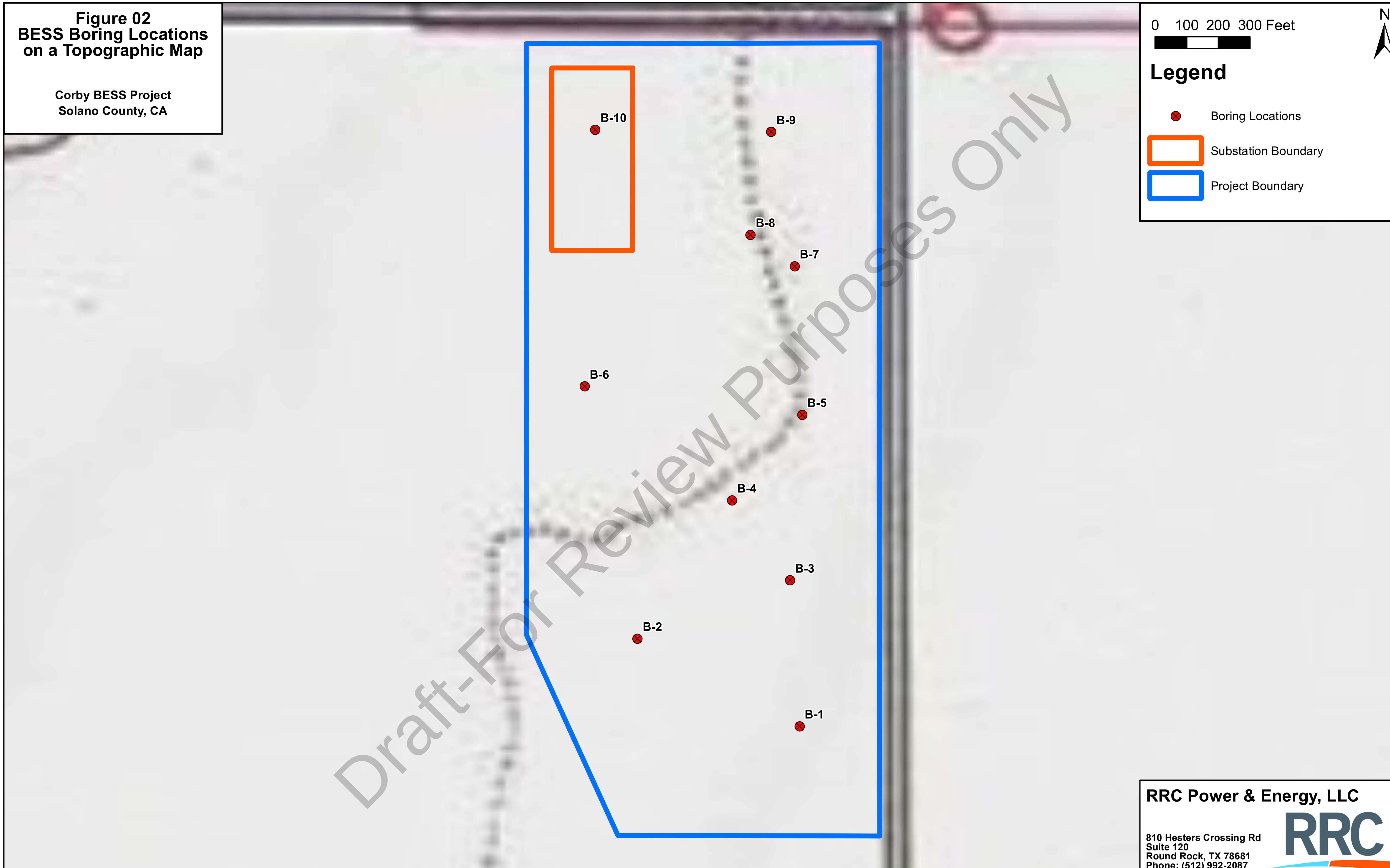

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Figure 03
Test Pit (TP) Locations Map

Corby BESS Project
Solano County, CA

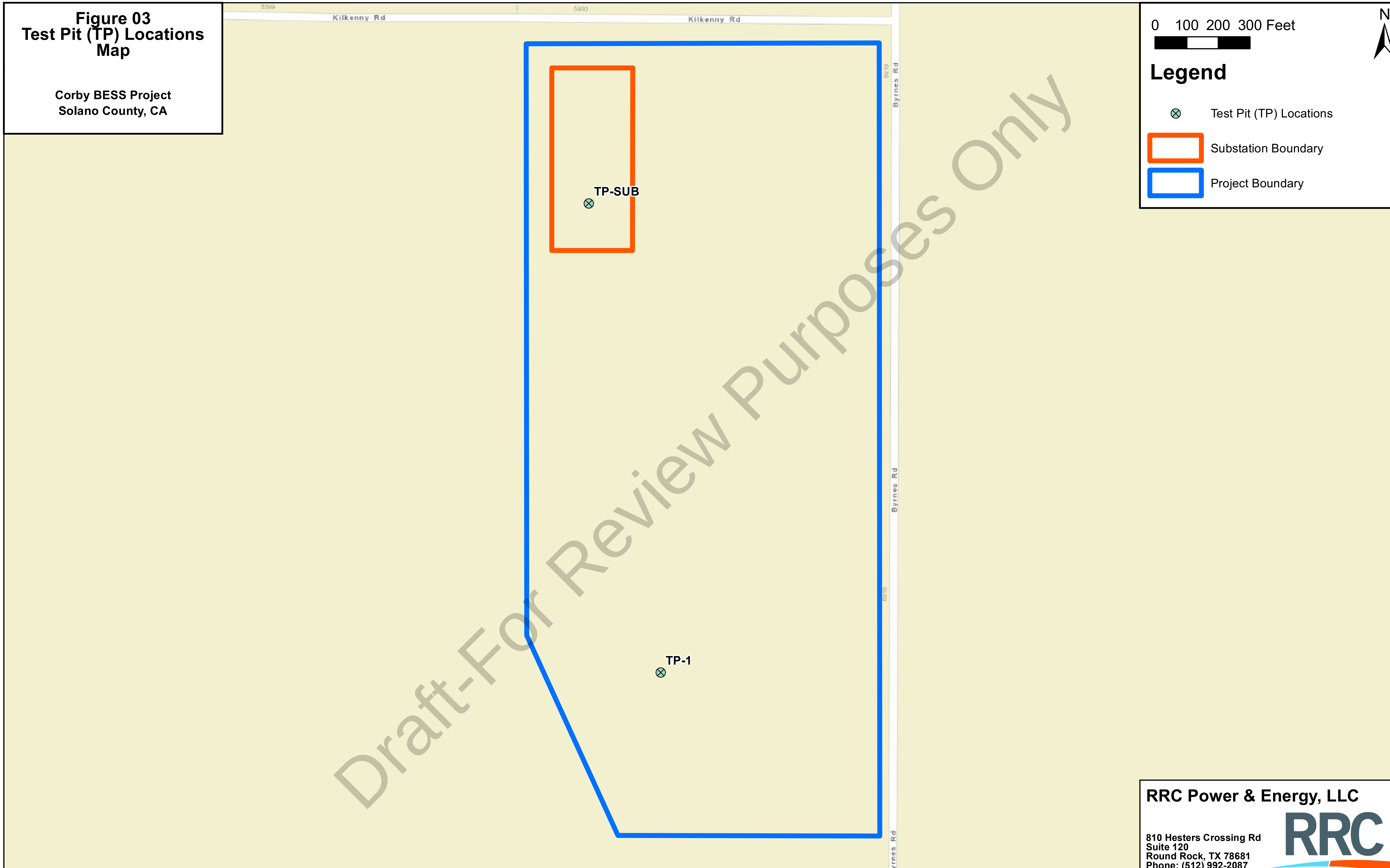

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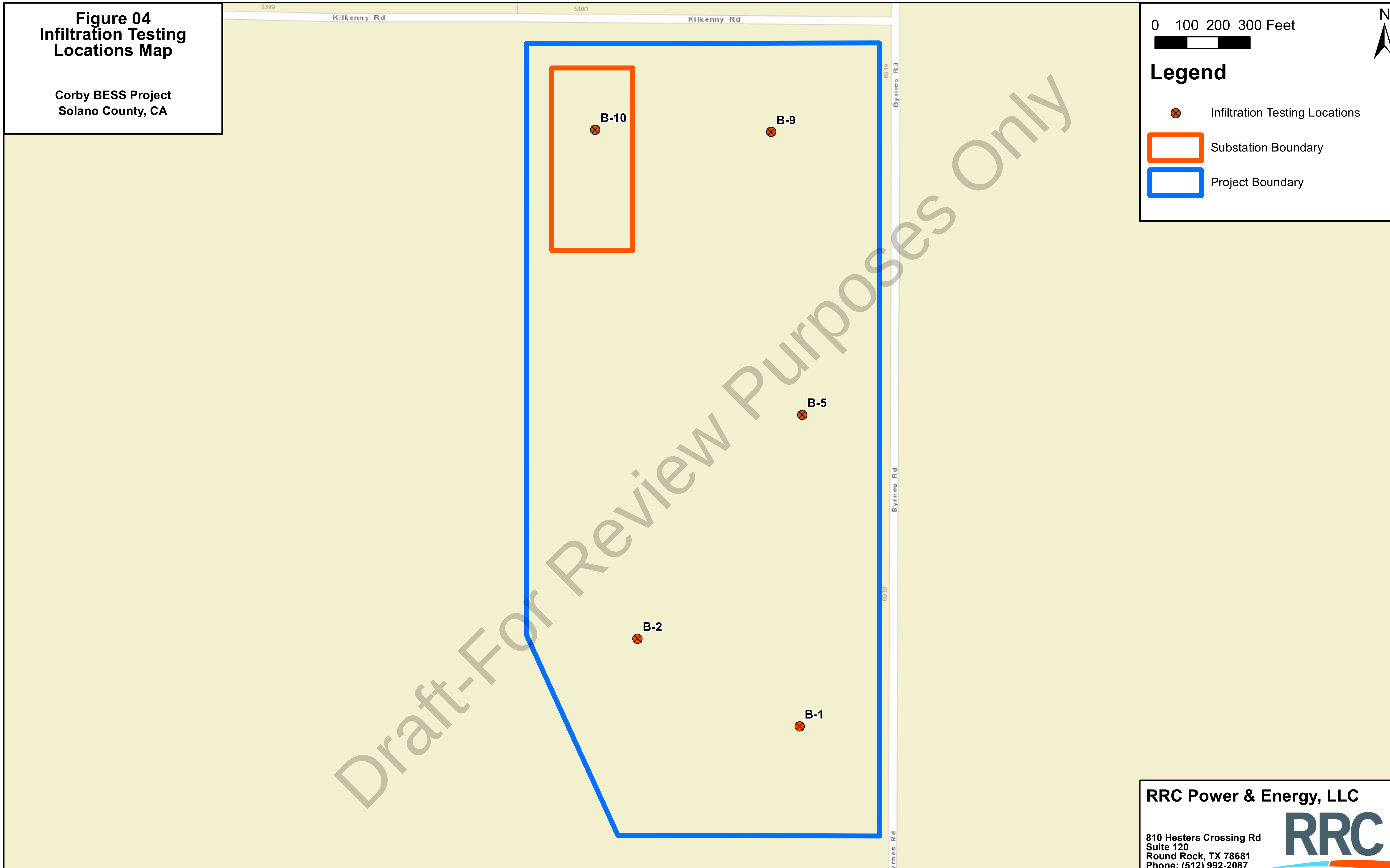
Figure 04
Infiltration Testing Locations Map

Corby BESS Project
Solano County, CA



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Round Rock, TX 78681
Phone: (512) 992-2087

RRC

Figure 05
Site Vicinity Geologic Map

Corby BESS Project
Solano County, CA

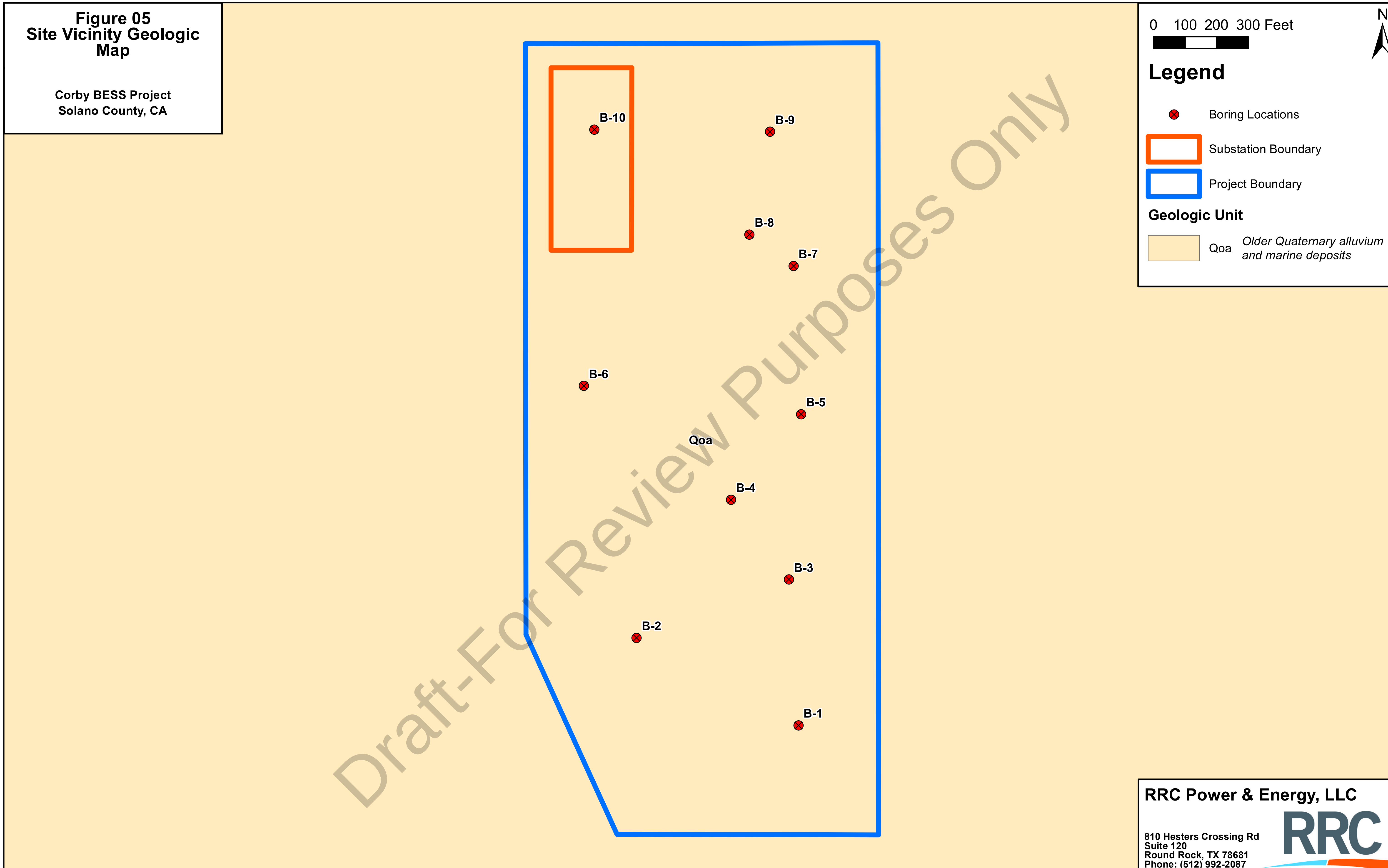
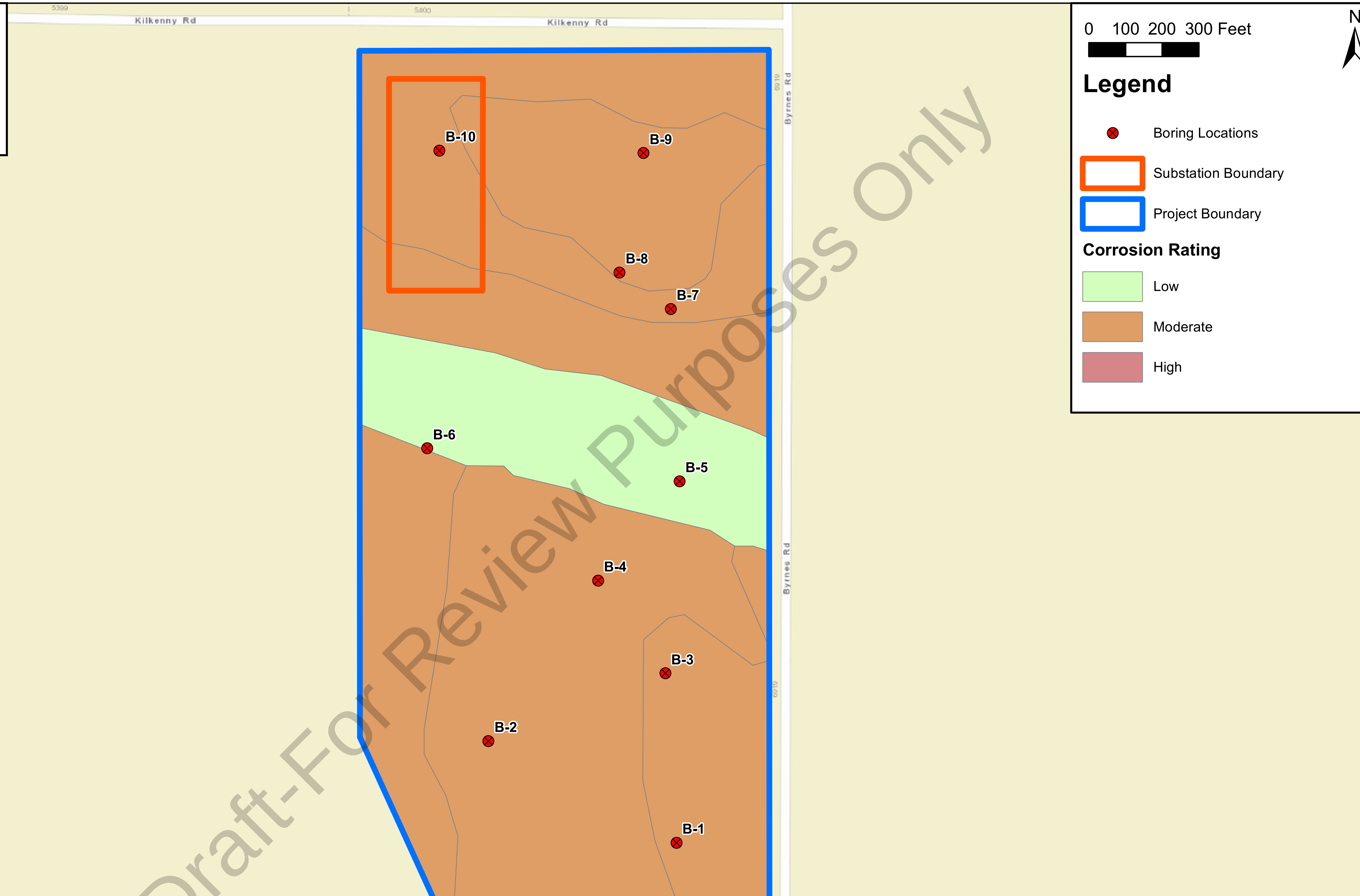


Figure 06
Corrosion of Concrete Map

Corby BESS Project
Solano County, CA



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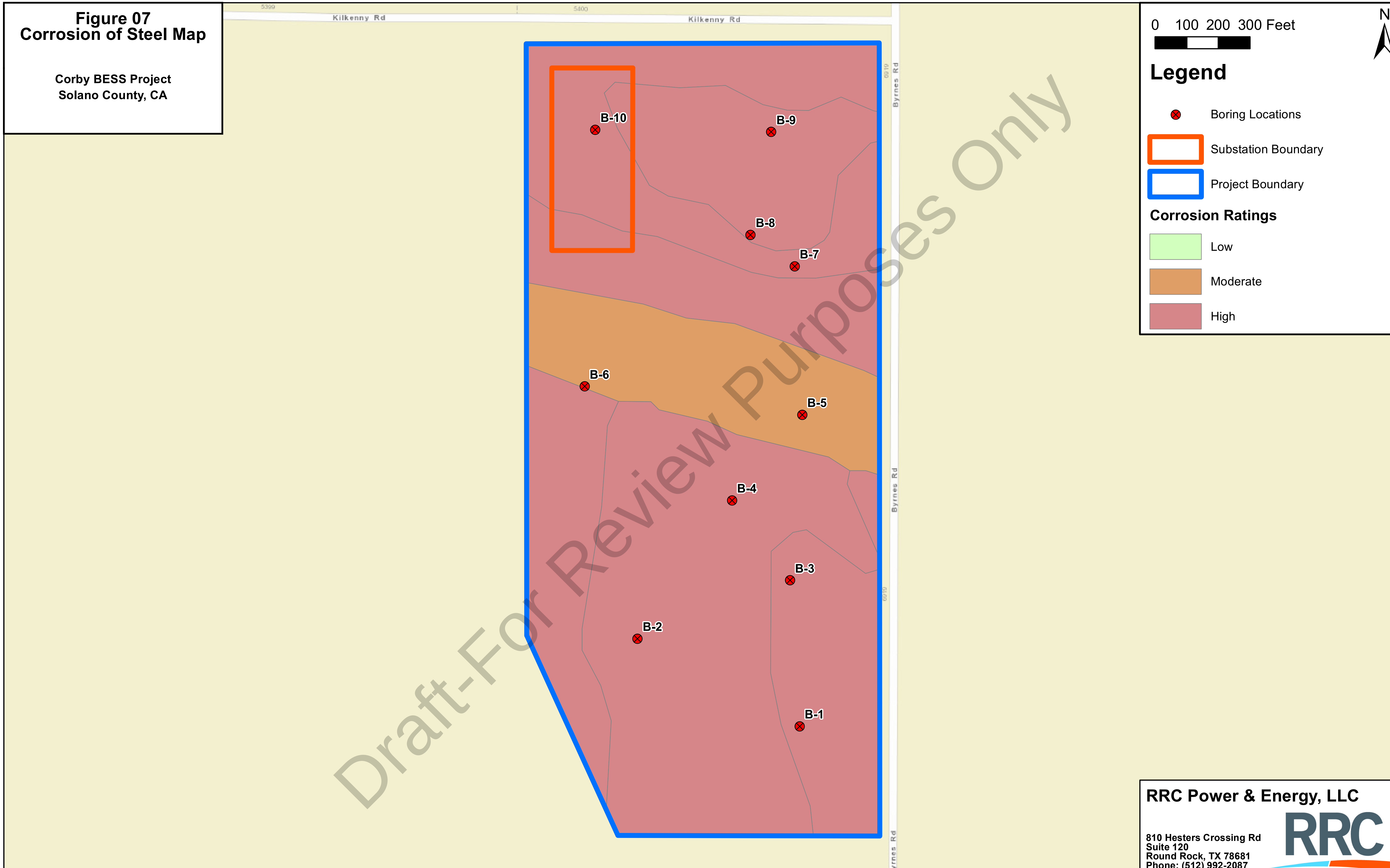
Suite 120
Round Rock, TX 78681
Phone: (512) 992-2087

Figure 07
Corrosion of Steel Map

Corby BESS Project
Solano County, CA



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Round Rock, TX 78681
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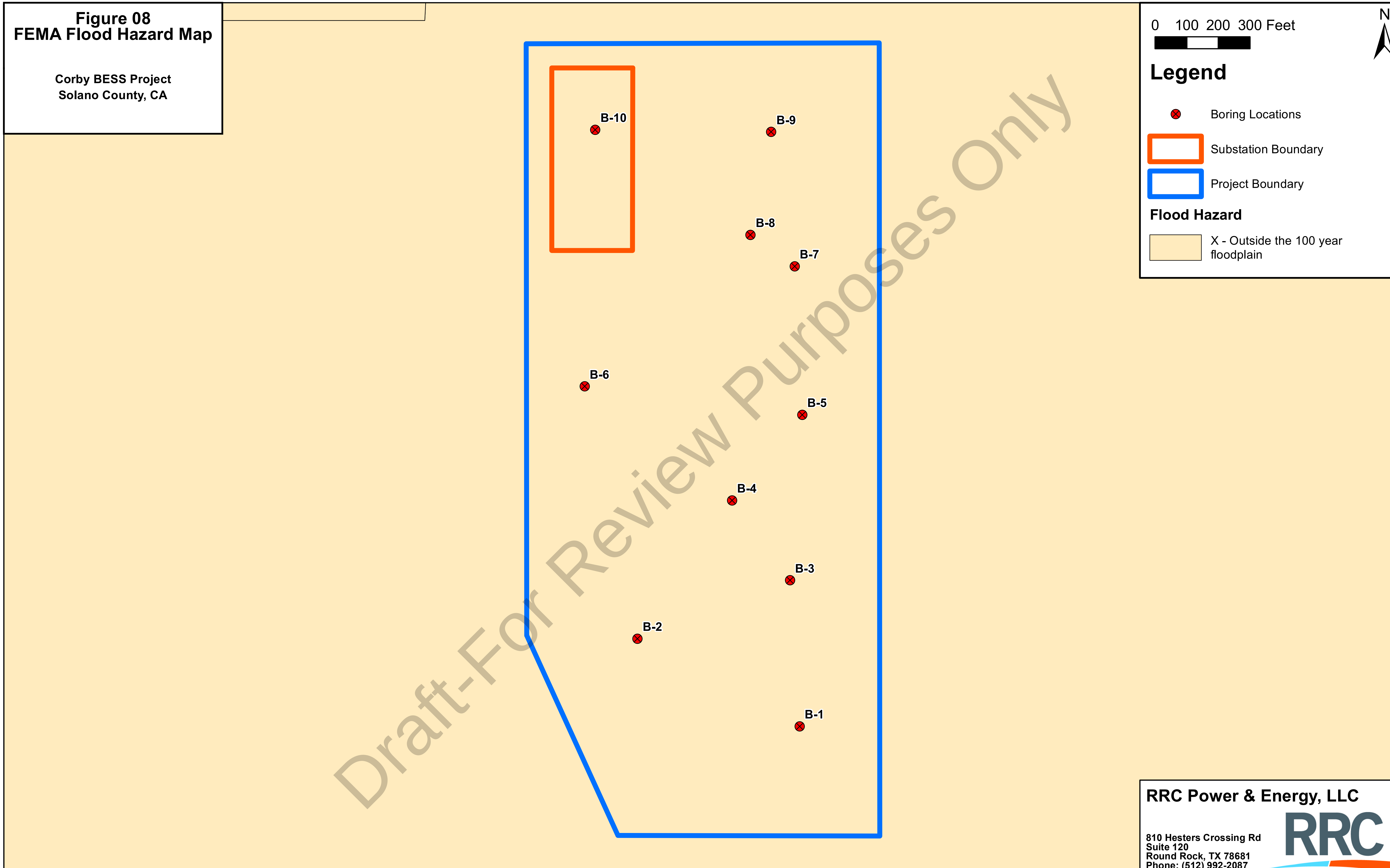
Figure 08
FEMA Flood Hazard Map

Corby BESS Project
Solano County, CA



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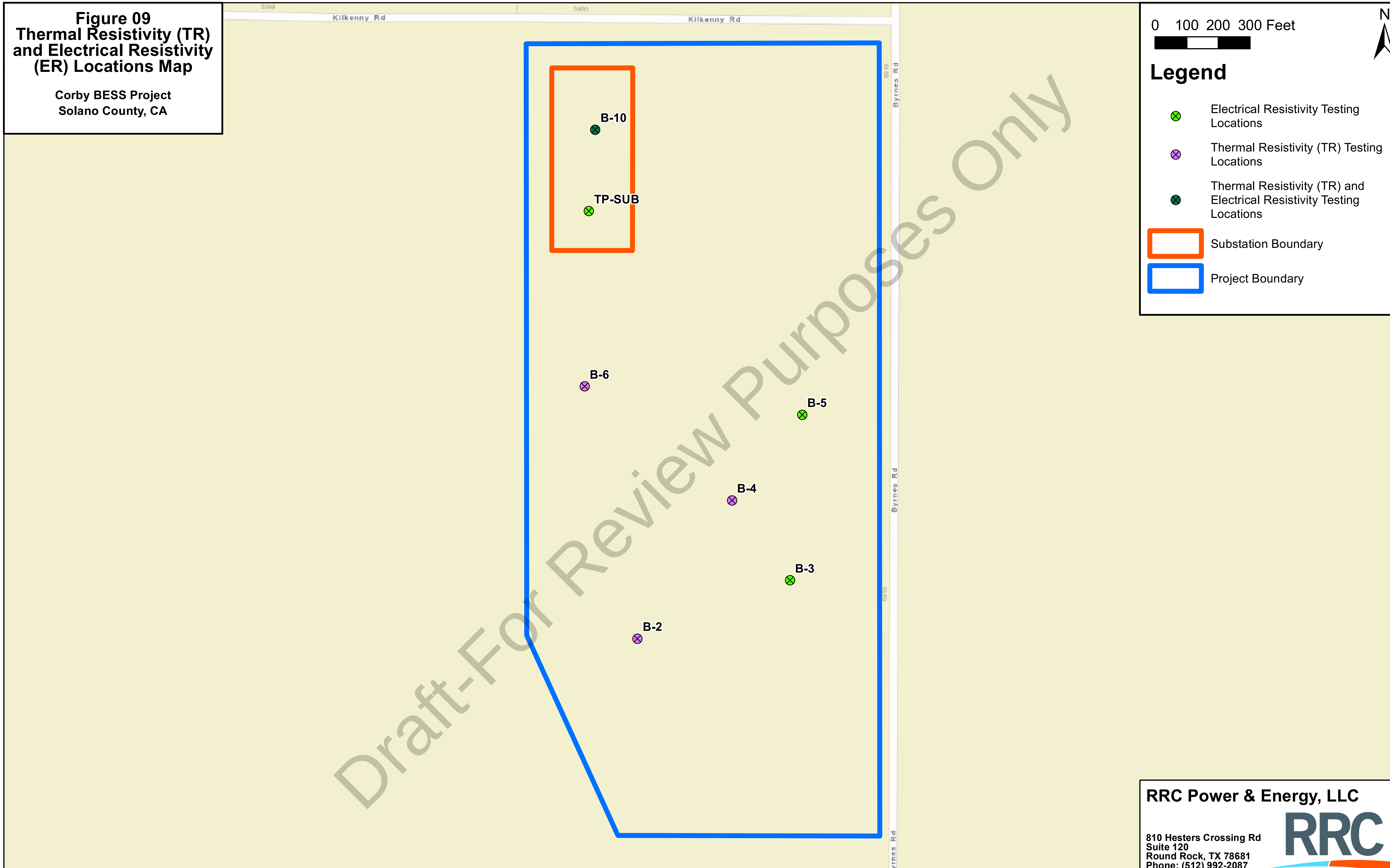
Phone: (512) 992-2087

Figure 09
Thermal Resistivity (TR) and Electrical Resistivity (ER) Locations Map

Corby BESS Project
Solano County, CA



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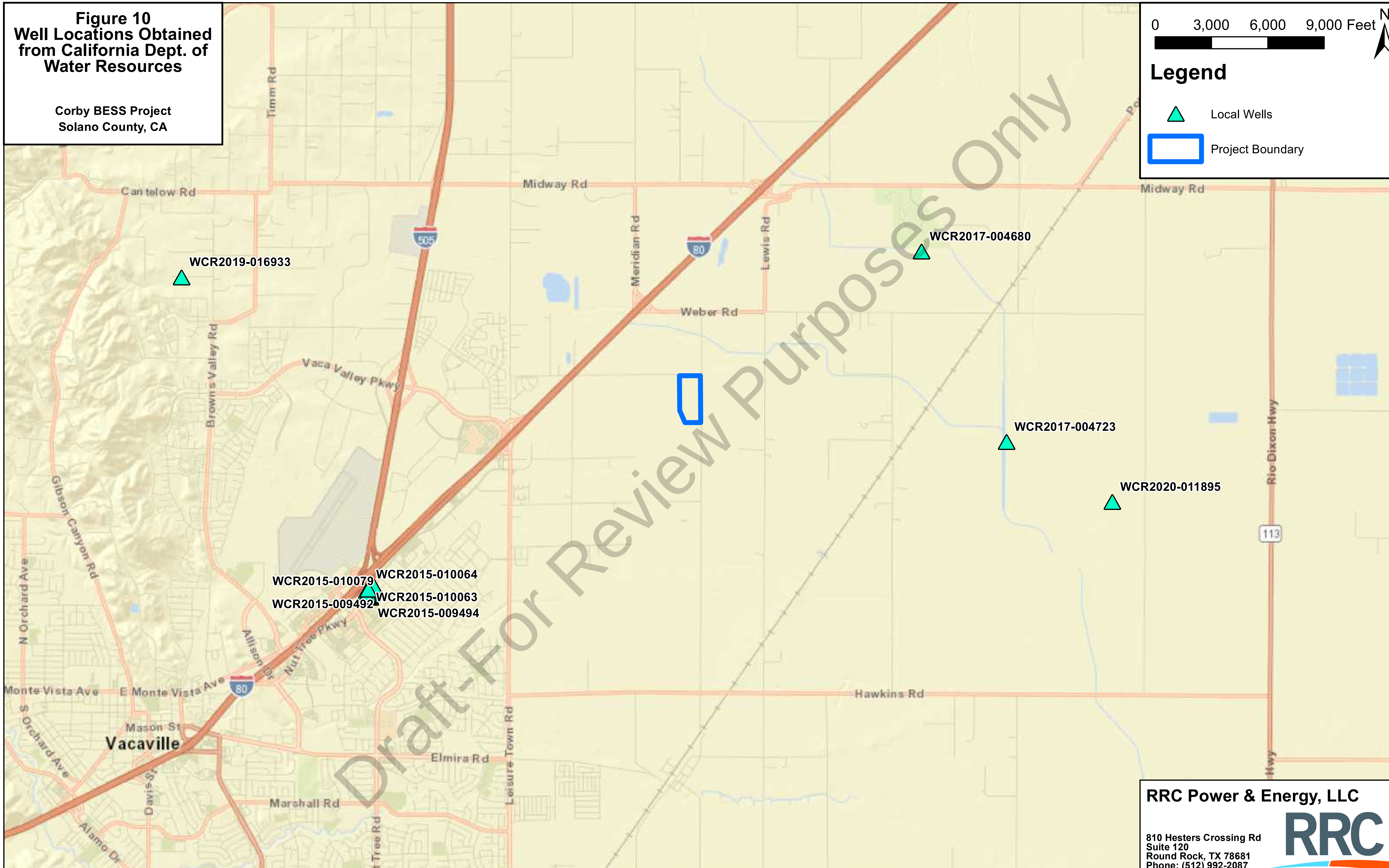
Phone: (512) 992-2087

Figure 10
Well Locations Obtained from California Dept. of Water Resources

Corby BESS Project
Solano County, CA



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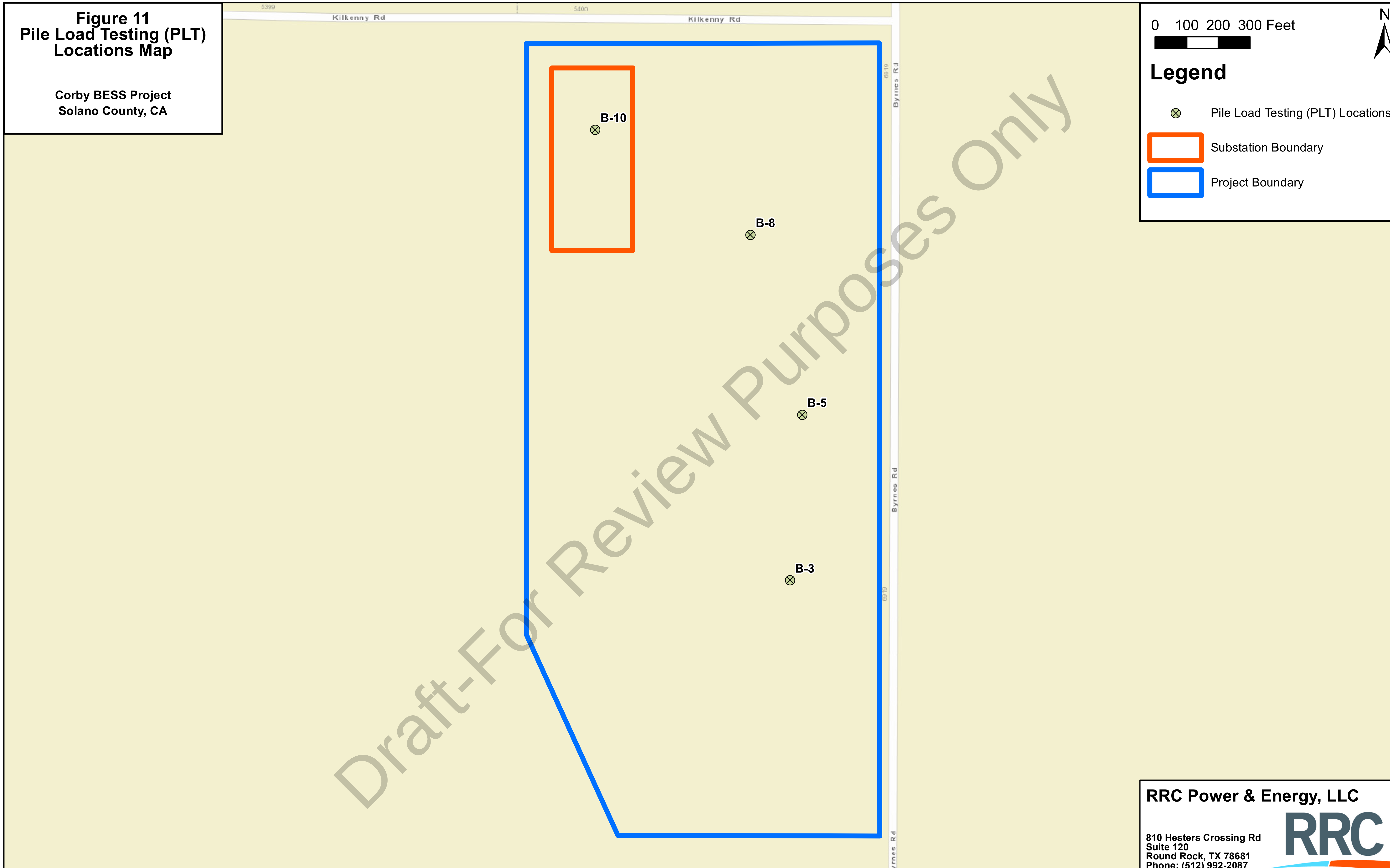
Phone: (512) 992-2087

Figure 11
Pile Load Testing (PLT) Locations Map

Corby BESS Project
Solano County, CA



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Round Rock, TX 78681
Phone: (512) 992-2087

Figure 12
Piezometer (PZ) Location Map

Corby BESS Project
Solano County, CA

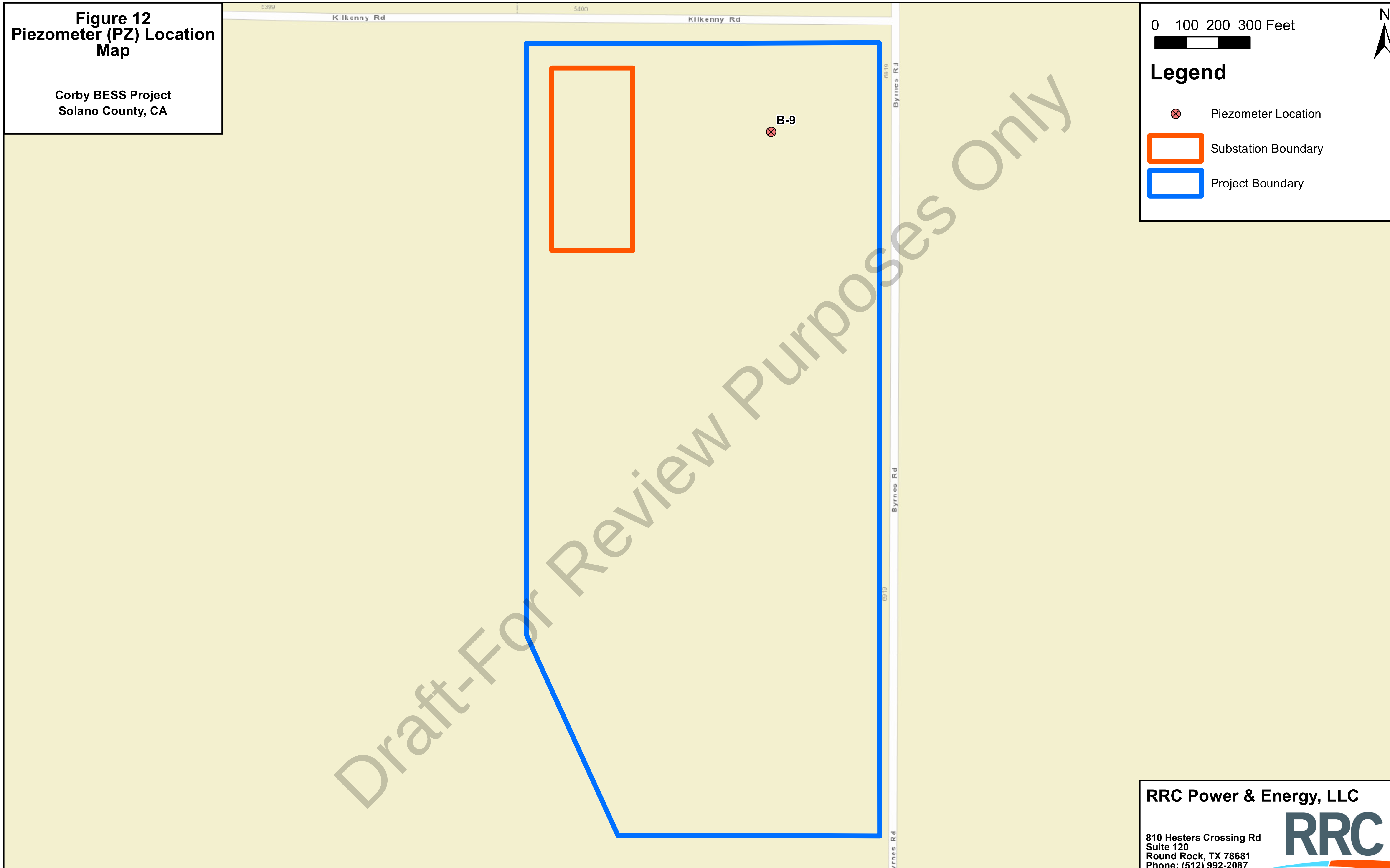
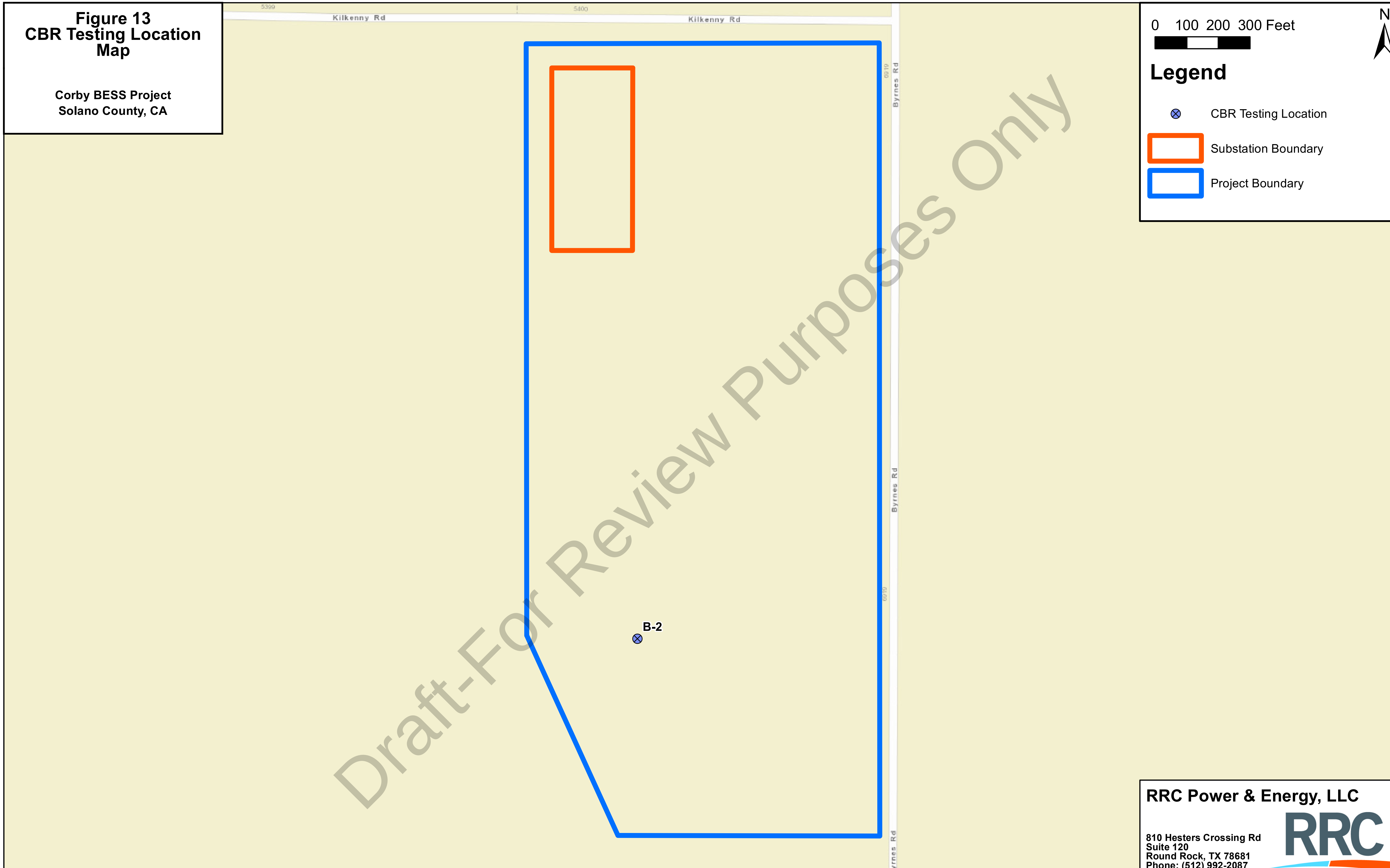
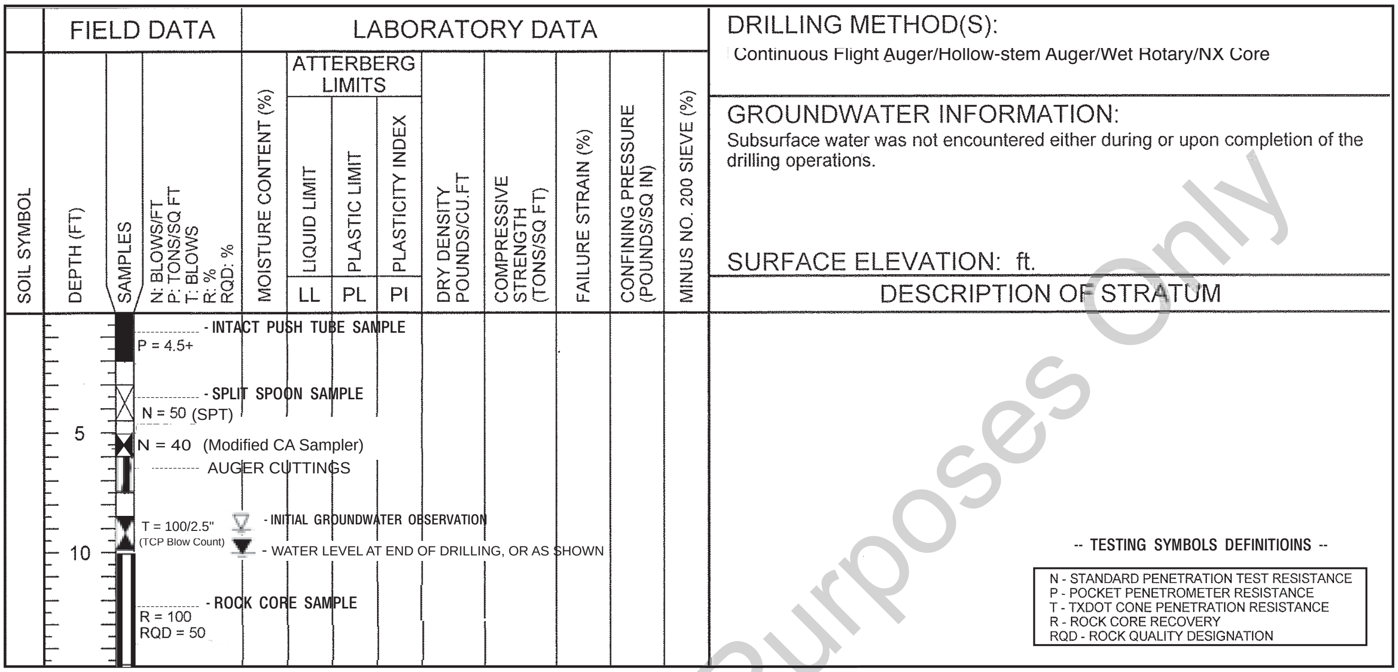


Figure 13
CBR Testing Location Map

Corby BESS Project
Solano County, CA



BORING LOG KEY



TYPICAL SOIL AND ROCK SYMBOLS (USCS CLASSIFICATION)

Lean Clay (CL)	Poorly-Graded Sand (SP)	Claystone
Fat Clay (CH)	Well-Graded Sand (SW)	BASALT
Silt (ML)	Poorly-Graded Gravel (GP)	Limestone
Elastic Silt (MH)	Well-Graded Gravel (GW)	Sandstone
Silty Sand (SM)	Clayey Gravel (GC)	Siltstone
Clayey Sand (SC)	Silty Gravel (GM)	Fill Material
Silty, Clayey Sand (SC-SM)	Silty Clay (CL-ML)	Shale

DEGREE OF WEATHERING

1) Unweathered: No evidence of any chemical or mechanical alteration.
2) Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10% of the rock volume altered.
3) Moderately weathered: Discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering "halos" evident, 10% to 50% of the rock volume altered.
4) Highly weathered: Entire mass discolored, alteration pervading nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away.
5) Decomposed: rock reduced to a soil with relicit rock texture, generally molded and crumbled by hand.

SOIL STRUCTURE

Calcareous.......... Containing calcium carbonate

Slickensided........ The presence of planes of weakness having a slick and glossy appearance

Interbedded......... Alternating layers of varying material

LOG OF BORING B-01

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/13/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 14 ft. during drilling and measured at 11 ft. immediately after drilling

SURFACE ELEVATION (FT):

DEPTH (FT)	FIELD DATA: N: BLOWS/FT, P: TONS/SQ FT, T: BLOWS, R: %, RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
												6 in. Topsoil
	N = 8	20									E-11	FAT CLAY (CH), trace Sand, dark brown to gray, medium stiff, moist, trace roots
5	N = 10	20								69	E-10	SANDY FAT CLAY (CH), light brown, stiff, moist, calcareous
	P = 4.5	19	43	13	30	111	3.91	8.3	0.0	66	D-10	SANDY LEAN CLAY (CL), with Sand, brown, stiff to very stiff, dry to moist, calcareous
10	N = 24										D-9	
	N = 13	25	61	21	40						E-10	FAT CLAY (CH), with Sand, brown, stiff, moist, calcareous
15	N = 14										E-10	SANDY FAT CLAY (CH), light brown, stiff, moist to wet, iron stained, calcareous
20	N = 16										E-9	FAT CLAY (CH), with Sand, brown, very stiff, dry to moist, iron stained, calcareous
25	N = 15										D-9	SANDY LEAN CLAY (CL), brown, very stiff, dry to moist, iron stained, calcareous
30	N = 17										E-9	FAT CLAY (CH), with Sand, brown, very stiff, dry to moist, iron stained, calcareous
35	N = 24										D-9	LEAN CLAY (CL), with Sand, brown, very stiff, dry to moist, iron stained, calcareous
40	N = 22										C-5	CLAYEY SAND (SC), brown to gray, medium dense, moist, fine grained, iron stained

Total Depth = 40.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.390737, Long. -121.906687

LOG OF BORING B-02

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 10 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA			LABORATORY DATA										
SOIL SYMBOL	DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
	0												6 in. Topsoil
		P = 4.5	21 17	47 40	15 16	32 24	108				56 67	D-8	SANDY LEAN CLAY (CL), brown, stiff to hard, moist, iron stained
	5	N = 13	21									D-10	
		N = 19	20	42	16	26					75	D-9	LEAN CLAY (CL), with Sand, brown, very stiff, moist, iron stained, trace calcareous
	10	N = 12	22									C-5	POORLY-GRADED SAND (SP-SC), with Clay, brown, medium dense, wet, fine grained
		N = 15										E-9	SANDY FAT CLAY (CH), brown, very stiff, moist, iron stained
	15	N = 20										E-9	
	20	N = 19										C-5	POORLY-GRADED SAND (SP-SC), with Clay, brown, medium dense, moist to wet, fine grained
													Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.391106, Long. -121.908086

LOG OF BORING B-03

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/13/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 12 ft. during drilling and measured at 12 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA / LABORATORY DATA

DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
													6 in. Topsoil
		N = 4	19	41	14	27					79	D-12	LEAN CLAY (CL), with Sand, brown to gray, soft, moist, trace roots
5		P = 4.5	21	53	13	40	95				75	E-8	FAT CLAY (CH), with Sand, brown, hard, dry to moist
		N = 8	18								41	C-6	CLAYEY SAND (SC), light brown, loose, moist, fine grained
10		N = 16	20									D-9	LEAN CLAY (CL), with Sand, brown, very stiff, moist
		N = 15										C-5	POORLY-GRADED SAND (SP-SC), with Clay, brown, medium dense, moist to wet, fine grained
15		N = 16	30								67	E-9	SANDY FAT CLAY (CH), brown, very stiff, moist
20		N = 10										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist
25		N = 13										D-10	LEAN CLAY (CL), with Sand, brown, stiff, moist
30		N = 26										E-9	FAT CLAY (CH), with Sand, brown, very stiff, moist, trace Gypsum
35		N = 19										D-9	LEAN CLAY (CL), with Sand, brown to gray, very stiff, moist
40		N = 22										D-9	SANDY LEAN CLAY (CL), brown, very stiff, moist
													Total Depth = 40.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.391498, Long. -121.906770

LOG OF BORING B-04

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 15 ft. immediately after drilling

SURFACE ELEVATION (FT):

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
0												6 in. Topsoil
												SANDY LEAN CLAY (CL), brown, stiff to very stiff, dry to moist, trace calcareous nodules
	P = 2.0	21 16	47 32	16 14	31 18	106				65 61	D-10	
	N = 15	19									D-9	
5												
	N = 18										D-9	
10	N = 10	21	39	15	24						D-10	LEAN CLAY (CL), with Sand, brown, stiff, moist, iron stained
	N = 11										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist to wet, iron stained
15	N = 14	23									D-10	Grading moist
20	N = 23										D-9	LEAN CLAY (CL), trace Sand, brown, very stiff, dry to moist, iron stained
												Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.392040, Long. -121.907267

LOG OF BORING B-05

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 13 ft. during drilling and measured at 11 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA / LABORATORY DATA

SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
														6 in. Topsoil
			N = 10	16									D-10	LEAN CLAY (CL), with Sand, brown, stiff to hard, moist, trace roots
	5		P = 4.5	18	40	15	25	106				76	D-8	
			N = 14	21	48	18	30					86	D-10	Grading trace Sand, iron stained
	10		N = 13	25									E-10	FAT CLAY (CH), with Sand, brown, stiff, moist, iron stained
			N = 12	21									C-5	POORLY-GRADED SAND (SP-SC), with Clay, brown, medium dense, wet, fine grained
	15		N = 13										C-5	CLAYEY SAND (SC), brown, medium dense, wet, fine grained
	20		N = 16										E-9	FAT CLAY (CH), with Sand, brown, very stiff, dry to moist, iron stained
														Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.392609, Long. -121.906669

RENEWABLE LOG NEXTERA SBC - LOG A GNNL01.GDT - 10/31/23 09:47 - R:\OPERATIONS\OP2\02 DESIGN\GEOTECHNICAL\G DRIVE\GINT\PROJECTS\2023\CORBY BESS - GE2306031\CORBY BESS - GE2306031.GPJ

LOG OF BORING B-06

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 9 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA / LABORATORY DATA

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
												6 in. Topsoil
	P = 2.0	17 16	44 38	14 13	30 25	111				58 67	D-10	SANDY LEAN CLAY (CL), brown, stiff to very stiff, moist, iron stained
5	N = 16	18									D-9	
	N = 17	22	52	19	33					91	E-9	FAT CLAY (CH), trace Sand, brown, very stiff, moist, calcareous
10	N = 11	27								36	C-5	CLAYEY SAND (SC), brown, medium dense, wet, fine grained
	N = 15	19									C-5	POORLY-GRADED SAND (SP-SC), with Clay, brown, medium dense, wet, fine grained
15	N = 11										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist, calcareous
20	N = 19										E-9	FAT CLAY (CH), with Sand, brown, very stiff, dry to moist, trace Gypsum
												Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.392874, Long. -121.908534

LOG OF BORING B-07

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 9 ft. immediately after drilling

SURFACE ELEVATION (FT):

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
												6 in. Topsoil
	N = 2	24	44	15	29					65	D-12	SANDY LEAN CLAY (CL), dark brown, soft, moist, trace roots
	P = 4.0	24	51	20	31	100	1.26	1.8	0.0	89	E-11	FAT CLAY (CH), trace Sand, dark brown, medium stiff to stiff, moist
5	N = 11	26								82	E-10	Grading with Sand, brown
10	N = 12	17									C-5	CLAYEY SAND (SC), brown, medium dense, moist, fine grained
	N = 21	19									C-5	POORLY-GRADED SAND (SP), trace Clay, brown, medium dense, moist to wet, fine grained
15	N = 25										C-5	CLAYEY SAND (SC), brown, medium dense, moist to wet, fine grained
20	N = 10										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist, iron stained
												Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.393626, Long. -121.906731

LOG OF BORING B-08

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/12/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 7.5 ft. immediately after drilling

SURFACE ELEVATION (FT):

SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
														6 in. Topsoil
			N = 7	21	39	15	24					66	D-11	SANDY LEAN CLAY (CL), dark brown, medium stiff to very stiff, dry to moist
	5		N = 40										D-9	Grading brown
			N = 9	18								65	D-10	
	10		N = 8	21								29	C-6	CLAYEY SAND (SC), brown, loose, moist to wet, fine grained
			N = 18										C-5	POORLY-GRADED SAND (SP), trace Clay, brown, medium dense, wet, fine to medium grained
	15		N = 12										C-5	
	20		N = 31										C-4	CLAYEY SAND (SC), brown, dense, moist, fine grained
														Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.393837, Long. -121.907115

RENEWABLE LOG NEXTERA SBC - LOG A GNNL01.GDT - 10/31/23 09:48 - R:\OPERATIONS\OP2\02 DESIGN\GEOTECHNICAL\G DRIVE\GINT\PROJECTS\2023\CORBY BESS - GE2306031\CORBY BESS - GE2306031.GPJ

LOG OF BORING B-09

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/12/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 9 ft. during drilling and measured at 9 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA / LABORATORY DATA

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
												6 in. Topsoil
	N = 13										E-11	FAT CLAY (CH), trace Sand, brown, medium stiff to stiff, dry to moist
5	N = 14	23	53	22	31					86	E-10	
	N = 9	19								56	D-10	SANDY LEAN CLAY (CL), brown, stiff, moist
10	N = 7	27								37	C-6	CLAYEY SAND (SC), brown, loose, moist, fine grained
	N = 3	24								60	E-12	SANDY FAT CLAY (CH), brown, soft to stiff, moist to wet
15	N = 9										E-10	Grading moist
20	N = 10										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist
												FAT CLAY (CH), trace Sand, brown, stiff, moist
												Total Depth = 20.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.394527, Long. -121.906933

LOG OF BORING B-09A

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 10 ft. during drilling and measured at 9 ft. immediately after drilling

SURFACE ELEVATION (FT):

SOIL SYMBOL	DEPTH (FT)	SAMPLES	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT (LL)	PLASTIC LIMIT (PL)	PLASTICITY INDEX (PI)	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
	0–12													Undifferentiated Overburden. Refer to B-09 for detailed soil description.
	12–15.5		P = 3.5										D-10	SANDY LEAN CLAY (CL), brown, stiff, moist, iron stained
			N = 10										D-10	
														Total Depth = 15.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.394498, Long. -121.906980

LOG OF BORING B-10

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031
DATE(S) DRILLED: 9/14/2023

DRILLING METHOD(S):
Hollow Stem Auger

GROUNDWATER INFORMATION:
Groundwater encountered at 10 ft. during drilling and measured at 8 ft. immediately after drilling

SURFACE ELEVATION (FT):

FIELD DATA / LABORATORY DATA

DEPTH (FT)	N: BLOWS/FT P: TONS/SQ FT T: BLOWS R: % RQD: %	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY POUNDS/CU.FT	COMPRESSIVE STRENGTH (TONS/SQ. FT)	STRAIN AT FAILURE (%)	CONFINING PRESSURE (POUNDS/SQ IN)	MINUS NO. 200 SIEVE (%)	NEXT ERA SOIL BEHAVIOR CLASSIFICATION (SBC)	DESCRIPTION OF STRATUM
												6 in. Topsoil
	P = 4.5	23 19	55 36	17 15	38 21	103				87 74	D-8	LEAN CLAY (CL), with Sand, brown, stiff to hard, dry to moist
5	N = 19	20	47	21	26					85	D-9	
	N = 14	21									D-10	
10	N = 10	20								73	D-10	
	N = 11	22									D-10	
15	N = 16										D-9	
20	N = 13										D-10	Grading iron stained, calcareous
25	N = 10										D-10	
30	N = 20										D-9	
35	N = 28										D-9	
40	N = 35										D-8	

Total Depth = 40.5 ft.

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TXDOT CONE PENETRATION RESISTANCE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
GPS COORDINATES: Lat. 38.394553, Long. -121.908452

LOG OF TEST PIT TP-01

yy- 14Y, L& n=, Lt RFDD-
892 e, 7H,L7 - L477N=t yaFTdNHl 952
y 4d=a y 4UkFPJ : 8%89
P, E, 6S4=, I s. 95I BB5X528:

- DknoPI o, GHHLr n=, Lt Ry, 74dLU, 7Fk=U0
1ywVn- PI - 4LARMnTT
Dw- CPkwo I T4E=4 - 4d=HRF- r EN4L=Nk
omb Mnyl gn5/ 2%2/ 9
pCPnsTI nJ- ChCPnpl Bu58u5/
nJ- ChCPkwo bnPewpsTII
nGUrWH4L
gywmopOCPny locwybCPkwol
gL4d=aYrH,L=4H, =U4d=H,L, a adLN=t 4LN(, aNH,HRr)H,L
, GUrWH4=
TmycC- n nDnhCPkwol

TwkDTi b MwD	pn1Pe scPl	TCb 1DnT	ol MDwOTucP 1I PwoTuTf cP PI MDwOT ylv yf plv	b wkTPmyn - woPnoP svl	CPPny Mhyg DkbkPT: DkfmkpDkbkP DD	1DCTPk DkbkP 1D	1DCTPk kPi lopnJ 1k	pyi pnoTkPi 1wmopTu mcP	- wb 1ynTTkhn TPynogPe sPwoTuTf 0cPl	TPyCko CP cCkDmyn svl	- wockokog 1ynTTmyn s1wmopTuTf kol	b komTow0522 Tknhn svl	pnT- yk1 Pkwo wc TPy CPmb
													8 N=0P4674NE
	9		1 3 20										- DCi ni TCop sT- IFAL4Y=F(4NHF)N=, tLrN=, a
	5		1 3 90										
	/		1 3 902										cCP - DCi s- eIFYNS Tr=aFAL4Y=F(, aN(7HN) H4 7HN) F(4NH
	Q		1 3 50										
	.												
	%												
	:												
	8												TCopi DnCo - DCi s- DIFENSHAL4Y=F(4NH
	B												
	92												P4HEp, 6HS 3 92)H0

o XTPCopCyp 1nonPyCPkwo PnTPynTkTPCo- n
1 X1w- KnP 1nonPywbnPny ynTkTPCo- n
PXPJpwP- won 1nonPyCPkwo ynTkTPCo- n
y Xyw- K- wyn yn- whnyi
yfp Xyw- Kf mCDkPi pnTkgoCPkwo

ynbCyKTI
g1T - wwyplo CPnTI DrH0/ 80/ B28. : FD4=t 0X9590B2: 8B8

LOG OF TEST PIT TP-SUB

TennP 9 4) 9

yy- 14Y, L& n=, Lt RFDD-
892 e, 7H,lL7 - L477N‡t y aFTdNH 952
y 4d=a y 4UkFPJ : 8%89
P, E,6S4=, I s. 95I BB5X528:

- DknoPl	o, GnLr n=, Lt Ry, 74dLU, 7Fk=U0
1ywVn- Pl	- 4LARMnTT
Dw- CPkwol	T4E =4 - 4d=HRF- r EN4L=N
ombMnyl	gn5/ 2%2/ 9

pCPnsTl nJ- ChCPnpl Bu58u5/

nJ- ChCPkwo bnPewpsTll
nGUr W/H4L

gywmopOCPny locwybCPkwol
gL4d=aYrH,L=4H, =U4d=H,L, a adLN‡t 4LN (, aNH,IERr)H,L , GUr W/HM=

TmycC- n nDnhCPkwol

TwkDTi bMwD	pn1Pe scPl	TCb1DnT	ol MDwOTucP 1lPwoTuTf cP Pl MDwOT ylv yfplv	bwkTPmyn - woPnoPsvl	CPPnyMnyg DkbkPT: Dkf mkp DkbkP (DD)	1DCTPk DkbkP (1D)	1DCTPk kPi lopnJ (1k)	pyi pnoTkPi 1wmopTu mOcP	- wb1ynTTkn TPynogPe sPwoTuTf 0cPl	TPyCko CPcKDmyn svl	- wockokog 1ynTTmyn s/1wmopTuTf kol	bkomT ow0522 Tknhn svl	pnT- yk1Pkwo wc TPyCPmb
													99 N‡0P4674NE
	9		1 3 Q0										- DCi ni TCop sT- IFAL4Y=F(4NHF)N‡, tLrN‡, a
	5		1 3 Q0										
	/		1 3 90										
	Q		1 3 50										
	.												
	%												
	:												
	8												1wwyDi Xgy Cpnp TCop sT1IFHLrU, - ERFAL4Y= H4 ENSHAL4Y=F (4NHH4 Y, HF)N‡, tLrN‡, a
	B												
	92												P4HEp, 6HS 3 92)H0

ckn Dp pCPC — DCMwyCPwyi pCPC

o XTPCopCyp 1nonPyCPkwo PnTPynTkTPCo- n
1 X1w- KnP 1nonPywbnPny ynTkTPCo- n
PXPJpwP- won 1nonPyCPkwo ynTkTPCo- n
y Xyw- K- wyn yn- whnyi
yfp Xyw- Kf mCDkPi pnTkgoCPkwo

ynbCyKTl
g1T - wwyplo CPnTl DrH0/ 80/ BQ2QQFD4=t 0X9590B28. 98

ynonOCMDn Dwg XPnTP1kP XDwg Cgoo D290gpP X92Q5/ 9/ IG5 Xy llw1nyCPkwoTw15\25 pnTkgo\g nwPn- eok CDg pykn\g koP1ywVn- PT\525/ \- wyM MnTT Xgn5/ 2%2/ 9\- wyM MnTT Xgn5/ 2%2/ 9\g 1V

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810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78661
512.992.2087

APPENDIX B

SUMMARY OF LABORATORY RESULTS

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Borehole	Depth (ft)	USCS	Water Content (%)	Dry Unit Weight (pcf)	< No. 200	LL	PL	PI	Compressive Strength (tsf)	Strain at Failure (%)	Confining Pressure (psi)	Organic Matter (%)	Chlorides (%/weight)	Sulfates (%/weight)	pH	Minimum Resistivity (ohm-cm)
B-01	1.0		20									2.4				
B-01	1.0												0.0002	0.0018	8.5	871
B-01	3.5		20		69											
B-01	6.0	CL	19	111	66	43	13	30	3.91	8.3	0.0					
B-01	12.0		25			61	21	40								
B-02	1.0	CL	21	108	56	47	15	32								
B-02	1.0	CL	17		67	40	16	24								
B-02	3.5		21													
B-02	6.0	CL	20		75	42	16	26								
B-02	9.0		22													
B-03	1.0	CL	19		79	41	14	27								
B-03	3.5	CH	21	95	75	53	13	40								
B-03	6.0		18		41											
B-03	9.0		20													
B-03	14.0		30		67											
B-04	1.0	CL	21	106	65	47	16	31								
B-04	1.0	CL	16		61	32	14	18								
B-04	1.0												0.0006	0.0023	8.2	2,077
B-04	3.5		19													
B-04	9.0		21			39	15	24								
B-04	14.0		23													
B-05	1.0		16									1.8				
B-05	3.5	CL	18	106	76	40	15	25								
B-05	6.0	CL	21		86	48	18	30								
B-05	9.0		25													
B-05	12.0		21													
B-06	1.0	CL	17	111	58	44	14	30								
B-06	1.0	CL	16		67	38	13	25								
B-06	3.5		18									2.2				
B-06	6.0	CH	22		91	52	19	33								
B-06	9.0		27		36											

ND Not Detected;
*Denotes Total Unit Weight

SUMMARY OF LABORATORY RESULTS

RRC Power & Energy, LLC
810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78681
Telephone: (512) 992-2087

CLIENT: NextEra Energy Resources, Inc.
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

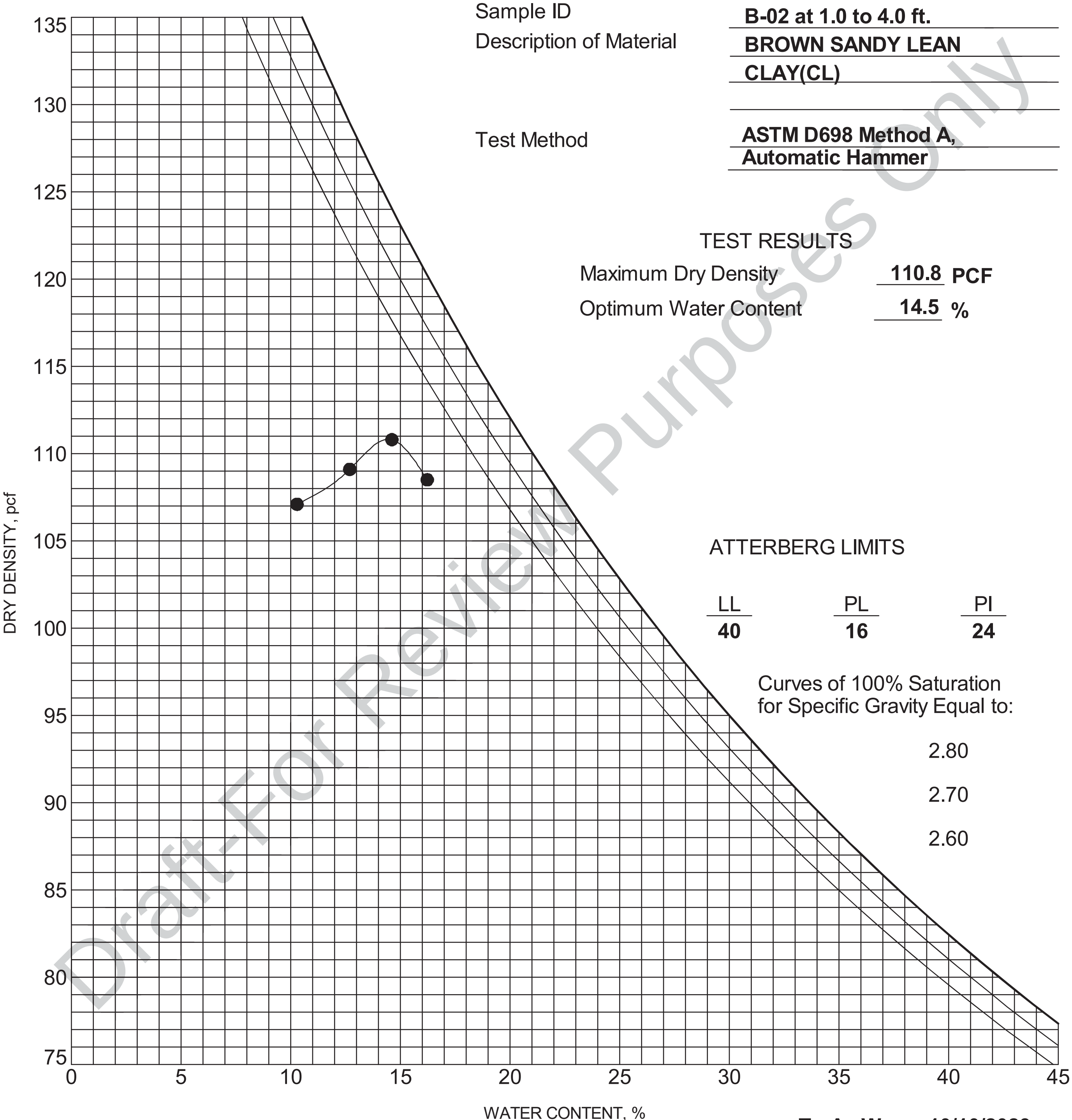
Borehole	Depth (ft)	USCS	Water Content (%)	Dry Unit Weight (pcf)	< No. 200	LL	PL	PI	Compressive Strength (tsf)	Strain at Failure (%)	Confining Pressure (psi)	Organic Matter (%)	Chlorides (%/weight)	Sulfates (%/weight)	pH	Minimum Resistivity (ohm-cm)
B-06	12.0		19													
B-07	1.0	CL	24		65	44	15	29								
B-07	1.0												0.0006	0.0023	8.4	804
B-07	2.5	CH	24	100	89	51	20	31	1.26	1.8	0.0					
B-07	4.5		26		82											
B-07	9.0		17													
B-07	12.0		19													
B-08	1.0	CL	21		66	39	15	24								
B-08	6.0		18		65											
B-08	9.0		21		29											
B-09	3.5	CH	23		86	53	22	31								
B-09	6.0		19		56											
B-09	9.0		27		37											
B-09	12.0		24		60											
B-10	1.0	CH	23	103	87	55	17	38								
B-10	1.0	CL	19		74	36	15	21								
B-10	1.0												0.0020	0.0037	8.0	1,273
B-10	3.5	CL	20		85	47	21	26								
B-10	6.0		21													
B-10	9.0		20		73											
B-10	12.0		22													

ND Not Detected;
*Denotes Total Unit Weight

MOISTURE-DENSITY RELATIONSHIP

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031



Sample ID: **B-02 at 1.0 to 4.0 ft.**

Description of Material: **BROWN SANDY LEAN CLAY(CL)**

Test Method: **ASTM D698 Method A, Automatic Hammer**

TEST RESULTS

Maximum Dry Density **110.8** **PCF**

Optimum Water Content **14.5** **%**

ATTERBERG LIMITS

LL	PL	PI
40	**16**	**24**

Te-An Wang, 10/10/2023

Analysis & Quality Review/Date

Specimens prepared by: T.W.

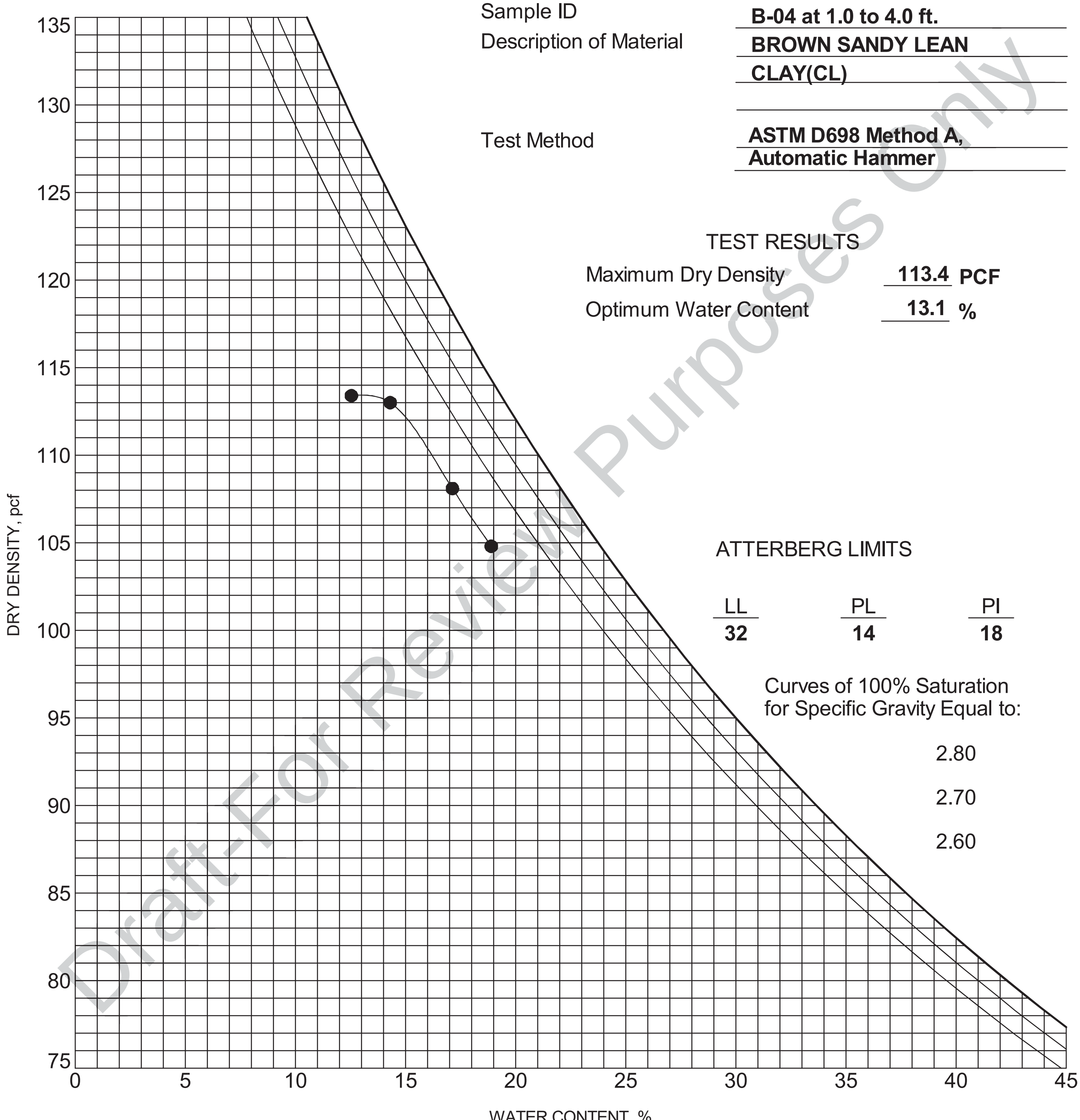
MOISTURE-DENSITY RELATIONSHIP

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-04 at 1.0 to 4.0 ft.**
Description of Material: **BROWN SANDY LEAN CLAY(CL)**

Test Method: **ASTM D698 Method A, Automatic Hammer**



TEST RESULTS

Maximum Dry Density **113.4 PCF**
Optimum Water Content **13.1 %**

ATTERBERG LIMITS

LL	PL	PI
32	14	18

Curves of 100% Saturation for Specific Gravity Equal to:

2.80
2.70
2.60

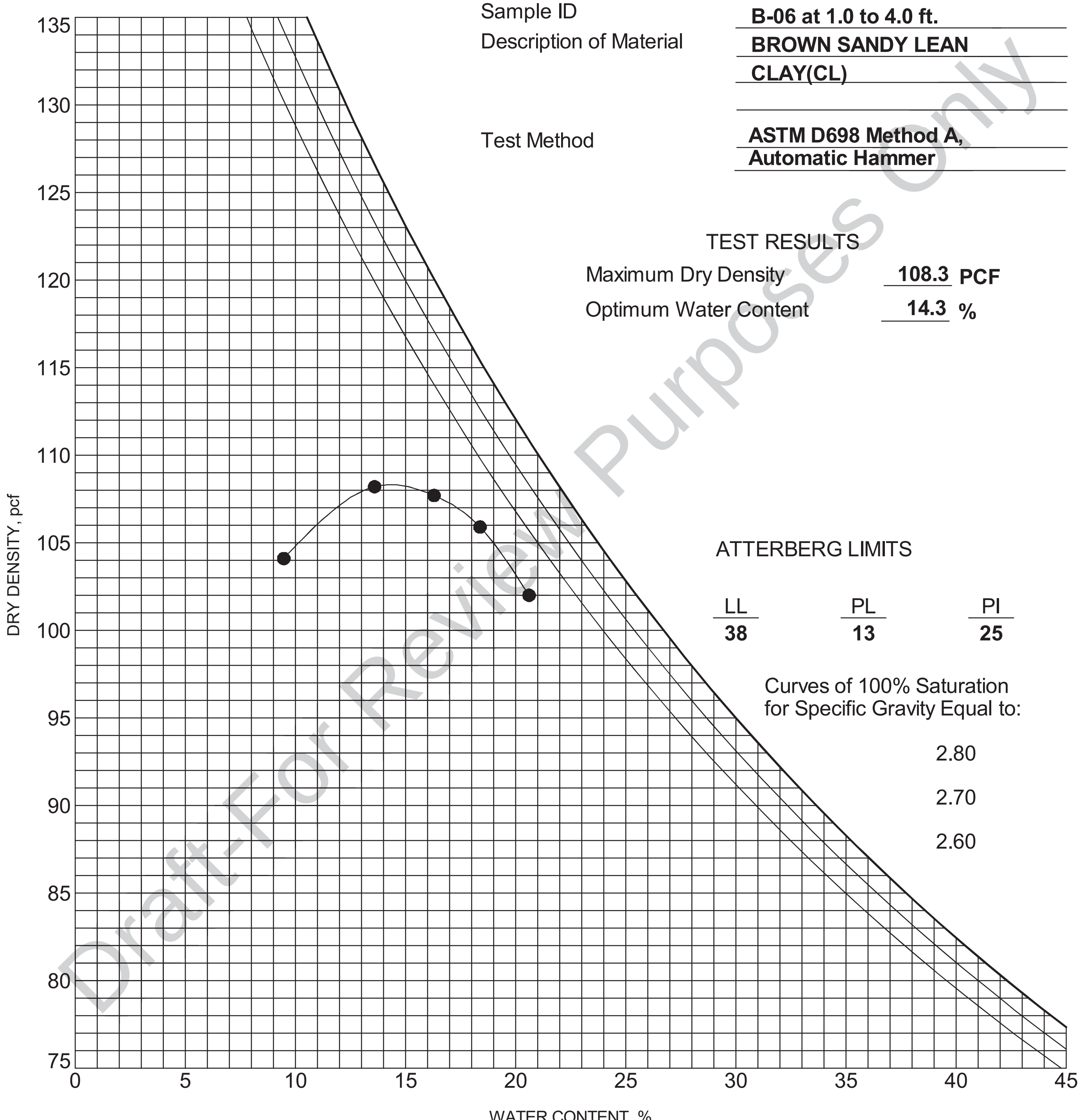
Te-An Wang, 10/10/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

MOISTURE-DENSITY RELATIONSHIP

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031



Sample ID: **B-06 at 1.0 to 4.0 ft.**

Description of Material: **BROWN SANDY LEAN CLAY(CL)**

Test Method: **ASTM D698 Method A, Automatic Hammer**

TEST RESULTS

Maximum Dry Density **108.3** **PCF**

Optimum Water Content **14.3** **%**

ATTERBERG LIMITS

LL	PL	PI
38	**13**	**25**

Curves of 100% Saturation for Specific Gravity Equal to:

2.80

2.70

2.60

Te-An Wang, 10/10/2023

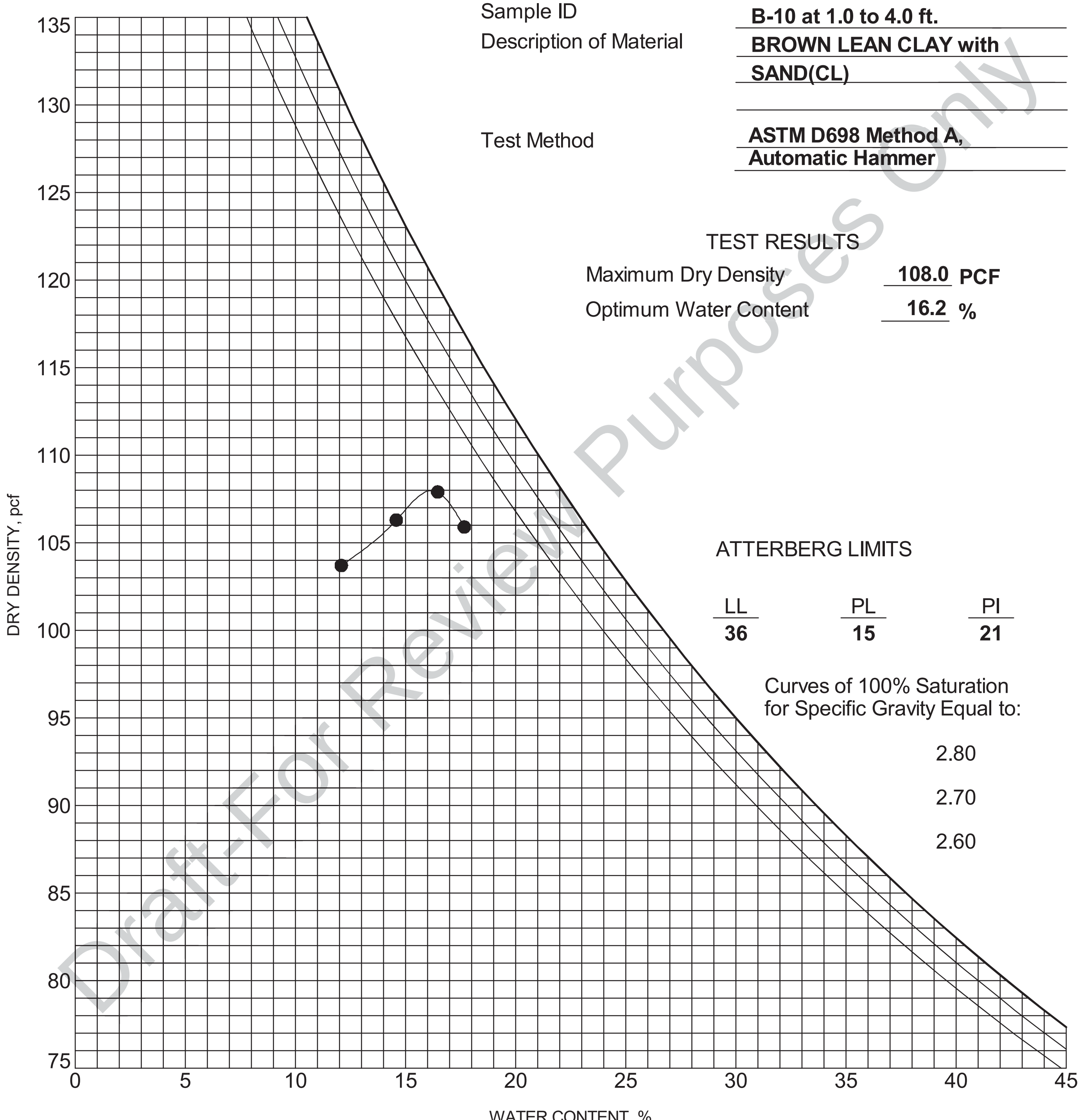
Analysis & Quality Review/Date

Specimens prepared by: T.W.

MOISTURE-DENSITY RELATIONSHIP

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031



Sample ID: **B-10 at 1.0 to 4.0 ft.**

Description of Material: **BROWN LEAN CLAY with SAND(CL)**

Test Method: **ASTM D698 Method A, Automatic Hammer**

TEST RESULTS

Maximum Dry Density **108.0 PCF**

Optimum Water Content **16.2 %**

ATTERBERG LIMITS

LL	PL	PI
36	**15**	**21**

Te-An Wang, 10/11/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

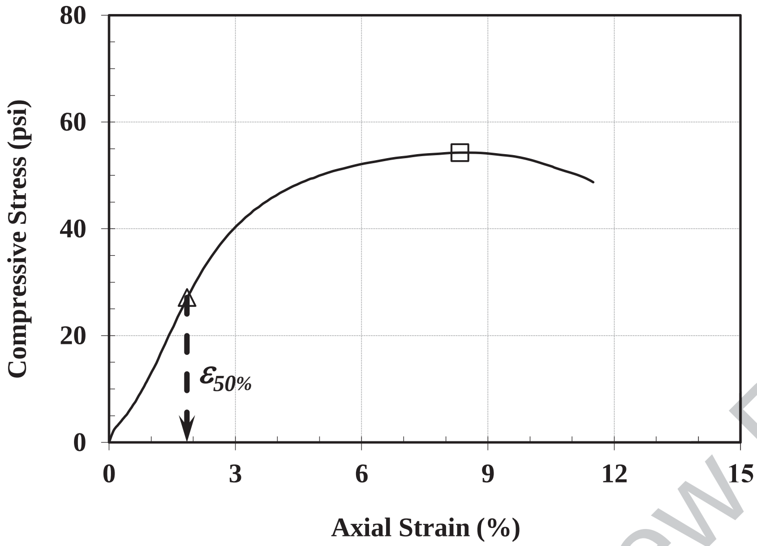
Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

Unconfined Compression Test Report

Client: RRC Power & Energy, LLC
Project: Corby BESS
Sample I.D.: B-1 at 6 ft

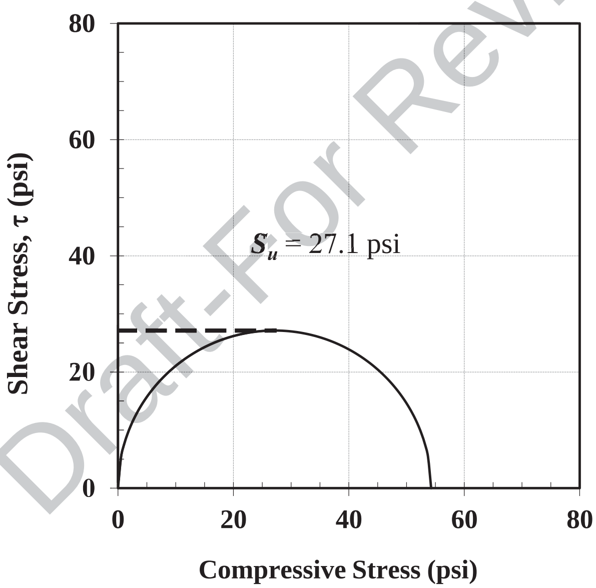
Project No.: GE2306031
Test Method: ASTM D2166
Test Date: 10/3/2023

Type of Specimen: Shelby Tube
Strain Rate: 1.0 %/min



Initial Specimen Conditions		
Avg. Diameter (in)	D_o	2.86
Avg. Height (in)	H_o	5.59
In-situ Moisture Content (%)	w_o	18.6
Total Unit Weight (pcf)	γ_{total}	131.8
Dry Unit Weight (pcf)	γ_{dry}	111.1
Saturation (%)	S_r	97.2
Void Ratio	e_o	0.52
Specific Gravity (Assumed)	G_s	2.70

Mohr Circles for Peak Stress at Failure



Stresses at Failure	
Unconfined Compressive Strength, q_u (psi)	54.3
Axial Strain at Failure (%)	8.3
Axial Strain at 50 % of q_u (%)	1.9
Total Stresses at Failure	
Major Principal Stress, σ_1 (psi)	54.3
Minor Principal Stress, σ_3 (psi)	0
Undrained Shear Strength, S_u (tsf)	1.95

Note: Failure was determined at the maximum deviator stress or deviator stress at 15 % axial strain, whenever is obtained first.

Te-An Wang, EIT, 10/10/23
Quality Review/Date
Specimen prepared & tested by: A.B.

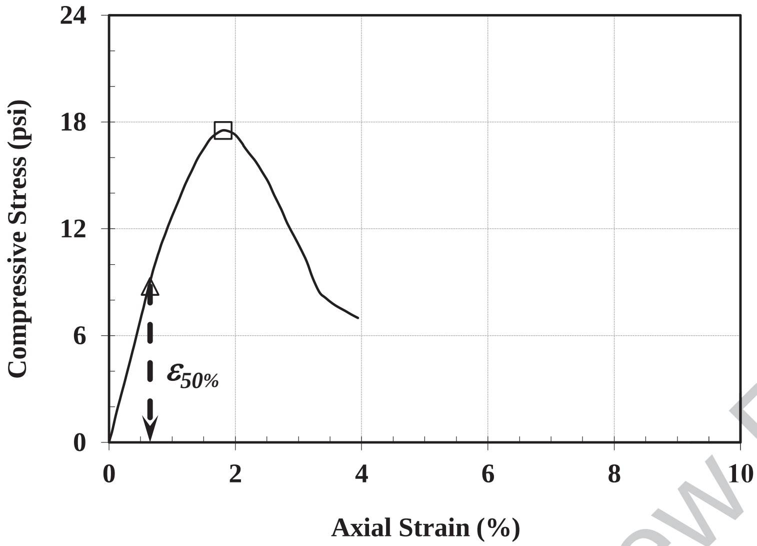
Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

Unconfined Compression Test Report

Client: RRC Power & Energy, LLC
Project: Corby BESS
Sample I.D.: B-7 at 2.5 ft

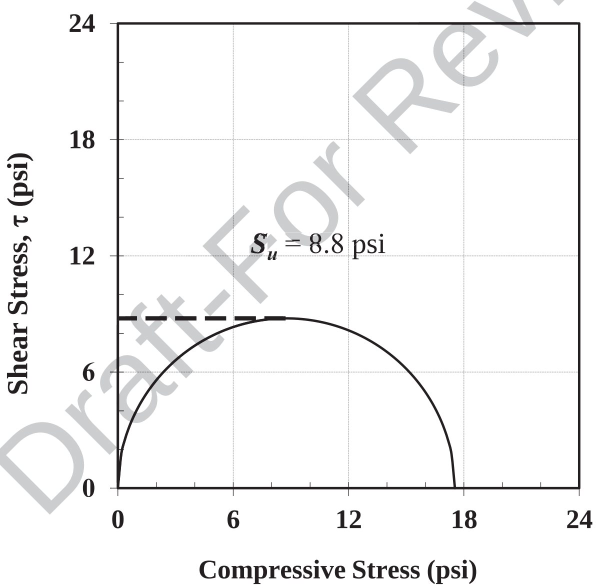
Project No.: GE2306031
Test Method: ASTM D2166
Test Date: 10/3/2023

Type of Specimen: Shelby Tube
Strain Rate: 1.0 %/min



Initial Specimen Conditions		
Avg. Diameter (in)	D_o	2.86
Avg. Height (in)	H_o	5.60
In-situ Moisture Content (%)	w_o	24.4
Total Unit Weight (pcf)	γ_{total}	124.2
Dry Unit Weight (pcf)	γ_{dry}	99.9
Saturation (%)	S_r	95.6
Void Ratio	e_o	0.69
Specific Gravity (Assumed)	G_s	2.70

Mohr Circles for Peak Stress at Failure



Stresses at Failure	
Unconfined Compressive Strength, q_u (psi)	17.5
Axial Strain at Failure (%)	1.8
Axial Strain at 50 % of q_u (%)	0.7
Total Stresses at Failure	
Major Principal Stress, σ_1 (psi)	17.5
Minor Principal Stress, σ_3 (psi)	0
Undrained Shear Strength, S_u (tsf)	0.63

Note: Failure was determined at the maximum deviator stress or deviator stress at 15 % axial strain, whenever is obtained first.

Te-An Wang, EIT, 10/10/23
Quality Review/Date
Specimen prepared & tested by: A.B.

Beyond Engineering Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

One-Dimensional Swell or Collapse of Soils

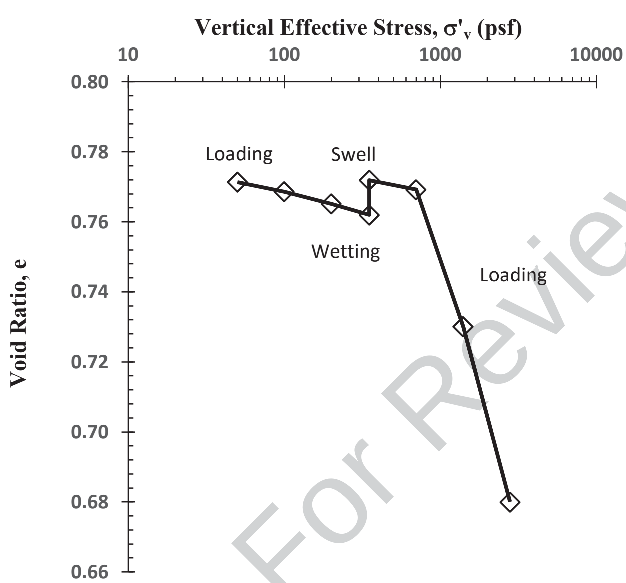
Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby BESS	**Test Method:**	ASTM D4546, Method C
Sample ID:	B-3 at 3.5 ft	**Test Date:**	10/6/23-10/11/23

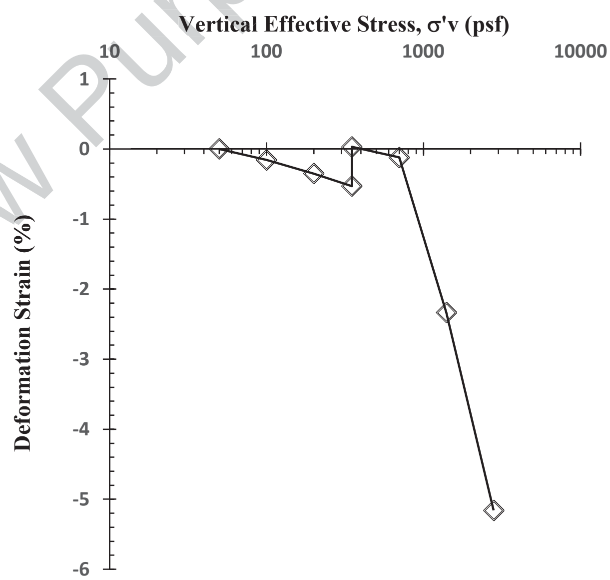
Specimen Conditions	
Avg. Water Content of Trimmings (%)	21.1
Final Specimen Water Content (%)	25.1
Initial Specimen Height (in)	0.796
Final Specimen Height (in)	0.754
Initial Dry Unit Weight, γ_o (pcf)	95.1
Final Dry Unit Weight, γ_f (pcf)	100.3
Initial Void Ratio, e_o	0.771
Final Void Ratio, e_f	0.680
Initial Degree of Saturation (%)	73.9
Final Degree of Saturation (%)	99.6
Swell Strain (%)	0.6

Specimen was inundated with tap water during testing. Loading increment duration was minimum 24 hours. . The calculation included the machine deflections that measured in each loading steps.

Gs assumed to be 2.70

Specimen Diameter: 2.496 inches





	Dry				Inundated			
Stage No.	1	2	3	4	5	6	7	8
σ'_v (psf)	50	100	200	350	350	700	1400	2800
Height (inch)	0.796	0.794	0.793	0.791	0.796	0.795	0.777	0.754
Void Ratio, e	0.771	0.769	0.765	0.762	0.772	0.769	0.730	0.680
Axial Strain (%)	0.00	-0.16	-0.35	-0.53	0.03	-0.12	-2.33	-5.16

Huamiao Cao, P.E. 10/13/23

Analysis & Quality Review/Date

Specimen prepared and tested by: J.Z.

Beyond Engineering Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

One-Dimensional Swell or Collapse of Soils

Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby BESS	**Test Method:**	ASTM D4546, Method C
Sample ID:	B-3 at 3.5 ft	**Test Date:**	10/6/23-10/11/23

Beyond Engineering Testing, LLC
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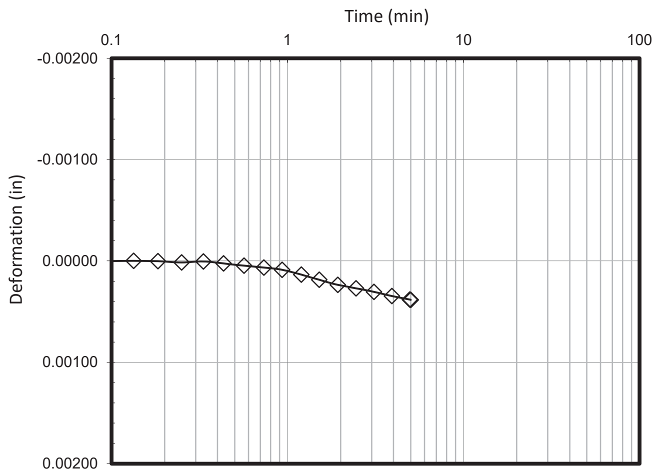
Round Rock, TX 78664
(512) 358-6048

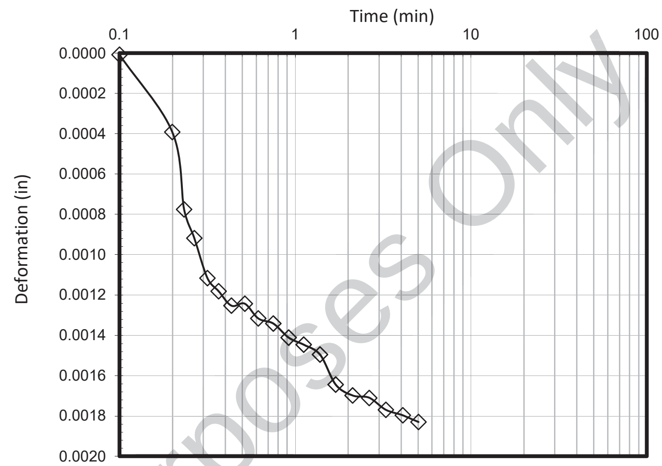
One-Dimensional Swell or Collapse of Soils

Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby BESS	**Test Method:**	ASTM D4546, Method C
Sample ID:	B-3 at 3.5 ft	**Test Date:**	10/6/23-10/11/23

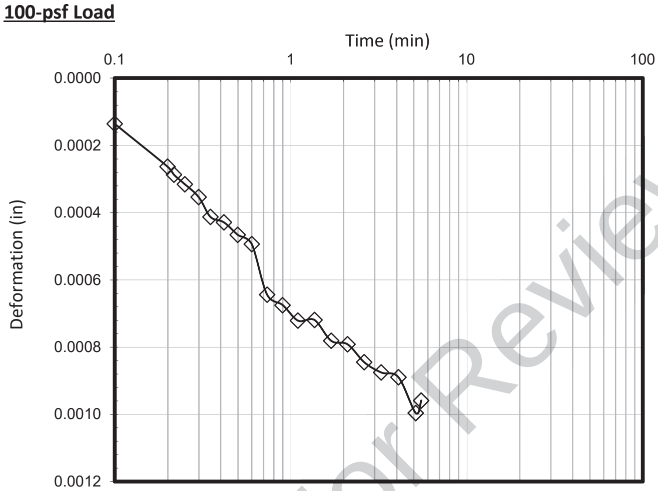
50-psf Load (Seating Load)



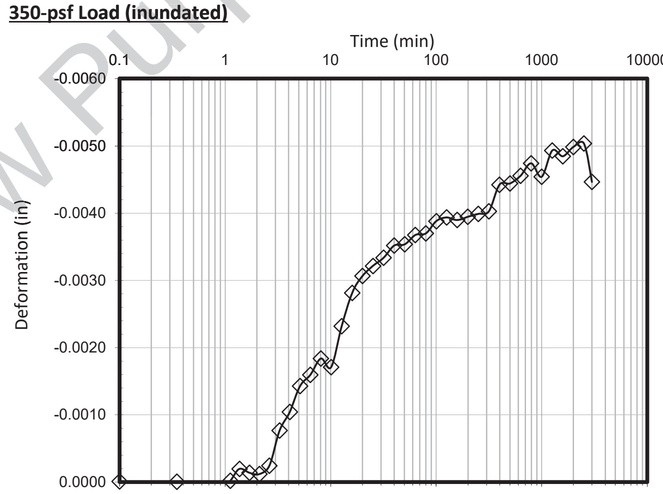
350-psf Load



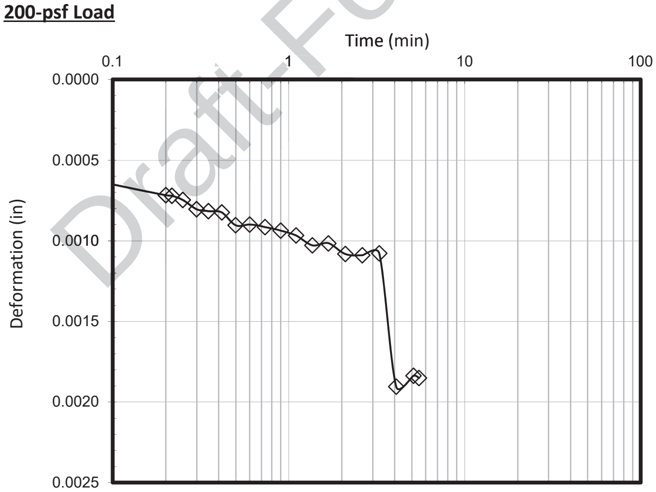
100-psf Load



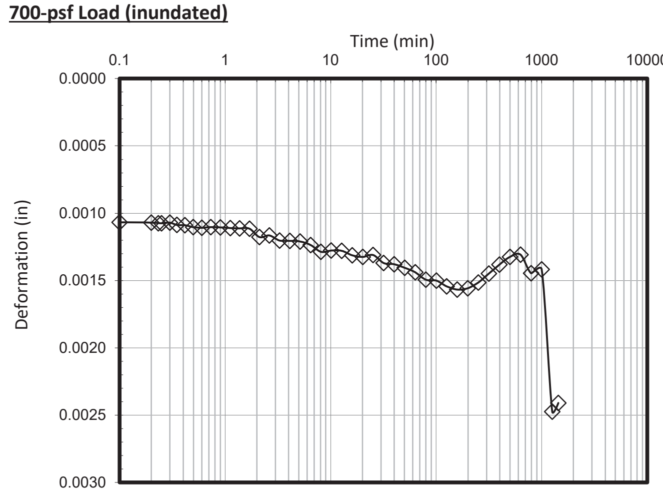
350-psf Load (inundated)



200-psf Load



700-psf Load (inundated)



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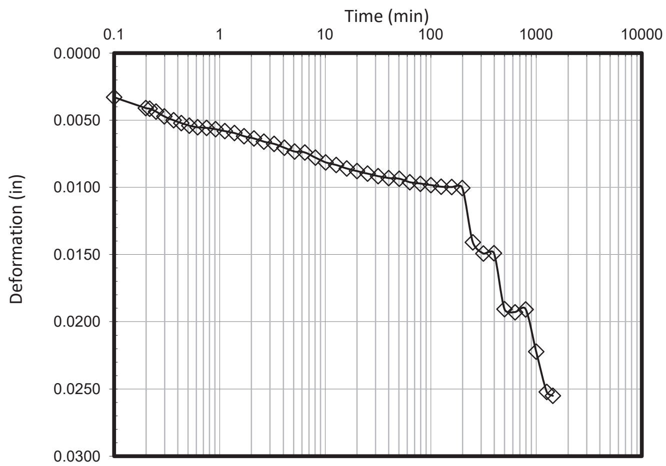
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

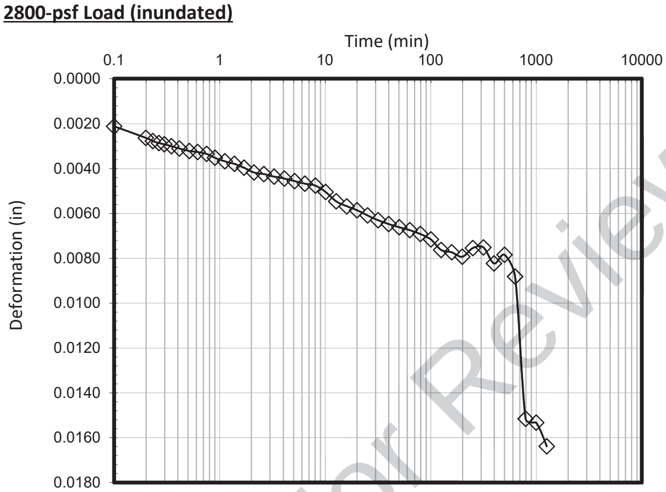
One-Dimensional Swell or Collapse of Soils

Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby BESS	**Test Method:**	ASTM D4546, Method C
Sample ID:	B-3 at 3.5 ft	**Test Date:**	10/6/23-10/11/23

1400-psf Load (inundated)



2800-psf Load (inundated)



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Round Rock, TX 78664
(512) 358-6048

One-Dimensional Swell or Collapse of Soils

Client: RRC Power & Energy, LLC
Project Name: Corby ' ESS
Sample ID: ' -5 at 3.5 ft

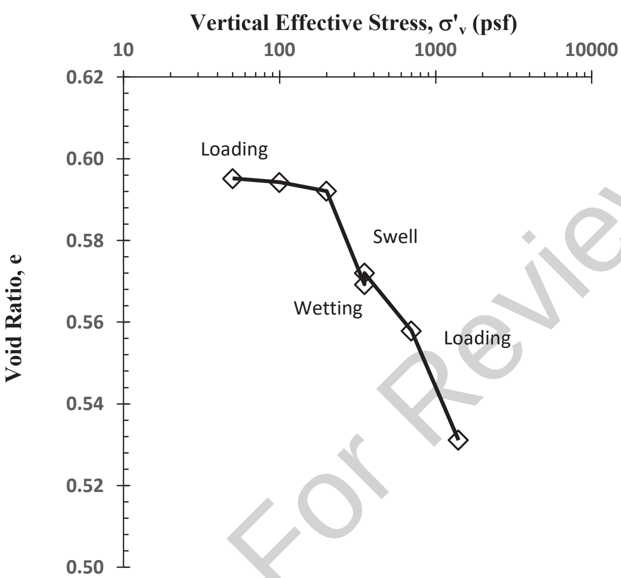
Beyond Project No.: GE2306031
Test Method: ASI N D4546, N ethod C
Test Date: 10V6V23-10V10V23

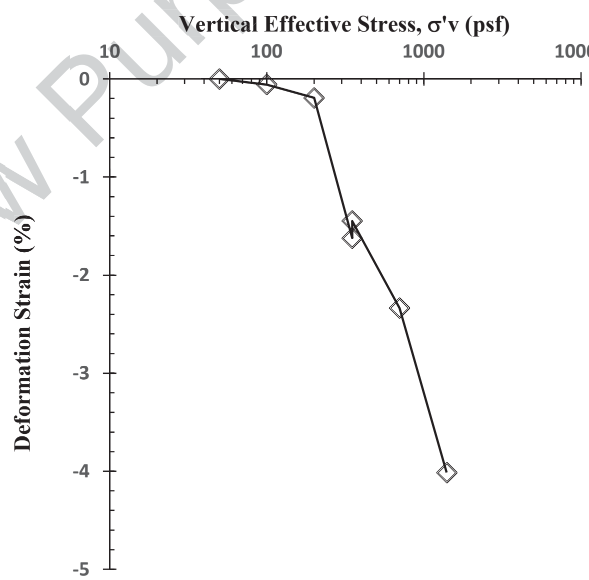
Specimen Conditions	
ATg. Z ater Content of I rimmings (%)	18.3
xinal Specimen Z ater Content (%)	19.1
Jnitial Specimen Ueight (in)	0.789
xinal Specimen Ueight (in)	0.757
Jnitial Dry Mnit Z eight, γ_o (pcf)	105.6
xinal Dry Mnit Z eight, γ_f (pcf)	110.0
Jnitial Woid Ratio, e_o	0.595
xinal Woid Ratio, e_f	0.531
Jnitial Degree of Saturation (%)	82.9
xinal Degree of Saturation (%)	97.3
Swell Strain (%)	0.2

Specimen was inundated with tap water during testing. Loading increment duration was minimum 24 hours. . I he calculation included the machine deflections that measured in each loading steps.

Gs assumed to be 2.70

Specimen Diameter: 2.496 inches





	Dry				Jnundated		
Stage F o.	1	2	3	4	5	6	7
σ'_{v} (psf)	50	100	200	350	350	700	1400
Ueight (inch)	0.789	0.789	0.787	0.776	0.778	0.771	0.757
Woid Ratio, e	0.595	0.594	0.592	0.569	0.572	0.558	0.531
AHial Strain (%)	0.00	-0.06	-0.19	-1.62	-1.45	-2.34	-4.01

Uuamiao Cao, P.E. 10V13V23
Analysis & v uality ReTiewVDate
Specimen prepared and tested by: q.z .

Beyond Engineering Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

One-Dimensional Swell or Collapse of Soils

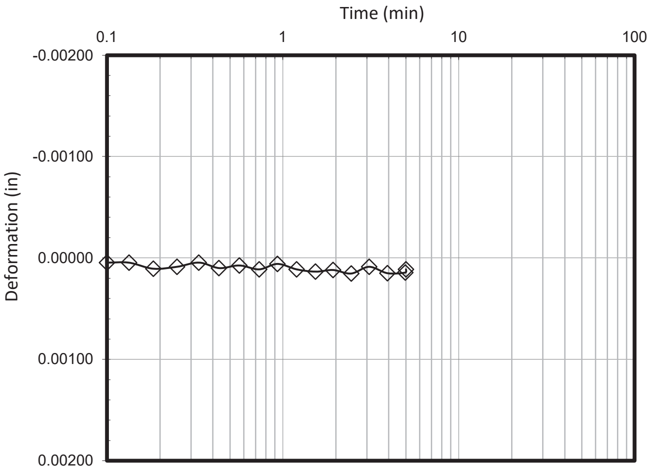
Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby ' ESS	**Test Method:**	ASI N D4546, N ethod C
Sample ID:	' -5 at 3.5 ft	**Test Date:**	10V6V23-10V10V23

Beyond Engineering Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
(512) 358-6048

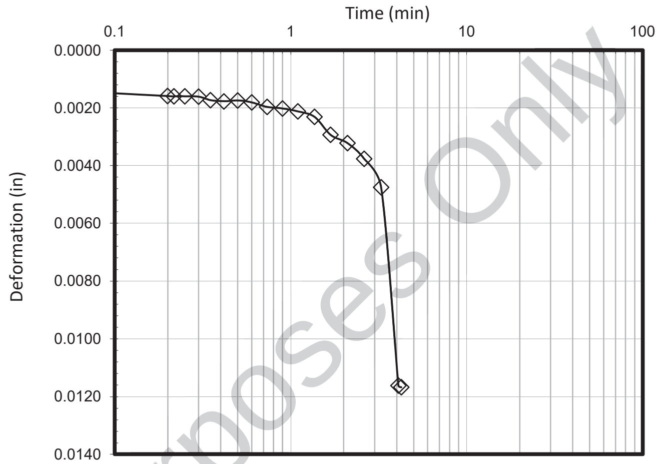
One-Dimensional Swell or Collapse of Soils

Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby ' ESS	**Test Method:**	ASI N D4546, N ethod C
Sample ID:	' -5 at 3.5 ft	**Test Date:**	10V6V23-10V10V23

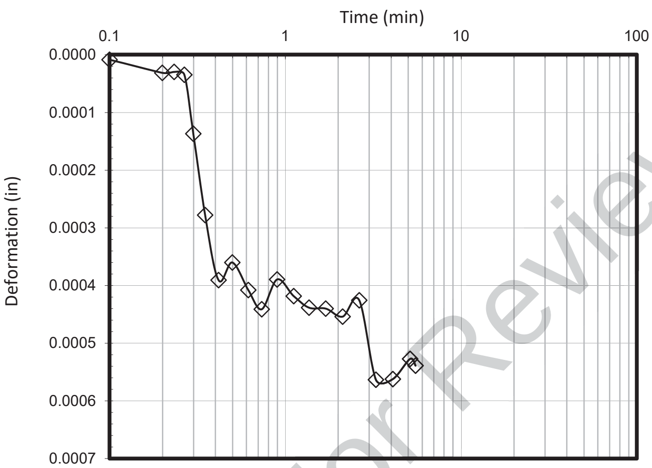
50-psf Load (Seating Load)



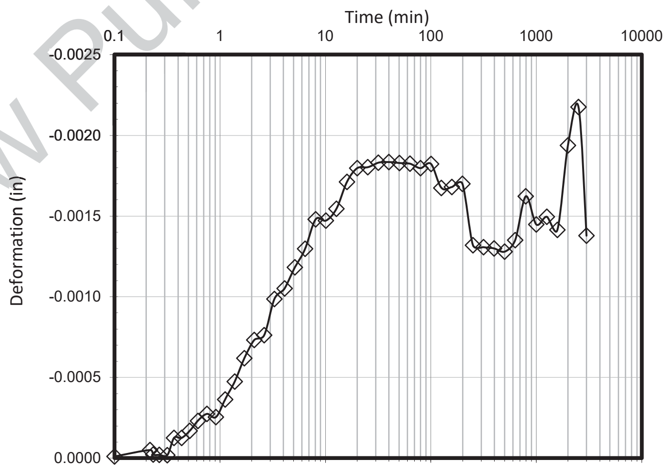
350-psf Load



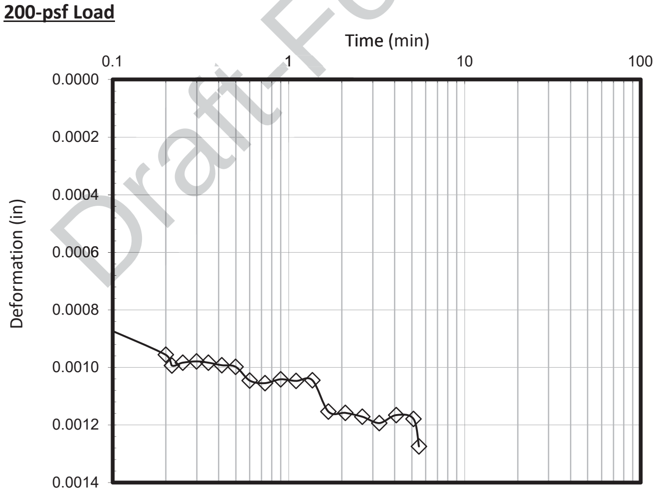
100-psf Load



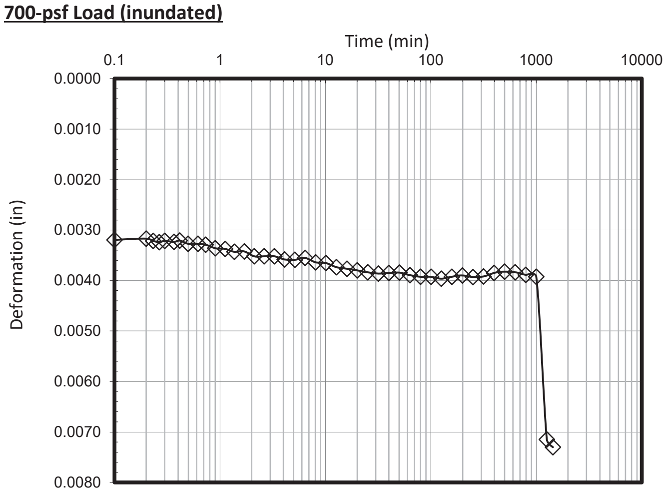
350-psf Load (inundated)



200-psf Load



700-psf Load (inundated)



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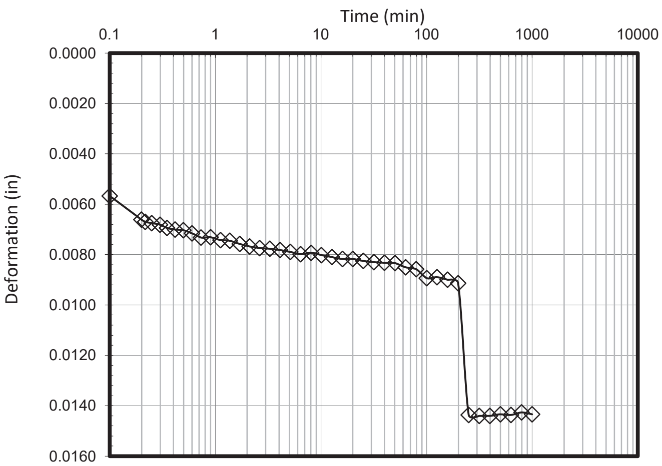
Round Rock, TX 78664
(512) 358-6048

One-Dimensional Swell or Collapse of Soils

Client:	RRC Power & Energy, LLC	**Beyond Project No.:**	GE2306031
Project Name:	Corby ' ESS	**Test Method:**	ASI N D4546, N ethod C
Sample ID:	' -5 at 3.5 ft	**Test Date:**	10V6V23-10V10V23

1400-psf Load (inundated)



Beyond Engineering & Testing, LLC
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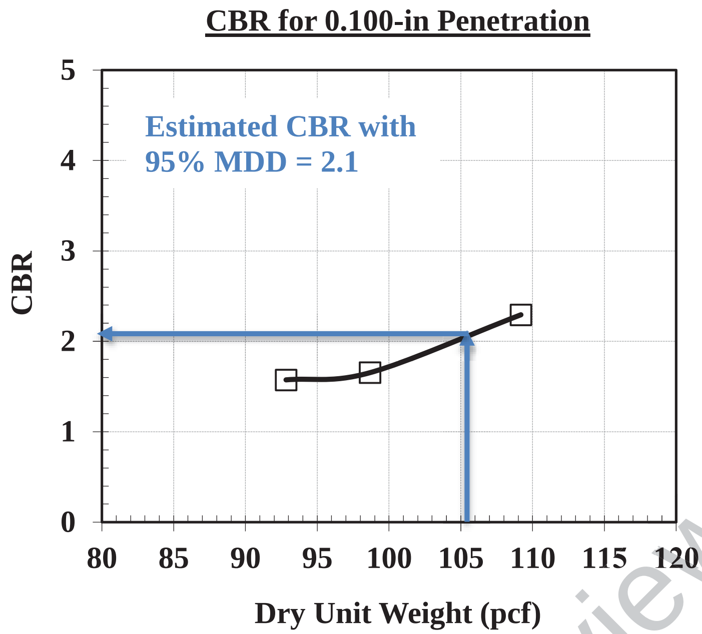
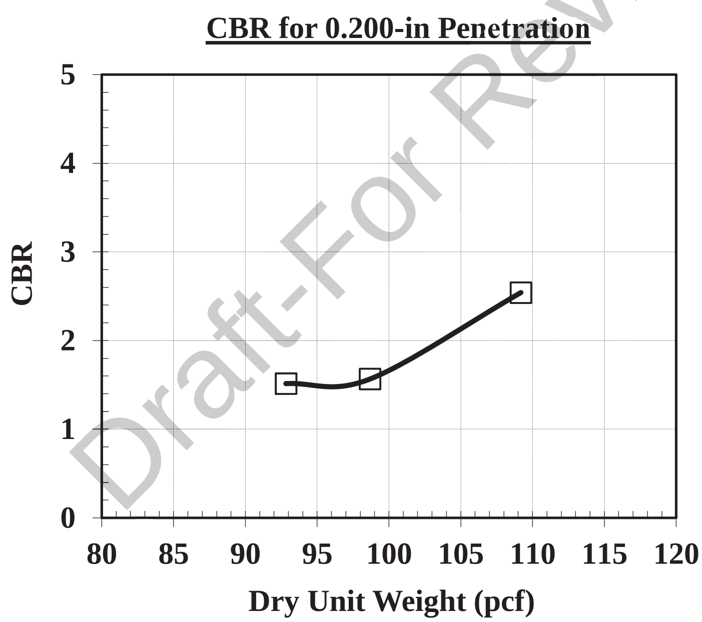
Round Rock, TX 78664
(512) 358-6048

CBR (California Bearing Ratio) Test

Client: RRC Power & Energy, LLC
Project: Corby BESS
Sample No: B-2 at 1-4 ft

Beyond Project No.: GE2306031
Test Method: ASTM D1883
Test Date: 10/18/2023
Rate of Penetration: 0.05 in/min





Initial Conditions			
Specimen No.	1	2	3
Blows per Layer	20	35	60
Surcharge Weight (lbs)	10	10	10
Water Content (%)	22.7	16.3	13.1
Dry Unit Weight (pcf)	92.8	98.7	109.2
Percent Compaction (%)	83.8	89.1	98.5
Final Conditions (soaked)			
Water Content (%) at top 1-in layer after soaking	30.4	30.5	26.4
Swell (% of initial height)	2.4	2.3	2.8
Bearing Ratio of Sample at 0.100 in penetration	1.6	1.7	2.3
Bearing Ratio of Sample at 0.200 in penetration	1.5	1.6	2.5

Note: Soil specimens were molded to a range of densities using 20, 35 and 60 blows at optimum moisture content as per ASTM D 1883 to develop the CBR versus dry density curve. It was allowed the specimens to soak for 96 hrs prior bearing test. Removed the free water and allow the specimens to drain out for 15 min. The 10-lbs surcharge load was placed during bearing test.

Xuewei Ning, 10/19/23

Analysis & Quality Review/Date

Specimens prepared and tested by: A.S.

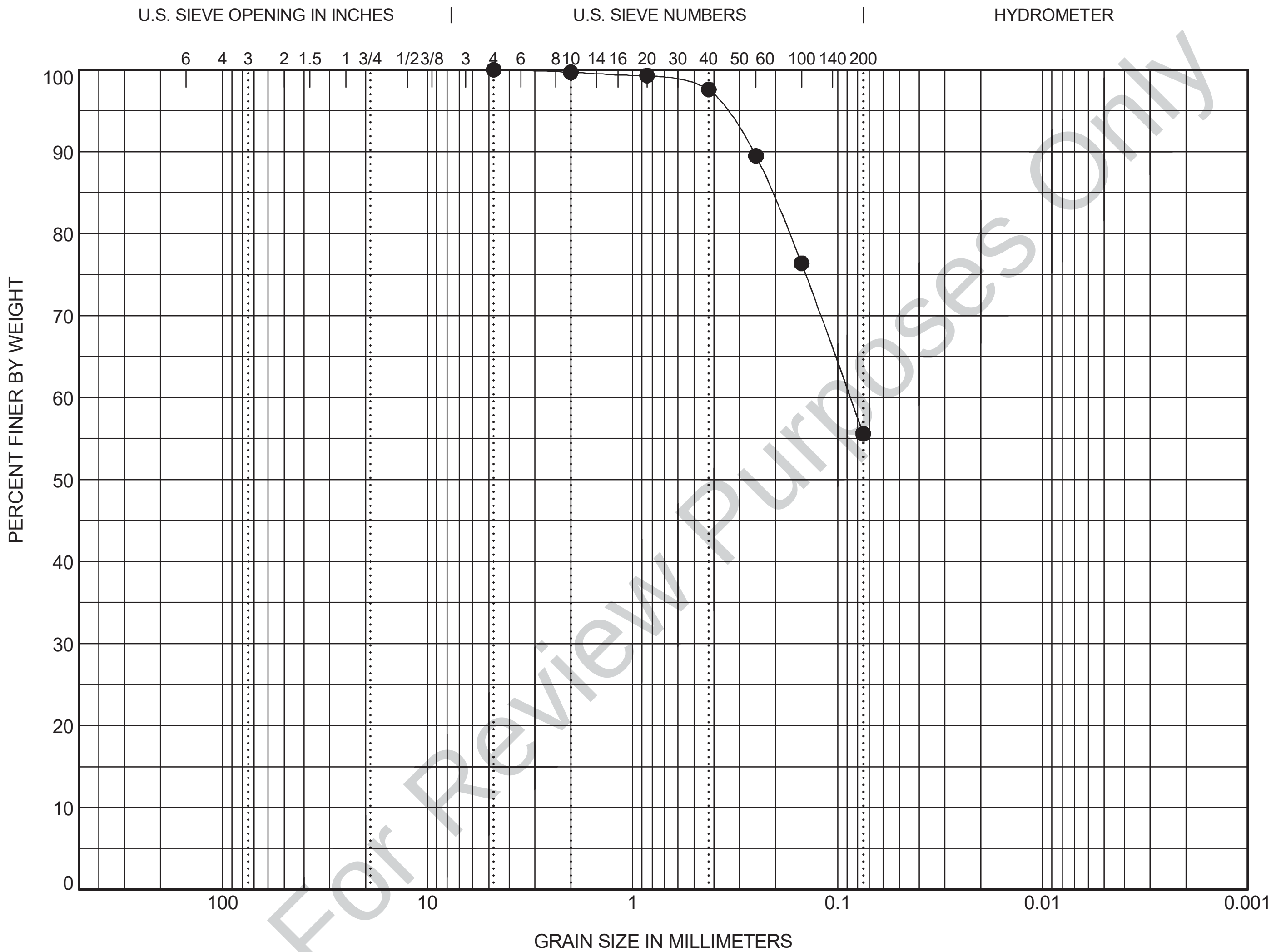
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-02 at 1.0 ft.** Date: 10/24/2023

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**47**	**15**	**32**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75	**0.087**			**0.0**	**44.4**	**55.6**	

Te-An Wang, 10/24/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

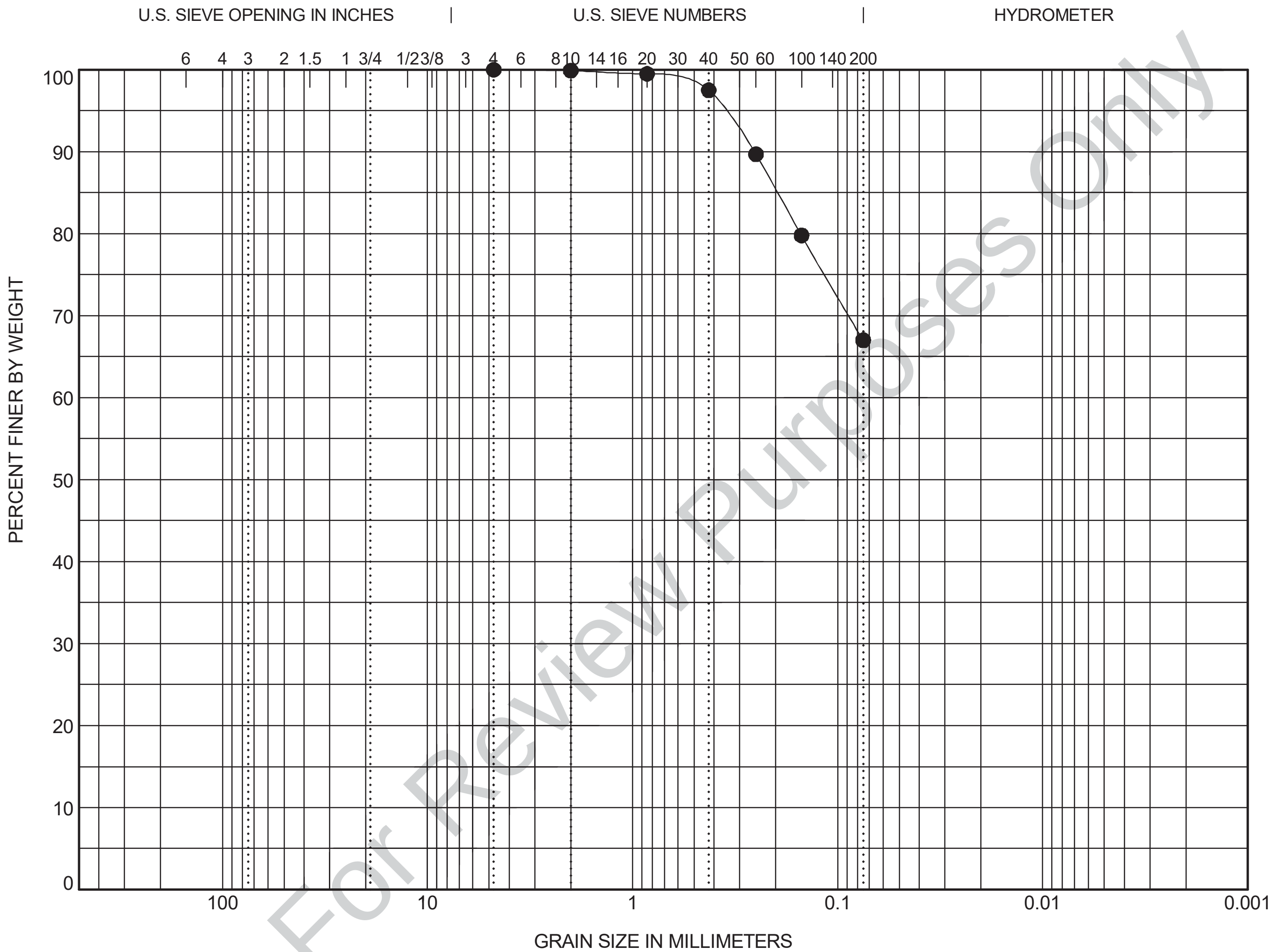
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-02 at 1.0 ft.** Date: ___________

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**40**	**16**	**24**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75				**0.0**	**33.0**	**67.0**	

Te-An Wang,
Analysis & Quality Review/Date

Specimens prepared by: T.W.

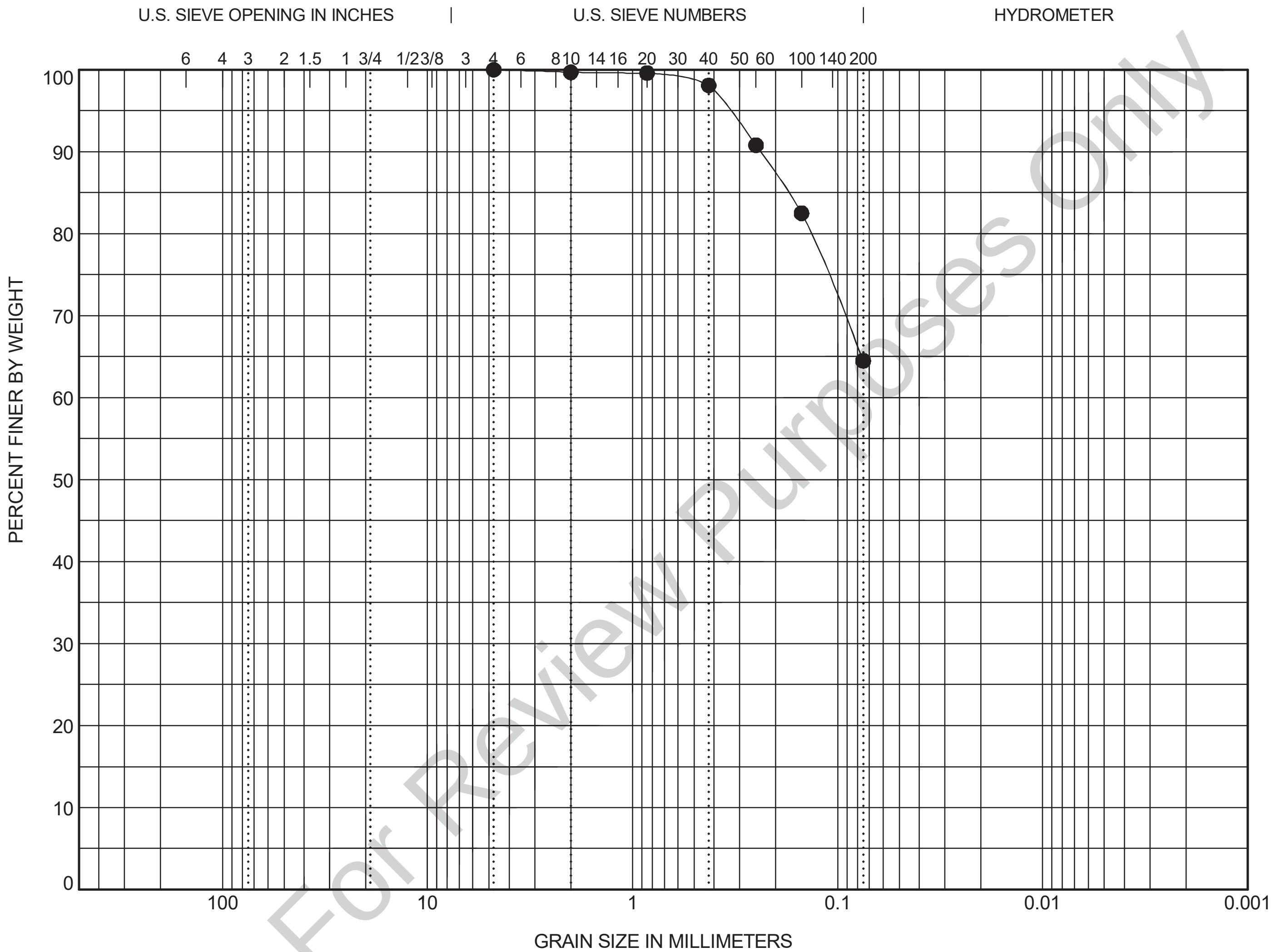
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-04 at 1.0 ft.** Date: 10/24/2023

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**47**	**16**	**31**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75				**0.0**	**35.5**	**64.5**	

Te-An Wang, 10/24/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

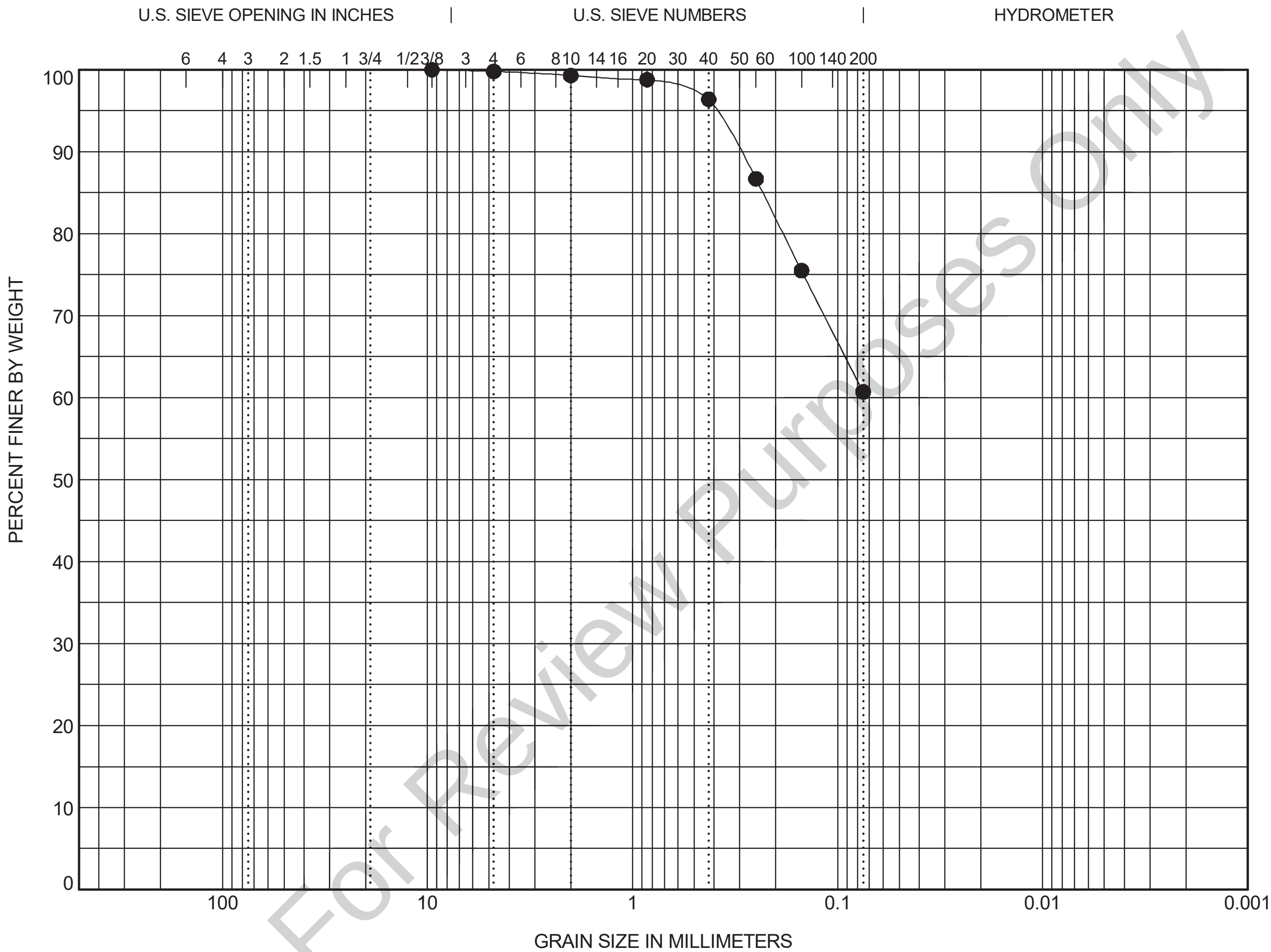
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-04 at 1.0 ft.** Date: ____________

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**32**	**14**	**18**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
9.5				**0.2**	**39.1**	**60.7**	

Te-An Wang,
Analysis & Quality Review/Date

Specimens prepared by: T.W.

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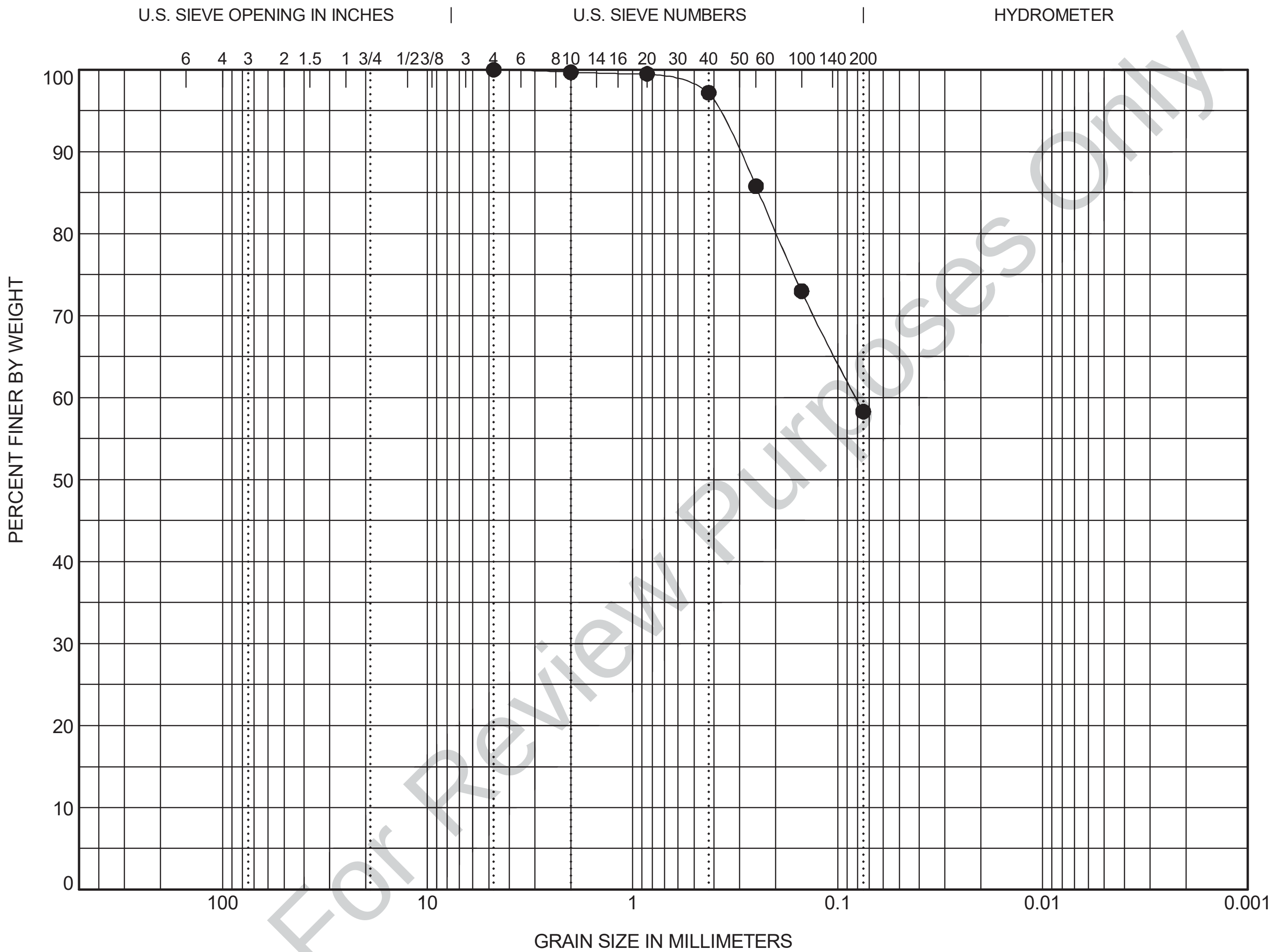
Round Rock, TX 78664
Telephone: (512) 358-6048

GRAIN SIZE DISTRIBUTION

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-06 at 1.0 ft.** Date: 10/24/2023

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**44**	**14**	**30**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75	**0.081**			**0.0**	**41.7**	**58.3**	

Te-An Wang, 10/24/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

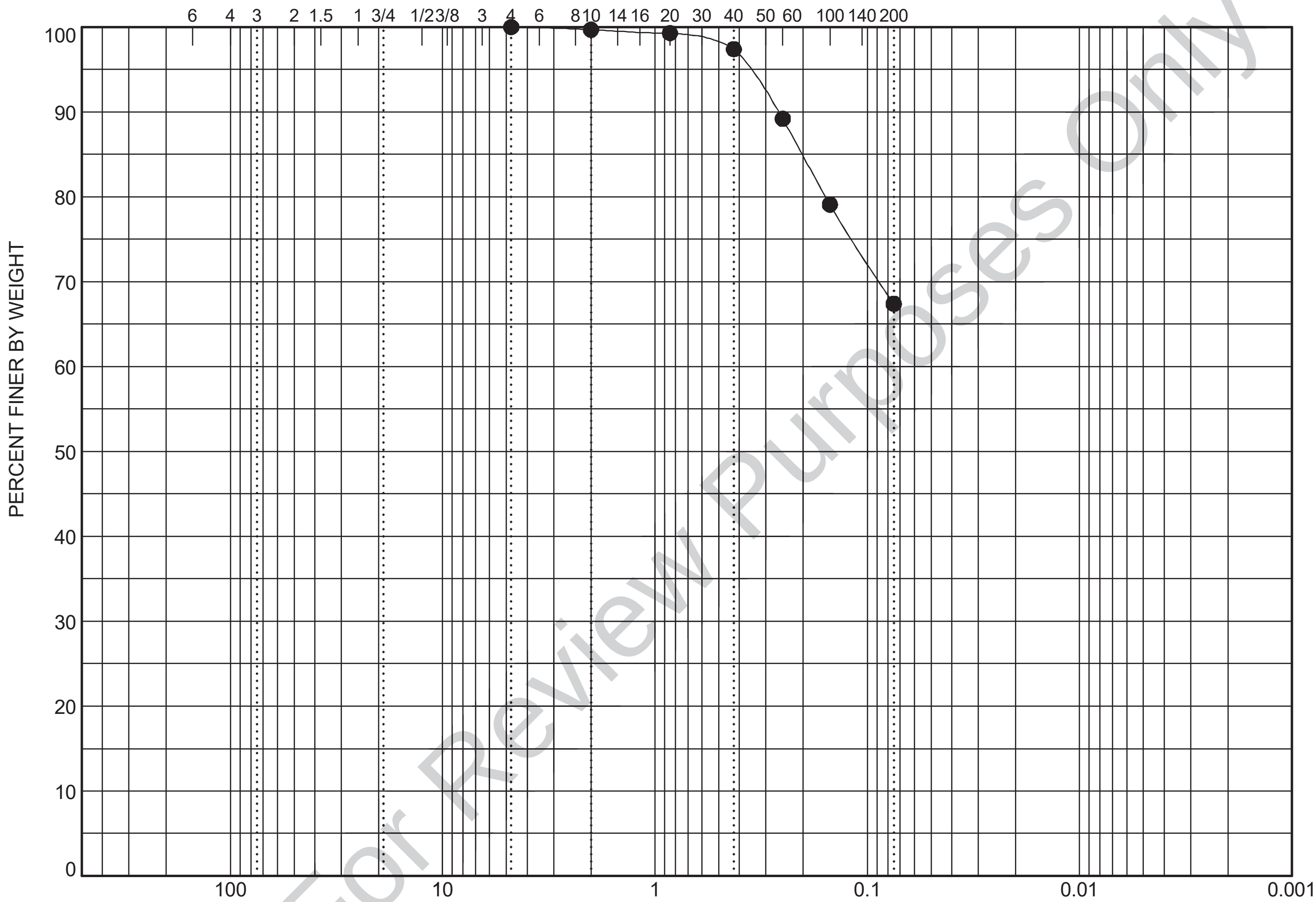
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-06 at 1.0 ft.** Date:

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
SANDY LEAN CLAY(CL)	**38**	**13**	**25**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75				**0.0**	**32.6**	**67.4**	

Te-An Wang,
Analysis & Quality Review/Date

Specimens prepared by: T.W.

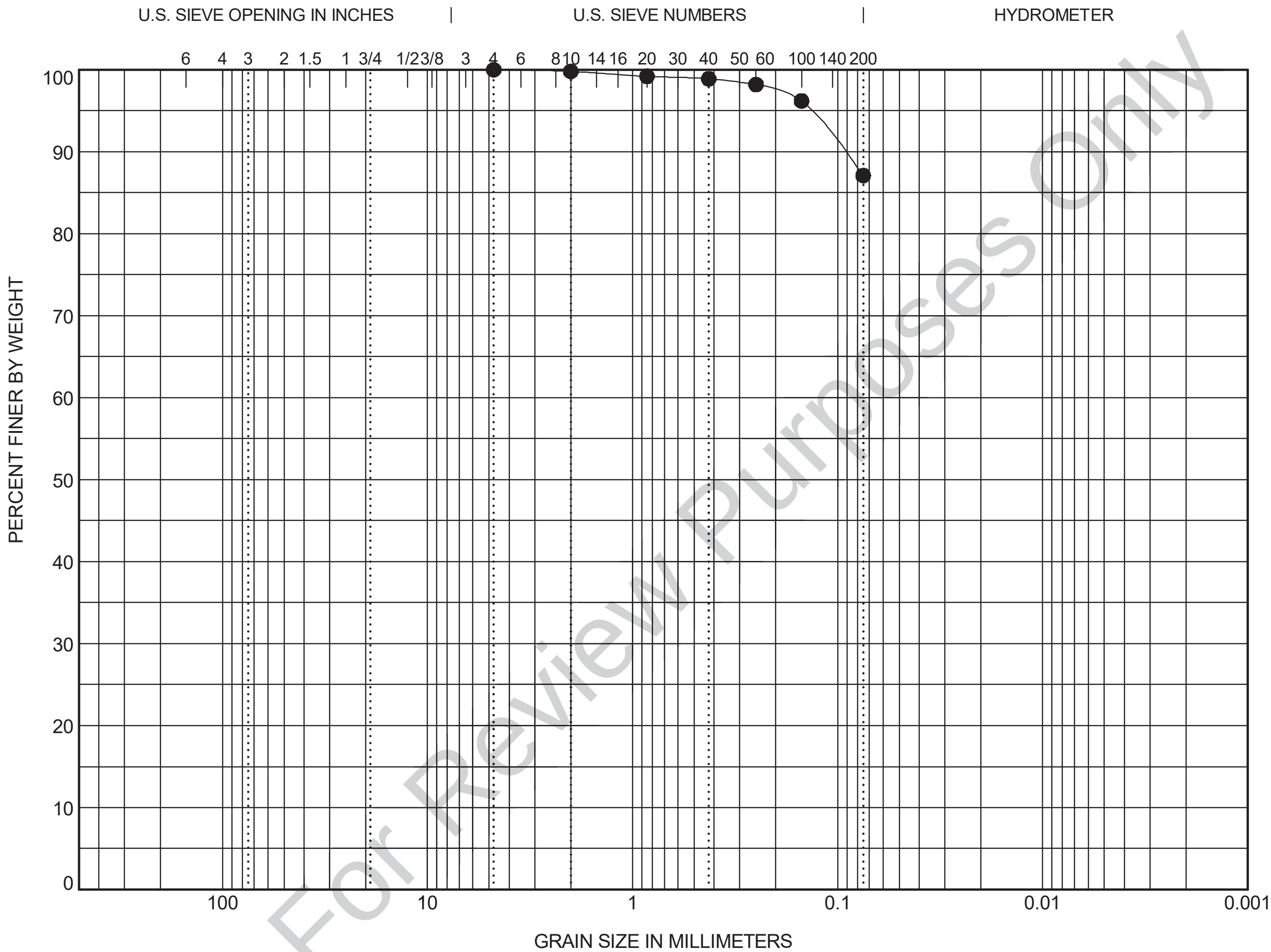
GRAIN SIZE DISTRIBUTION

Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-10 at 1.0 ft.** Date: 10/24/2023

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
FAT CLAY(CH)	**55**	**17**	**38**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75				**0.0**	**12.9**	**87.1**	

Te-An Wang, 10/24/2023
Analysis & Quality Review/Date

Specimens prepared by: T.W.

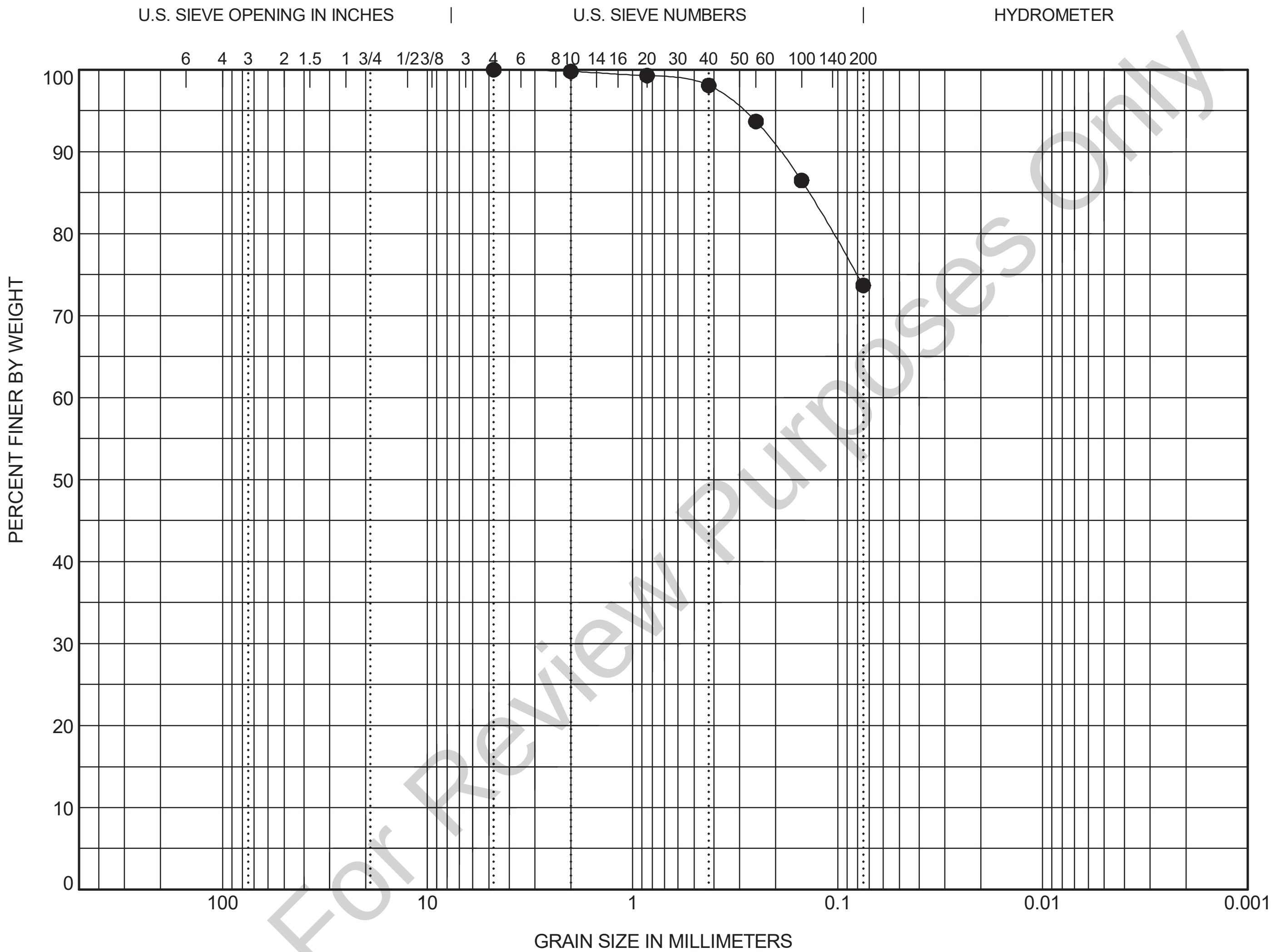
Beyond Engineering & Testing, LLC
3801 Doris Lane, Suite B
Round Rock, TX 78664
Telephone: (512) 358-6048

GRAIN SIZE DISTRIBUTION

CLIENT: RRC Power & Energy, LLC
PROJECT: Corby BESS
LOCATION: Solano County, California
NUMBER: GE2306031

Sample ID: **B-10 at 1.0 ft.** Date:

Test Method: **ASTM D6913**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Classification	LL	PL	PI	Cc	Cu
LEAN CLAY with SAND(CL)	**36**	**15**	**21**		

D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
4.75				**0.0**	**26.3**	**73.7**	

Te-An Wang,
Analysis & Quality Review/Date

Specimens prepared by: T.W.

Soil Analysis Lab Results

Client: RRC Power & Energy LLC
Job Name: Corby Bess
Client Job Number: GE2306031
Project X Job Number: S231002I
October 3, 2023

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-01	1-3	17.9	0.0018	2.0	0.0002	938	871	8.5	164	1.3	2.5	1.1	ND	70.5	1.5	51.5	92.9	12.3	5.2
B-04	1-3	23.2	0.0023	6.4	0.0006	2,613	2,077	8.2	144	3.3	8.8	1.2	ND	18.5	4.5	50.4	150.9	6.7	4.1
B-07	1-3	22.8	0.0023	5.8	0.0006	871	804	8.4	146	0.6	0.7	1.0	ND	25.8	3.2	28.0	86.5	7.8	88.5
B-10	1-3	36.6	0.0037	20.4	0.0020	1,340	1,273	8.0	155	3.1	1.3	0.2	ND	31.9	2.8	48.1	141.2	6.7	5.7

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

Percolation Test Data Sheet

Project:	Corby BESS			Project No:	GE2306031	Date:	9/15/2023
Test Hole No:	B-1			Tested By:	Edgar Moriel		
Latitude	38.3905098			Longitude	-121.906686		
Depth of Test Hole, D_T:		58 in.		USCS Soil Classification:		CH with Sand	
Total Hole/Piezometer Dimensions (inches)						Length	Width
Diameter (if round)=		4.75 in.	Sides =			N/A	N/A
Trail No.	Start Time (min.)	Stop Time (min.)	Δt Time Interval (min.)	D_o Initial Depth to Water (cm)	D_f Final Depth to Water (cm)	ΔD Change in Water Level (cm)	Greater than or Equal to 6"? (Y/N)
1	0	30	30	3.40	4.40	1.00	N
2	30	60	30	4.40	5.30	0.90	N
3	60	90	30	5.30	6.00	0.70	N
4	90	120	30	6.00	6.80	0.80	N
5	120	150	30	6.80	7.60	0.80	N

COMMENTS: Pre-soaked 14 hrs before testing - then filled up twice before testing
Reference point of measuring water level is at the top of the hole.

Percolation Test Data Sheet

Project:	Corby BESS			Project No:	GE2306031	Date:	9/15/2023
Test Hole No:	B-2			Tested By:	Edgar Moriel		
Latitude	38.3911034			Longitude	-121.90808		
Depth of Test Hole, D_T:		58 in.		USCS Soil Classification:		CH with Sand	
Total Hole/Piezometer Dimensions (inches)						Length	Width
Diameter (if round)=		7.5 in.	Sides =			N/A	N/A
Trail No.	Start Time (min.)	Stop Time (min.)	Δt Time Interval (min.)	D_o Initial Depth to Water (cm)	D_f Final Depth to Water (cm)	ΔD Change in Water Level (cm)	Greater than or Equal to 6"? (Y/N)
1	0	30	30	3.10	4.20	1.10	N
2	30	60	30	4.20	5.20	1.00	N
3	60	90	30	5.20	6.00	0.80	N
4	90	120	30	6.00	6.80	0.80	N
5	120	150	30	6.80	7.50	0.70	N

COMMENTS: Pre-soaked 18 hrs before testing - then filled up twice before testing
Reference point of measuring water level is at the top of the hole.

Percolation Test Data Sheet

Project:	Corby BESS			Project No:	GE2306031	Date:	9/15/2023
Test Hole No:	B-5			Tested By:	Edgar Moriel		
Latitude	38.3926204			Longitude	-121.906663		
Depth of Test Hole, D_T:		58 in.		USCS Soil Classification:		CH with Sand	
Total Hole/Piezometer Dimensions (inches)						Length	Width
Diameter (if round)=		7.5 in.	Sides =			N/A	N/A
Trail No.	Start Time (min.)	Stop Time (min.)	Δt Time Interval (min.)	D_o Initial Depth to Water (cm)	D_f Final Depth to Water (cm)	ΔD Change in Water Level (cm)	Greater than or Equal to 6"? (Y/N)
1	0	30	30	1.00	1.50	0.50	N
2	30	60	30	1.50	1.80	0.30	N
3	60	90	30	1.80	2.10	0.30	N
4	90	120	30	2.10	2.50	0.40	N
5	120	150	30	2.50	2.80	0.30	N

COMMENTS: Pre-soaked 18 hrs before testing - then filled up twice before testing
Reference point of measuring water level is at the top of the hole.

Percolation Test Data Sheet

Project:	Corby BESS			Project No:	GE2306031	Date:	9/15/2023
Test Hole No:	B-9			Tested By:	Edgar Moriel		
Latitude	38.3945365			Longitude	-121.906931		
Depth of Test Hole, D_T:		58 in.		USCS Soil Classification:		CH with Sand	
Total Hole/Piezometer Dimensions (inches)						Length	Width
Diameter (if round)=		4.75 in.	Sides =			N/A	N/A
Trail No.	Start Time (min.)	Stop Time (min.)	Δt Time Interval (min.)	D_o Initial Depth to Water (cm)	D_f Final Depth to Water (cm)	ΔD Change in Water Level (cm)	Greater than or Equal to 6"? (Y/N)
1	0	30	30	8.80	10.80	2.00	N
2	30	60	30	10.80	12.50	1.70	N
3	60	90	30	12.50	13.40	0.90	N
4	90	120	30	13.40	14.30	0.90	N
5	120	150	30	14.30	15.10	0.80	N

COMMENTS: Pre-soaked 14 hrs before testing - then filled up twice before testing
Reference point of measuring water level is at the top of the hole.

Percolation Test Data Sheet

Project:	Corby BESS			Project No:	GE2306031	Date:	9/15/2023
Test Hole No:	B-10			Tested By:	Edgar Moriel		
Latitude	38.394552			Longitude	-121.908452		
Depth of Test Hole, D_T :		58 in.		USCS Soil Classification:		CH with Sand	
Total Hole/Piezometer Dimensions (inches)						Length	Width
Diameter (if round)=		7.5 in.	Sides =			N/A	N/A
Trail No.	Start Time (min.)	Stop Time (min.)	Δt Time Interval (min.)	D_o Initial Depth to Water (cm)	D_f Final Depth to Water (cm)	ΔD Change in Water Level (cm)	Greater than or Equal to 6"? (Y/N)
1	0	30	30	5.10	6.80	1.70	N
2	30	60	30	6.80	8.10	1.30	N
3	60	90	30	8.10	9.30	1.20	N
4	90	120	30	9.30	10.40	1.10	N
5	120	150	30	10.40	11.50	1.10	N

COMMENTS: Pre-soaked 21 hrs before testing - then filled up twice before testing
Reference point of measuring water level is at the top of the hole.

www.RRCcompanies.com

810 Hesters Crossing Rd, Suite 120
Round Rock, TX 78661
512.992.2087

APPENDIX C

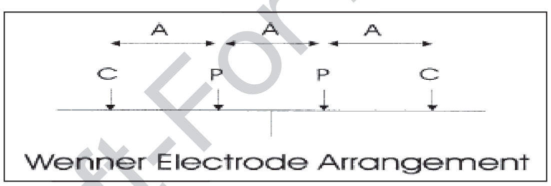
SOIL RESISTIVITY MEASURMENT DATA SHEET

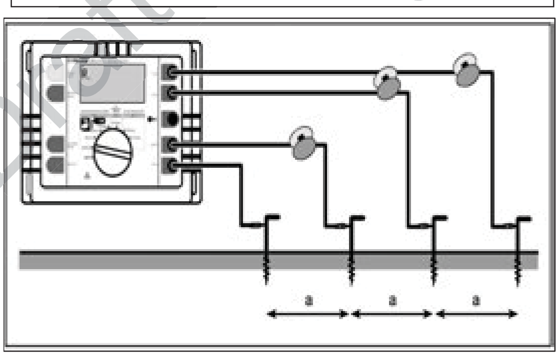
Survey ID ER-B3
DATE 9/25/2023
CLIENT NextEra Energy Resources, Inc.
PROJECT Corby BESS **Project No.** GE2306031
LOCATION: Solano County, California
LATITUDE : 38.391487
LONGITUDE : -121.906719
WEATHER: Sunny **Temp. (°F)** 62°F
TOP SOIL: Lean clay(CL), trace sand, Brown, Dry-Moist
TYPE OF TEST : Wenner 4-Pin Method
EQUIPMENT: Supersting
SERIAL NO. SS2106234
MODEL: R/8
CALIBRATION DUE DATE: 7/24/2023
TEST PERFORMED BY : RRC

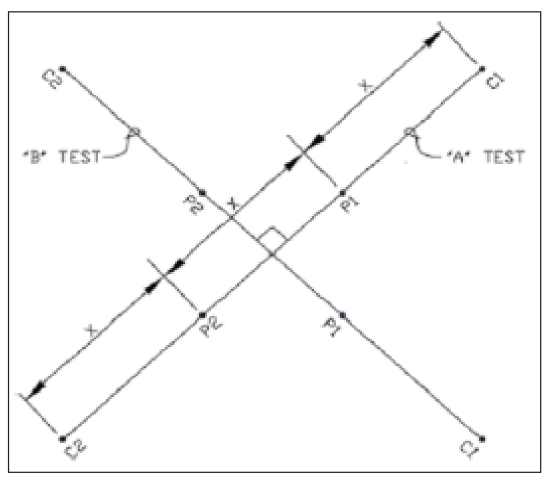
TEST SET RANGE
Meter Current: 1mA - 2000mA
Meter Resistance: 0.01Ohm - 19.99kOhom

PROB Spacing (ft)	PROBE C DEPTH (Inches)	PROBE P DEPTH (Inches)	APPARENT ELECTRICAL RESISTIVITY						
			NW-SE		NE-SW				
			Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Average Soil Resistivity (Ωm)
1	4	2	22.550	43.16	19.48	37.29			40.23
2	4	2	5.913	22.64	5.950	22.78			22.71
3	4	2	2.117	12.16	2.147	12.33			12.24
6	12	6	0.707	8.12	0.755	8.67			8.40
10	12	6	0.437	8.36	0.436	8.35			8.36
20	12	6	0.239	9.15	0.246	9.42			9.28
30	12+	6	0.164	9.42	0.170	9.76			9.59
60	12+	6	0.097	11.14	0.094	10.80			10.97
100	12+	6	0.068	13.02	0.062	11.87			12.44
200	12+	6	0.032	12.25	0.034	13.02			12.63
300	12+	6	0.023	13.21	0.022	12.63			12.92
400	12+	6	0.016	12.25	0.016	12.25			12.25

Notes:
1. Overhead Power line running N-S ~ 300' NE of the center of the profile line.
2. Overhead Power line running E-W ~ 1400' N of the center of the profile line.
3. Overhead Power line running E-W ~ 1200' S of the center of the profile line.





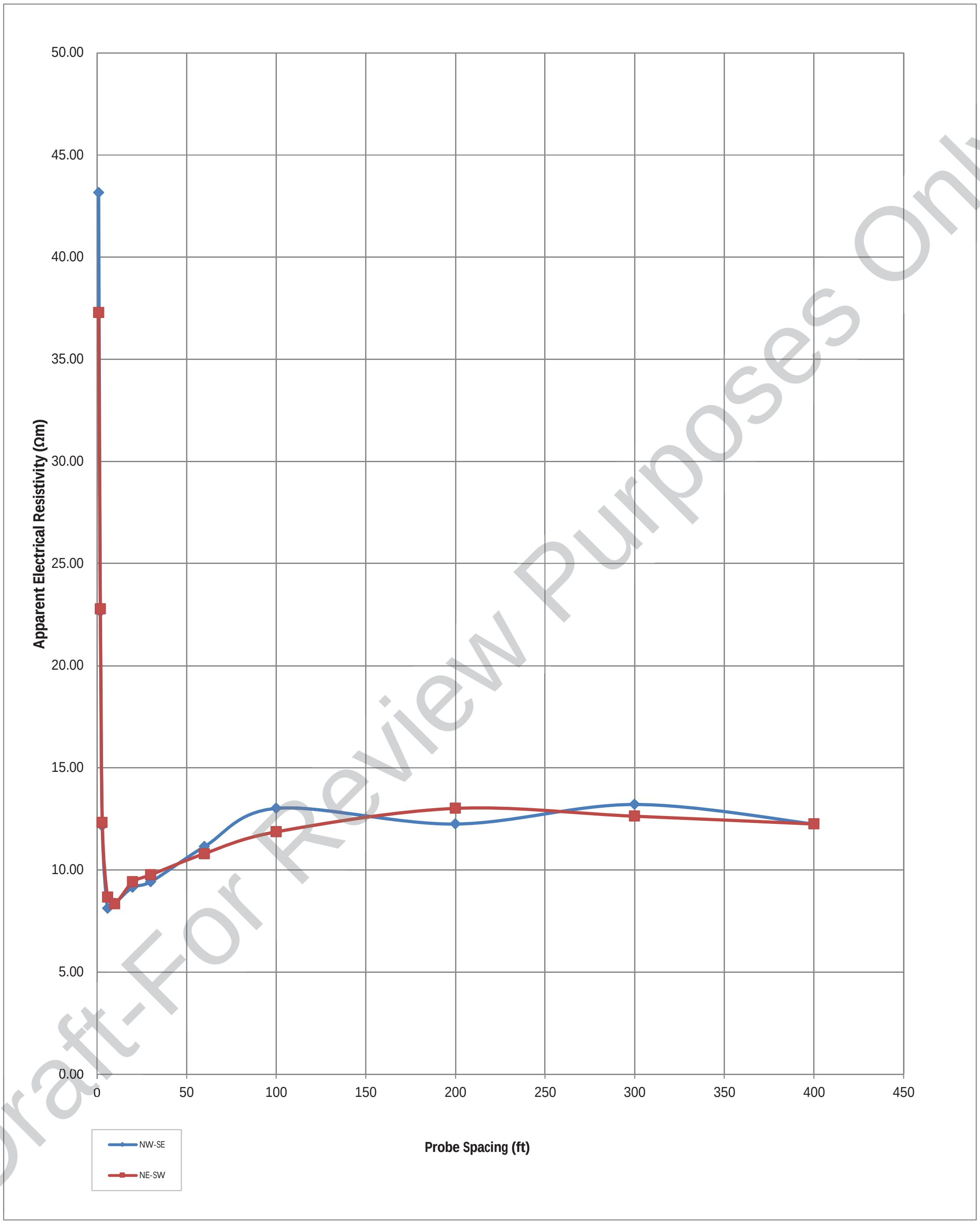


General Sketch of the test set up.

Total Array length is 3 times the probe spacing. The Apparent resistivity is calculated using the following equation: r=2*p*R*spacing*0.3048, where last item converts feet to meters. Wenner Array surveys were performed generally in accordance with IEEE std 81-2012 "IEEE Guide for Measuring Earth Resistivity, Ground Impedance and Earth Surface Potentials of a Grounding System." and ASTM G-57.

Corby BESS

Electrical Resistivity Survey at ER-B3



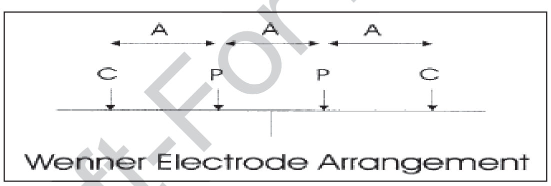
SOIL RESISTIVITY MEASURMENT DATA SHEET

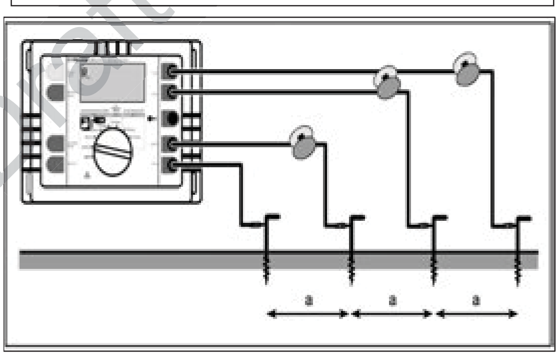
Survey ID ER-B5
DATE 9/26/2023
CLIENT NextEra Energy Resources, Inc.
PROJECT Corby BESS — **Project No.** GE2306031
LOCATION: Solano County, California
LATITUDE : 38.392661
LONGITUDE : -121.90667
WEATHER: Sunny — **Temp. (°F)** 75°F
TOP SOIL: Lean clay(CL), trace sand, Brown, Dry-Moist
TYPE OF TEST : Wenner 4-Pin Method
EQUIPMENT: Supersting
SERIAL NO. SS2106234
MODEL: R/8
CALIBRATION DUE DATE: 7/24/2023
TEST PERFORMED BY : RRC

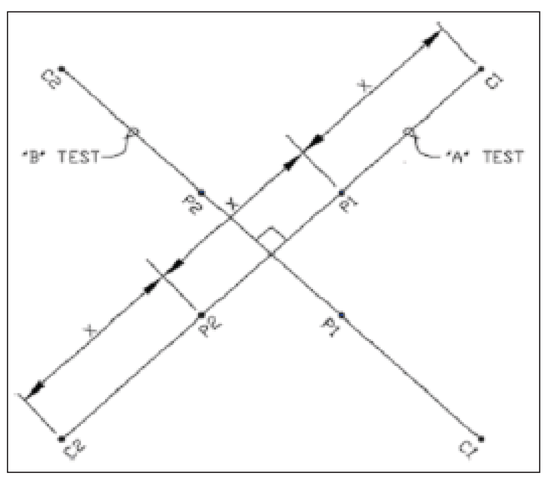
TEST SET RANGE
Meter Current: 1mA - 2000mA
Meter Resistance: 0.01Ohm - 19.99kOhom

PROB Spacing (ft)	PROBE C DEPTH (Inches)	PROBE P DEPTH (Inches)	APPARENT ELECTRICAL RESISTIVITY						
			NW-SE		NE-SW				
			Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Average Soil Resistivity (Ωm)
1	4	2	19.11	36.58	21.04	40.27			38.43
2	4	2	8.165	31.26	8.844	33.86			32.56
3	4	2	4.416	25.36	4.427	25.42			25.39
6	12	6	1.028	11.81	1.042	11.97			11.89
10	12	6	0.507	9.70	0.493	9.44			9.57
20	12	6	0.249	9.53	0.254	9.72			9.63
30	12+	6	0.175	10.05	0.176	10.11			10.08
60	12+	6	0.094	10.80	0.095	10.91			10.85
100	12+	6	0.066	12.63	0.065	12.44			12.54
200	12+	6	0.032	12.25	0.033	12.63			12.44
300	12+	6	0.021	12.06	0.021	12.06			12.06
400	12+	6	0.016	12.25	0.016	12.25			12.25

Notes:
1. Overhead Power line running N-S ~ 250' NE of the center of the profile line.
2. Overhead Power line running E-W ~ 970' N of the center of the profile line.
3. Overhead Power line running E-W ~ 1700' S of the center of the profile line.

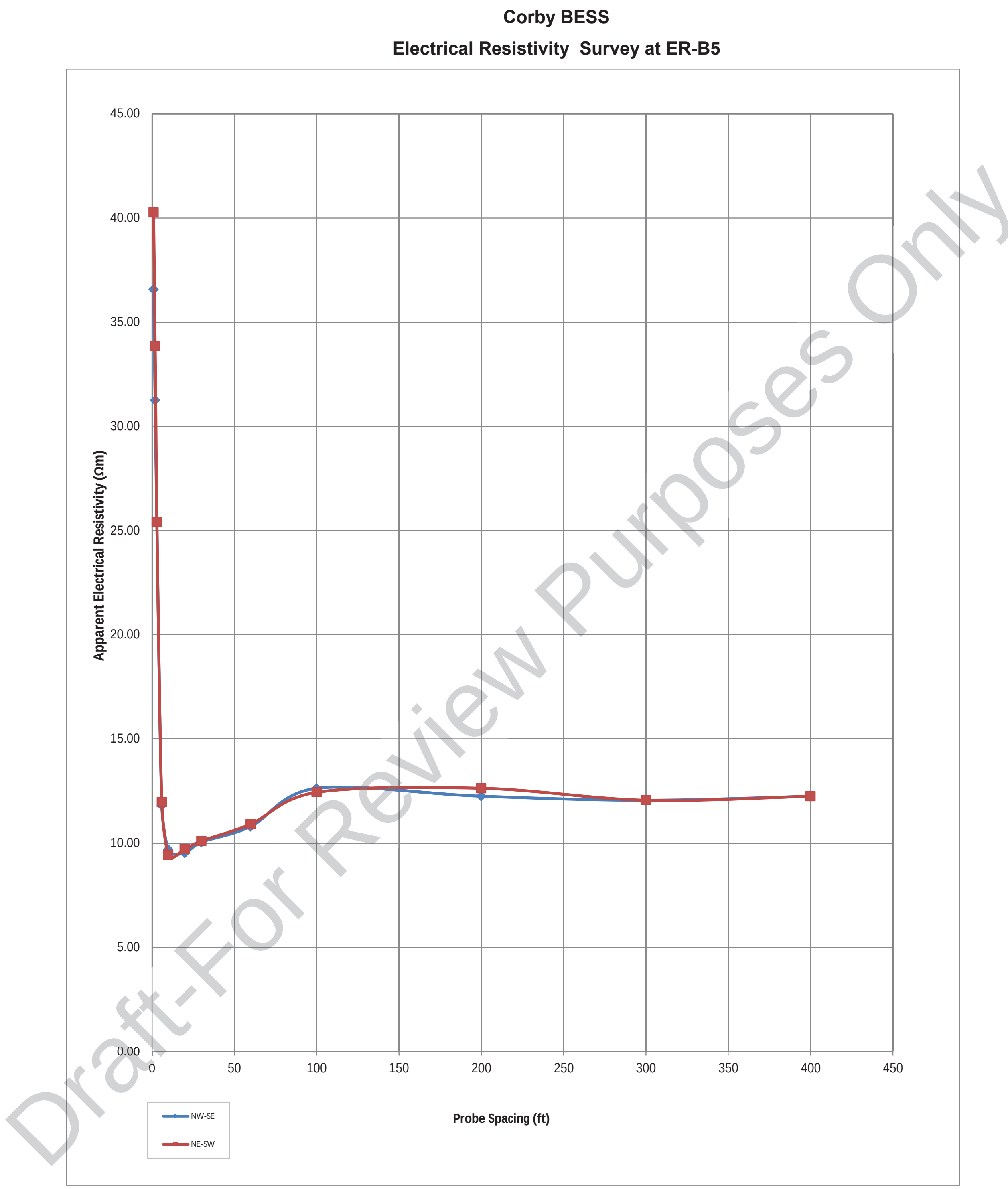






General Sketch of the test set up.

Total Array length is 3 times the probe spacing. The Apparent resistivity is calculated using the following equation: r=2*p*R*spacing*0.3048, where last item converts feet to meters. Wenner Array surveys were performed generally in accordance with IEEE std 81-2012 "IEEE Guide for Measuring Earth Resistivity, Ground Impedance and Earth Surface Potentials of a Grounding System." and ASTM G-57.


Corby BESS
Electrical Resistivity Survey at ER-B5
Apparent Electrical Resistivity (Ωm)
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0
50
100
150
200
250
300
350
400
450
Probe Spacing (ft)
NW-SE
NE-SW
Draft-For Review Purposes Only

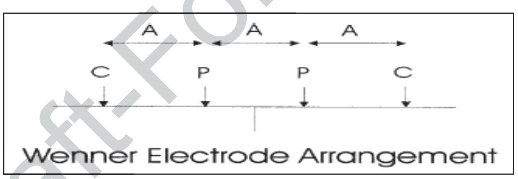
SOIL RESISTIVITY MEASURMENT DATA SHEET

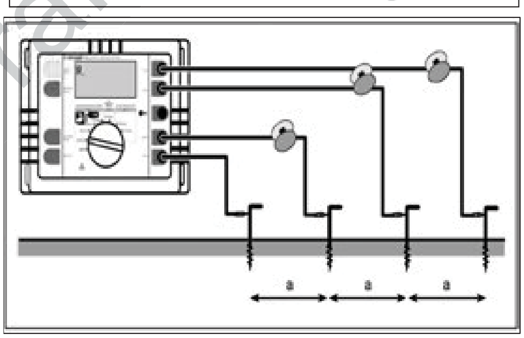
Survey ID ER-Substation
DATE 9/27/2023
CLIENT NextEra Energy Resources, Inc.
PROJECT Corby BESS **Project No.** GE2306031
LOCATION: Solano County, California
LATITUDE : 38.392758
LONGITUDE : -121.907455
WEATHER: Sunny **Temp. (°F)** 57°F
TOP SOIL: Lean clay(CL), trace sand, Brown, Dry-Moist
TYPE OF TEST : Wenner 4-Pin Method
EQUIPMENT: Superstring
SERIAL NO. SS2106234
MODEL: R/8
CALIBRATION DUE DATE: 7/24/2023
TEST PERFORMED BY : RRC

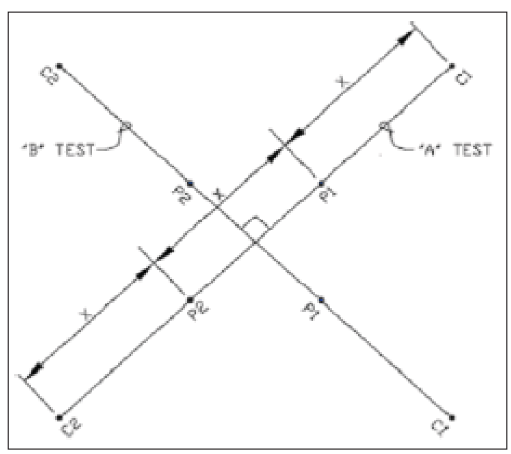
TEST SET RANGE
Meter Current: 1mA - 2000mA
Meter Resistance: 0.01Ohm - 19.99kOhom

PROB Spacing (ft)	PROBE C DEPTH (Inches)	PROBE P DEPTH (Inches)	APPARENT ELECTRICAL RESISTIVITY						
			NW-SE		NE-SW				
			Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Average Soil Resistivity (Ωm)
0.5	4	2	16.280	15.58	17.790	17.03			16.30
1	4	2	10.860	20.79	11.220	21.48			21.13
1.5	4	2	6.080	17.46	5.967	17.13			17.29
2	4	2	3.669	14.05	3.558	13.62			13.83
3	4	2	2.211	12.70	2.347	13.48			13.09
5	4	2	1.039	9.94	0.958	9.17			9.56
7	12	6	0.654	8.76	0.628	8.41			8.59
10	12	6	0.436	8.35	0.444	8.50			8.42
15	12	6	0.326	9.36	0.316	9.07			9.22
20	12+	6	0.255	9.76	0.242	9.26			9.51
30	12+	6	0.175	10.05	0.177	10.16			10.11
45	12+	6	0.123	10.59	0.129	11.11			10.85
70	12+	6	0.087	11.66	0.086	11.52			11.59
100	12+	6+	0.063	12.06	0.060	11.48			11.77
150	12+	6+	0.044	12.63	0.043	12.35			12.49
250	12+	6+	0.026	12.44	0.027	12.92			12.68
350	12+	6+	0.018	12.06	0.018	12.06			12.06
400	12+	6+	0.016	12.25	0.016	12.25			12.25
600	12+	6+	0.010	11.48	0.010	11.48			11.48

Notes:
1. Overhead Power line running N-S ~ 40' NE of the center of the profile line.
2. Overhead Power line running E-W ~ 940' N of the center of the profile line.
3. Overhead Power line running E-W ~ 1750' S of the center of the profile line.





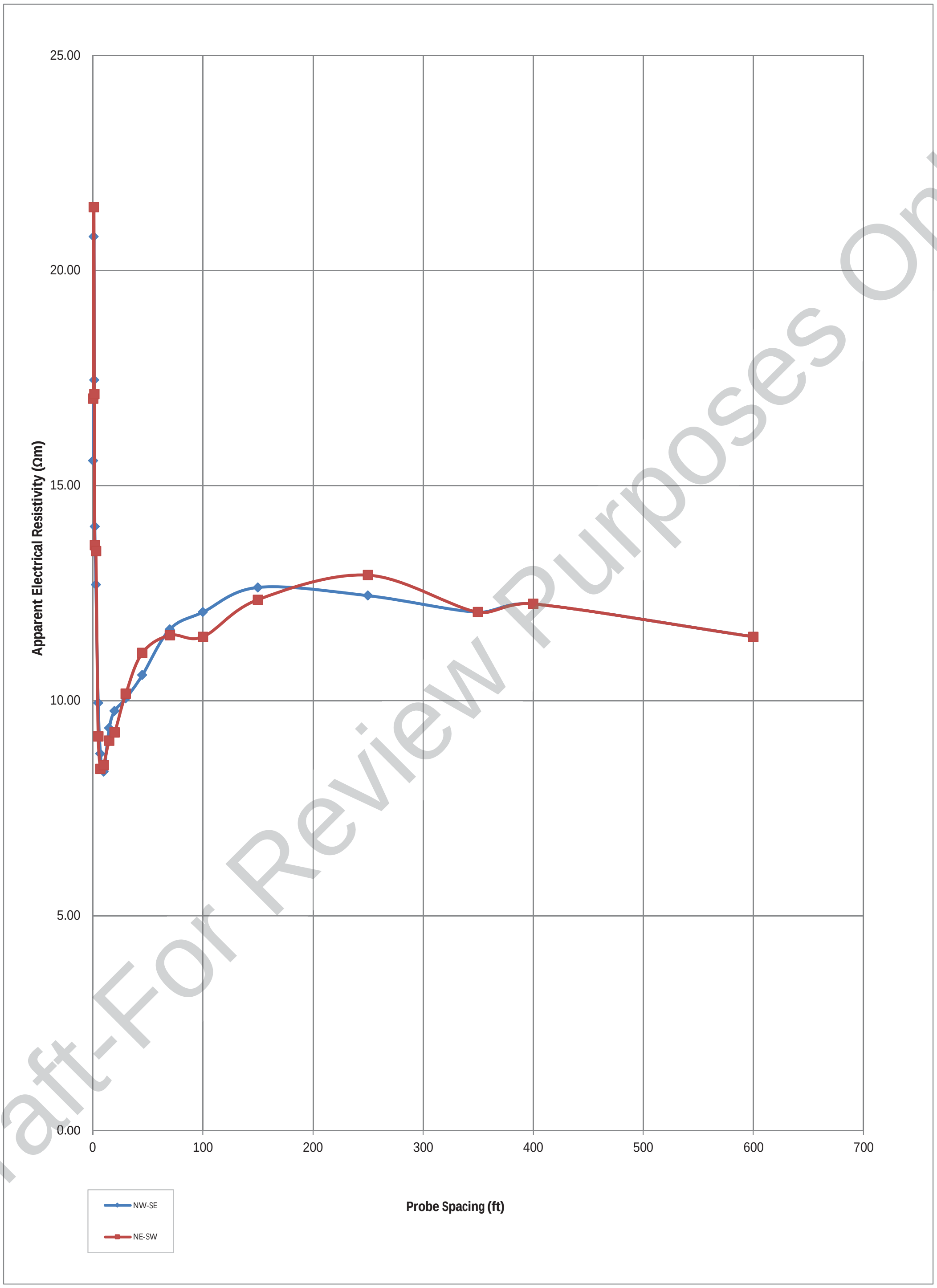


General Sketch of the test set up.

Total Array length is 3 times the probe spacing. The Apparent resistivity is calculated using the following equation: r=2*p*R*spacing*0.3048, where last item converts feet to meters. Wenner Array surveys were performed generally in accordance with IEEE std 81-2012 "IEEE Guide for Measuring Earth Resistivity, Ground Impedance and Earth Surface Potentials of a Grounding System." and ASTM G-57.

Corby BESS

Electrical Resistivity Survey at ER-Substation



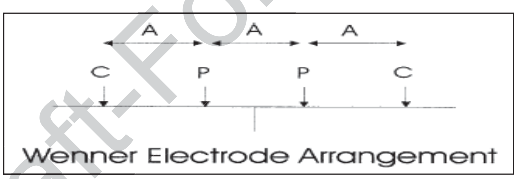
SOIL RESISTIVITY MEASURMENT DATA SHEET

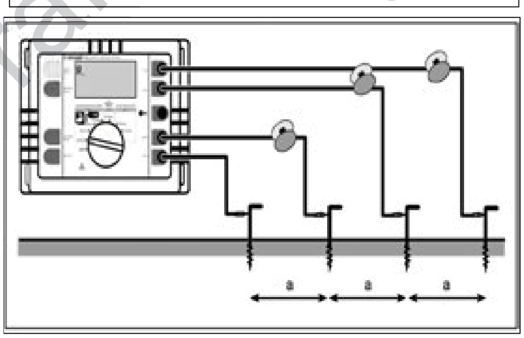
Survey ID ER-Sub B10
DATE 9/26/2023
CLIENT NextEra Energy Resources, Inc.
PROJECT Corby BESS **Project No.** GE2306031
LOCATION: Solano County, California
LATITUDE : 38.393586
LONGITUDE : -121.907597
WEATHER: Sunny **Temp. (°F)** 80°F
TOP SOIL: Lean clay(CL), trace sand, Brown, Dry-Moist
TYPE OF TEST : Wenner 4-Pin Method
EQUIPMENT: Superstring
SERIAL NO. SS2106234
MODEL: R/8
CALIBRATION DUE DATE: 7/24/2023
TEST PERFORMED BY : RRC

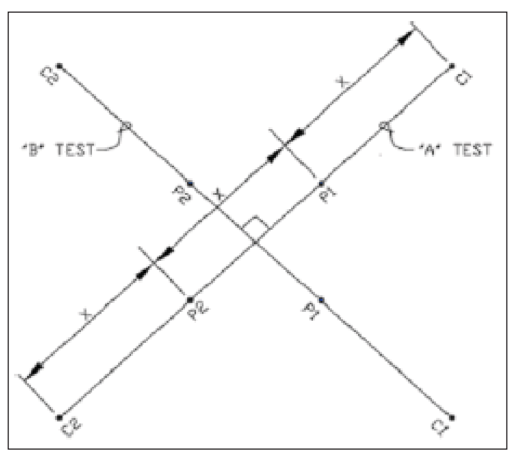
TEST SET RANGE
Meter Current: 1mA - 2000mA
Meter Resistance: 0.01Ohm - 19.99kOhom

PROB Spacing (ft)	PROBE C DEPTH (Inches)	PROBE P DEPTH (Inches)	APPARENT ELECTRICAL RESISTIVITY						
			NW-SE		NE-SW				
			Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Meter reading (Ω)	Soil Resistivity (Ωm)	Average Soil Resistivity (Ωm)
0.5	4	2	41.450	39.67	45.170	43.23			41.45
1	4	2	30.410	58.21	34.460	65.96			62.09
1.5	4	2	23.430	67.27	25.030	71.87			69.57
2	4	2	17.200	65.85	19.130	73.24			69.54
3	4	2	10.840	62.25	11.440	65.69			63.97
5	4	2	4.747	45.43	4.521	43.27			44.35
7	12	6	2.730	36.58	2.796	37.46			37.02
10	12	6	1.673	32.02	1.634	31.28			31.65
15	12	6	0.813	23.34	0.826	23.72			23.53
20	12+	6	0.454	17.38	0.454	17.38			17.38
30	12+	6	0.229	13.15	0.239	13.72			13.44
45	12+	6	0.132	11.37	0.142	12.23			11.80
70	12+	6	0.082	10.99	0.085	11.39			11.19
100	12+	6+	0.063	12.06	0.062	11.87			11.96
150	12+	6+	0.045	12.92	0.043	12.35			12.63
250	12+	6+	0.027	12.92	0.028	13.40			13.16
350	12+	6+	0.019	12.73	0.019	12.73			12.73
400	12+	6+	0.016	12.25	0.016	12.25			12.25
600	12+	6+	0.010	11.48	0.010	11.48			11.48

Notes:
1. Overhead Power line running N-S ~ 130' NE of the center of the profile line.
2. Overhead Power line running E-W ~ 520' N of the center of the profile line.
3. Overhead Power line running E-W ~ 2060' S of the center of the profile line.



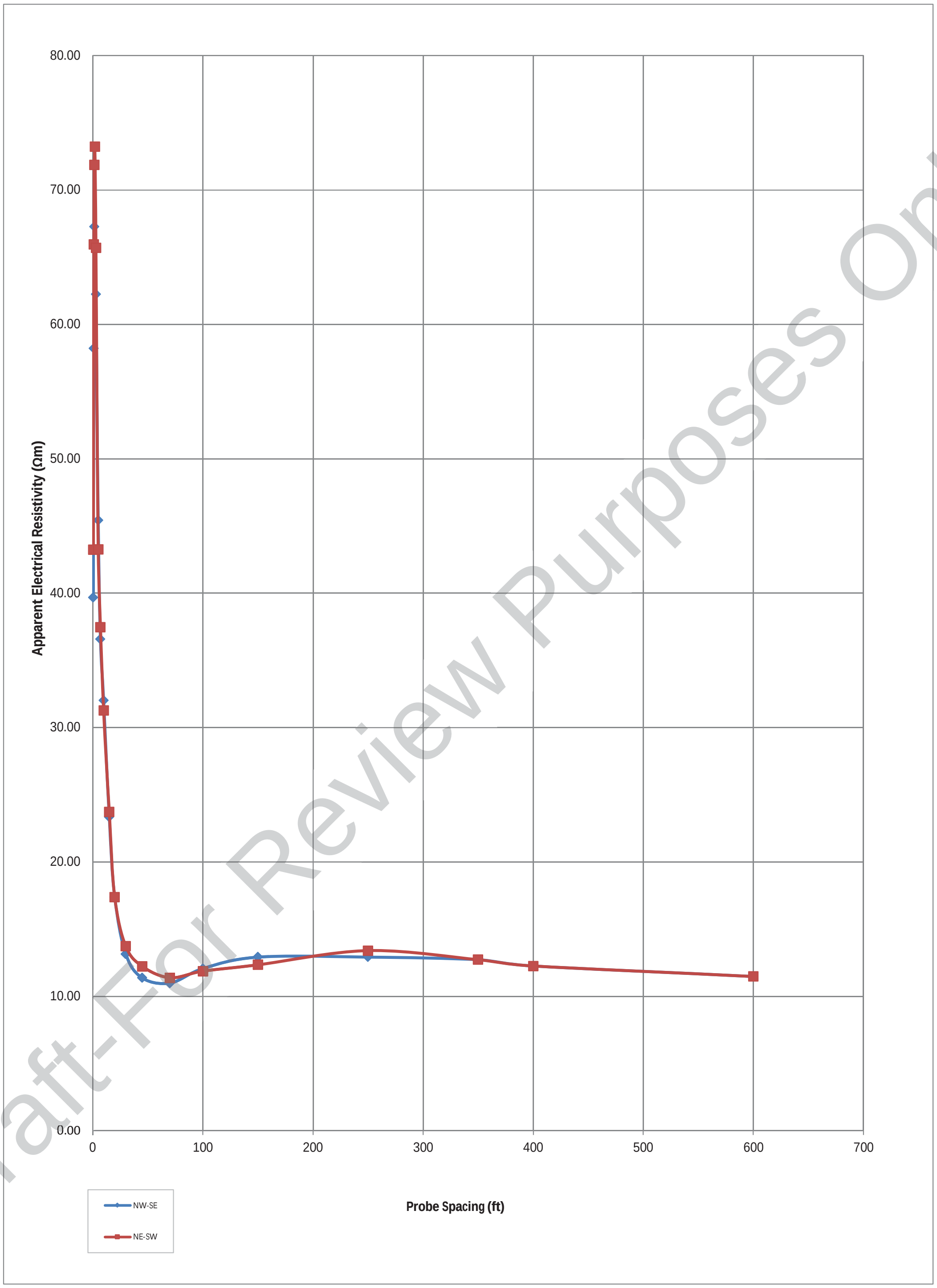




General Sketch of the test set up.

Total Array length is 3 times the probe spacing. The Apparent resistivity is calculated using the following equation: r=2*p*R*spacing*0.3048, where last item converts feet to meters. Wenner Array surveys were performed generally in accordance with IEEE std 81-2012 "IEEE Guide for Measuring Earth Resistivity, Ground Impedance and Earth Surface Potentials of a Grounding System." and ASTM G-57.

Corby BESS
Electrical Resistivity Survey at ER-Sub B10



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Soil Thermal Resistivity Sample & Testing Summary
Corby BESS Project (PN: GE2306031)

Remolded Samples							
Boring	*Depth*	*Percent Passing #200 Sieve*	*Maximum Dry Density (ASTM D698)*	*Optimum Water Content (ASTM D698)*	*LL*	*PL*	*PI*
	ft	*%*	*pcf*	*%*			
B-2	1-4	67.0	110.8	14.5	40	16	24
B-4	1-4	60.7	113.4	13.1	32	14	18
B-6	1-4	67.4	108.3	14.3	38	13	25
B-10	1-4	73.7	108.0	16.2	36	15	21

Remolded Samples		
Boring	*Depth*	*Soil Type (USCS)*
B-2	1-4	Sandy Lean Clay (CL)
B-4	1-4	Sandy Lean Clay (CL)
B-6	1-4	Sandy Lean Clay (CL)
B-10	1-4	Lean Clay with Sand (CL)

Remolded Samples						
Boring	*Remold Water Content*	*Percent Compaction*	*Target Remold Dry Density*	*Actual Remold Dry Density*	*Thermal Resistivity at Wet (IEEE 422-2017)*	*Thermal Resistivity at Dry (IEEE 422-2017)*
	%	*%*	*pcf*	*pcf*	*°C-cm/W*	*°C-cm/W*
B-2	16.8	90	99.7	100.1	71	157
B-4	16.6		102.1	102.4	69	161
B-6	16.5		97.5	97.6	77	188
B-10	19.3		97.2	97.5	77	170

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Soil Thermal Resistivity Sample & Testing Summary
Corby BESS Project (PN: GE2306031)

Intact Samples						
Boring	*Depth*	*Percent Passing #200 Sieve*	*LL*	*PL*	*PI*	*Soil Type (USCS)*
	ft	*%*				
B-2	1	55.6	47	15	32	Sandy Lean Clay (CL)
B-4	1	64.5	47	16	31	Sandy Lean Clay (CL)
B-6	1	58.3	44	14	30	Sandy Lean Clay (CL)
B-10	1	87.1	55	17	38	Fat Clay (CH)

Intact Samples						
Boring	*Depth*	*In-situ Water Content*	*In-situ Dry Unit Weight*	*Thermal Resistivity at Wet (IEEE 422-2017)*	*Thermal Resistivity at Dry (IEEE 422-2017)*	*Sample Type*
	ft	*%*	*pcf*	*°C-cm/W*	*°C-cm/W*	
B-2	3	20.7	107.7	63	147	MC Tube
B-4	3	20.8	106.4	65	150	MC Tube
B-6	3	17.2	110.9	53	156	Shelby Tube
B-10	3	23.1	103.4	65	131	Shelby Tube

HuaMiao Cao, P.E., 10/24/23

Analysis & Quality Review/Date

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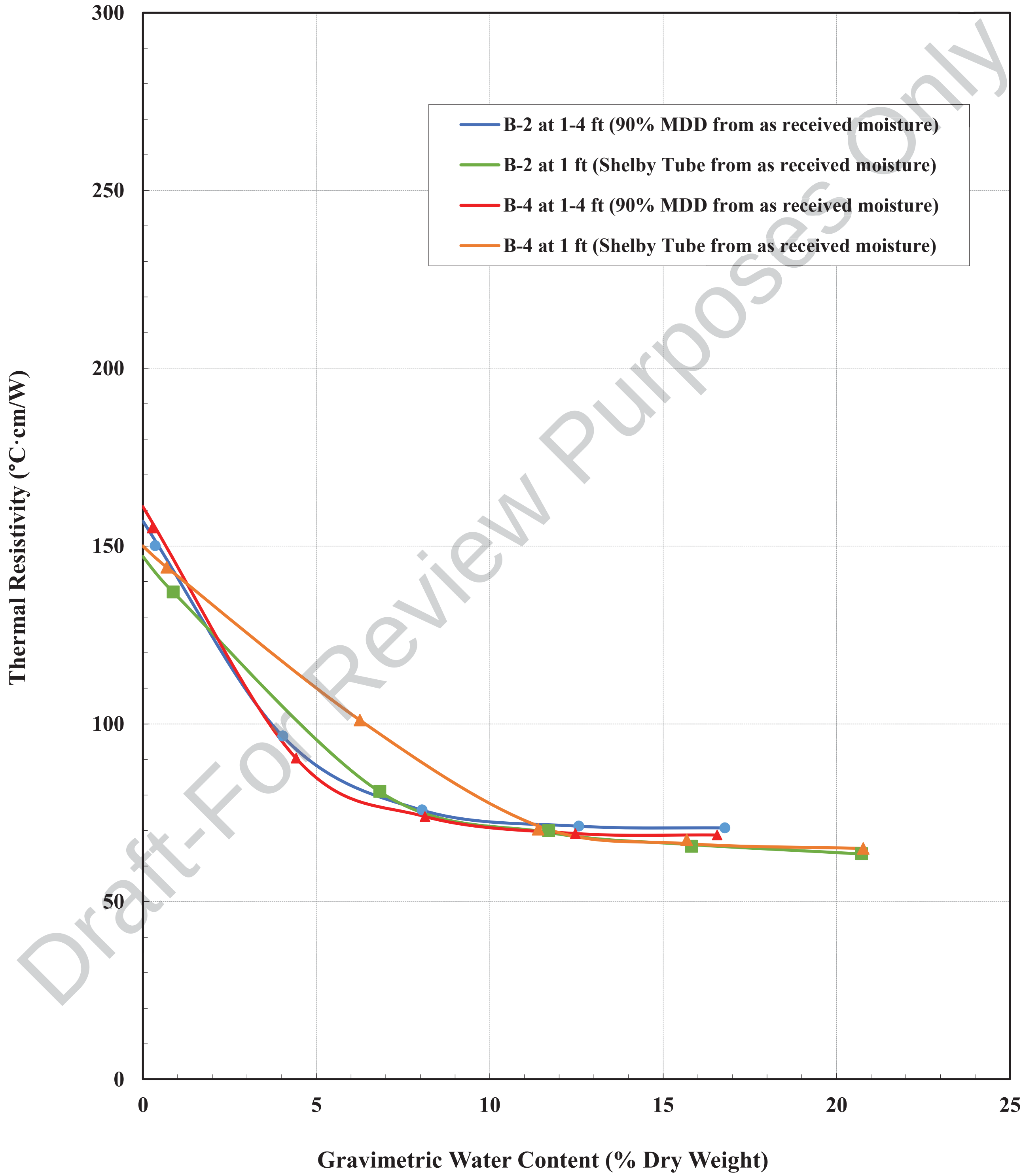
Soil Thermal Resistivity Testing Dryout Curves
Corby BESS Project (PN: GE2306031)



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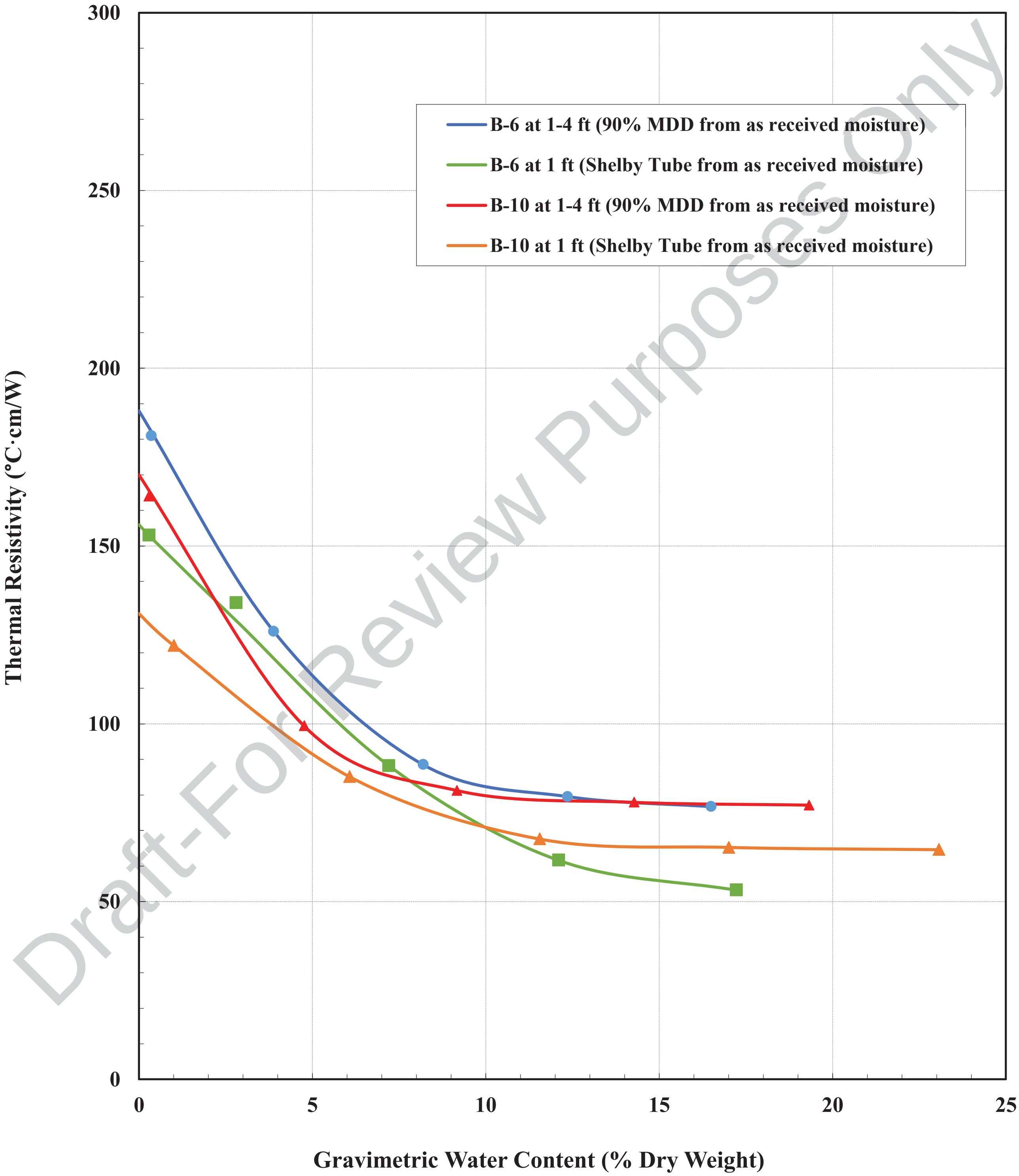
Soil Thermal Resistivity Testing Dryout Curves
Corby BESS Project (PN: GE2306031)



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APPENDIX D

Table D1.1 Soil Parameters for Driven Pile Capacity Analysis – BESS									
Soil Layer	Depth Interval (feet)	USGS Soil & Rock Classification	Effective Unit Weight (pcf)	Allowable Unit Skin Friction in Uplift (FOS = 1.5) (psf)		Allowable Unit Skin Friction in Compression (FOS = 1.5) (psf)		Allowable Bearing Pressure** (FOS = 2.0) (psf)	
				General Zone	Weak Zone (near B-3)	General Zone	Weak Zone (near B-3)	General Zone	Weak Zone (near B-3)
1*	0 to 3	SOFT CLAY / FAT CLAY	105	--	--	--	--	--	--
2*	3 to 7	LEAN CLAY	115	500	350	600	380	8,000	1,500
3	7 to 10	LEAN CLAY	115	550	350	650	380	10,000	2,500
4***	10 to 20	LEAN CLAY	120	550	350	650	380	14,000	2,500

Note: *Upper 3 feet of skin friction should be neglected due to seasonal moisture change and soil disturbance.
Based upon results of swell tests and load testing results, uplift forces on steel piles due to expansive clay can be estimated using a skin friction value of 400 psf acting on the "Box" area of the pile for the pile length in fat clays within the active zone which is about 3 feet below existing ground surface.
****Allowable Pile End Bearing Pressure may be applied using a maximum of 50% of H-pile box-area, for calculating the axial compressive pile capacity.**
*****Parameters provided in layer 4 are not obtained from pile load test. Recommend verifying with additional pile load testing if pile embedment depths are deeper than 10 feet below existing ground surface.**

Table D1.2 L-PILE Computer Program Parameters for Lateral Load Analysis – BESS													
Soil Layer	Depth Interval (feet)	LPILE Soil Type	K (pci)		γ (pcf)	C** (psf)	ϕ** (deg)	ε_{50} (in/in)	E_{rm} (psi)	UCS (psi)	RQD (%)	K_{rm}
			Static	Cyclic								
1*	0 to 1	Soft Clay (Matlock)	--	--	105	200	--	Program Default	--	--	--	--
2	1 to 3	Stiff Clay w/o Free Water (Reese)	--	--	105	1,500	--	Program Default	--	--	--	--
3	3 to 7	Stiff Clay w/o Free Water (Reese)	--	--	115	2,000	--	Program Default	--	--	--	--
4	7 to 10	Stiff Clay w/o Free Water (Reese)	--	--	120	3,000	--	Program Default	--	--	--	--
5***	10 to 20	Stiff Clay w/o Free Water (Reese)	--	--	120	3,000	--	Program Default	--	--	--	--

Notes: K is the modulus of subgrade reaction; γ is the effective unit weight; C is the cohesion of soil; ϕ is the friction angle of soil; ε_{50} is the soil strain parameter; E_{rm} is the rock mass modulus of the rock; UCS is average Unconfined Compressive Strength of rock; RQD is average Rock Quality Designation; K_{rm} is the rock strain parameter.

Notes: *For upper 12 inches or scour depth, whichever is deeper, design parameters have been reduced due to seasonal moisture change and soil disturbance.
**** The Undrained Shear Strengths and Friction Angles used in this table were correlated from geotechnical exploration data, not pile testing results, and may not be presentative of the actual undrained shear strength or friction angle of the subsurface materials.**
***** Parameters provided in layer 5 are not obtained from pile load test. Recommend verifying with additional pile load testing if pile embedment depths are deeper than 10 feet below existing ground surface.**

Table D2.1 – LPILE Computer Program Parameters for Lateral Load Analysis for Substation

Soil Layer	Depth (feet)	LPILE Soil Type	K (pci)		γ'	C	ϕ	ε_{50}	E_{rm}	UCS	RQD	K_{rm}
			Static	Cyclic	(pcf)	(psf)	(degree)		(psi)	(psi)	(%)	
1*	0 to 3	Soft Clay (1)	--	--	103	--	--	--	--	--	--	--
2	3 to 6	Stiff Clay w/o Free Water	--	--	115	1,850	--	0.007	--	--	--	--
3	6 to 14	Stiff Clay w/o Free Water	--	--	53	1,450	--	0.007	--	--	--	--
4	14 to 29	Stiff Clay w/o Free Water	--	--	53	1,590	--	0.007	--	--	--	--
5	29 to 34	Stiff Clay w/o Free Water	--	--	53	2,650	--	0.005	--	--	--	--
6	34 to 39	Stiff Clay w/o Free Water	--	--	53	3,720	--	0.005	--	--	--	--

Notes: *Upper 3 feet of soil may be neglected due to seasonal moisture change.

Table D2.2 – Direct Embedment/Drilled Pier Foundation Design Parameters for Substation

Soil Layer	Depth (feet)	USCS Soil & Rock Classification	γ	ϕ	C	C'_{Rock}	K_p	SPT N-Value (blows/ft)	Deformation Modulus (ksi)	Allowable Unit Skin Friction (FS=2.5)[(1)] (psf)	Allowable Bearing Pressure (FS=3) (psf)
			(pcf)	(degree)	(psf)	(psf)					
1*	0 to 3	Soft Clay	103	--	--	--	--	--	--	--	--
2	3 to 6	CL	115	--	1,850	--	2.04	14	1.5	400	3,800
3	6 to 14	CL	53	--	1,450	--	2.04	11	1.3	320	3,800
4	14 to 29	CL	53	--	1,590	--	2.04	12	1.4	350	4,200
5	29 to 34	CL	53	--	2,650	--	2.04	20	1.9	580	7,900
6	34 to 39	CL	53	--	3,720	--	2.04	28	2.3	780	11,100

Notes:

*Upper 3 feet of soils may be neglected due to seasonal moisture change; Kp: Rankine Passive Earth Pressure Coefficient; γ': Effective Unit Weight ($\gamma' = \gamma_{Total}$-62.4 pcf); ϕ: Angle of Internal Friction.

(1) For uplift resistance, the allowable skin friction provided in table above should be reduced by 25 percent.

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APPENDIX E

Table E1: Corby BESS - Test Pile Driving/Installing Time

Location/ Boring ID	Latitude	Longitude	Test Pile ID	Pile Section	Embedment Depth (ft)*	Stick-up Height (ft)	Pile Length (ft)	Pile Installation Date	Pile Drive Time at 1, 2, 3, 4, 5, 6, 7, 8, 9, 10 ft (sec) 1'	2'	3'	4'	5'	6'	7'	8'	9'	10'	Total Time (sec)	Notes
B-3	38.39149962	-121.9067698	PLT-B-3A	W6x9	7.0	3.0	10.0	10/3/2023	2.8	1.2	3.5	8.4	14.6	20.0	25.2				75.7	
			PLT-B-3B	W6x9	10.0	3.0	13.0	10/3/2023	WOH	0.7	3.0	6.0	12.3	17.1	20.1	22.1	25.6	30.8	137.7	
B-5	38.39262045	-121.9066633	PLT-B-5A	W6x9	6.0	3.0	9.0	10/3/2023	3.2	2.2	1.5	5.2	11.1	18.1					41.2	
			PLT-B-5B	W6x9	8.0	3.0	11.0	10/3/2023	3.8	2.5	2.3	6.8	13.8	16.1	24.2	29.3			98.7	
			PLT-B-5B-retest	W6x9	8.0	3.0	11.0	10/9/2023	1.9	2.1	1.2	4.4	10.2	16.6	21.0	29.5			86.9	Re-install*
B-8	38.39383999	-121.907111	PLT-B-8A	W6x9	7.0	3.0	10.0	10/3/2023	2.6	2.5	2.6	8.0	18.5	30.7	43.7				108.5	
			PLT-B-8A-retest	W6x9	7.0	3.0	10.0	10/9/2023	3.0	1.8	3.6	9.1	15.1	20.9	27.7				81.3	Re-install*
			PLT-B-8B	W6x9	10.0	3.0	13.0	10/3/2023	3.4	1.4	2.9	7.7	15.6	28.9	38.4	36.1	37.7	41.8	213.8	
			PLT-B-8B-retest	W6x9	10.0	3.0	13.0	10/9/2023	2.3	1.6	3.6	10.3	15.8	19.7	25.6	35.3	47.0	54.9	216.3	Re-install*
B-10	38.39455202	-121.9084521	PLT-B-10A	W6x9	7.0	3.0	10.0	10/3/2023	0.7	1.6	2.3	8.4	15.1	22.8	28.1				79.0	
			PLT-B-10A-retest	W6x9	7.0	3.0	10.0	10/9/2023	3.3	3.3	3.6	8.3	14.7	19.3	31.6				84.1	Re-install*
			PLT-B-10B	W6x9	10.0	3.0	13.0	10/3/2023	2.3	0.6	0.6	6.1	13.1	23.4	31.6	34.8	42.1	43.9	198.3	
			PLT-B-10B-retest	W6x9	10.0	3.0	13.0	10/9/2023	1.8	1.8	3.4	7.8	15.3	22.0	30.7	32.8	36.0	39.9	198.3	Re-install*

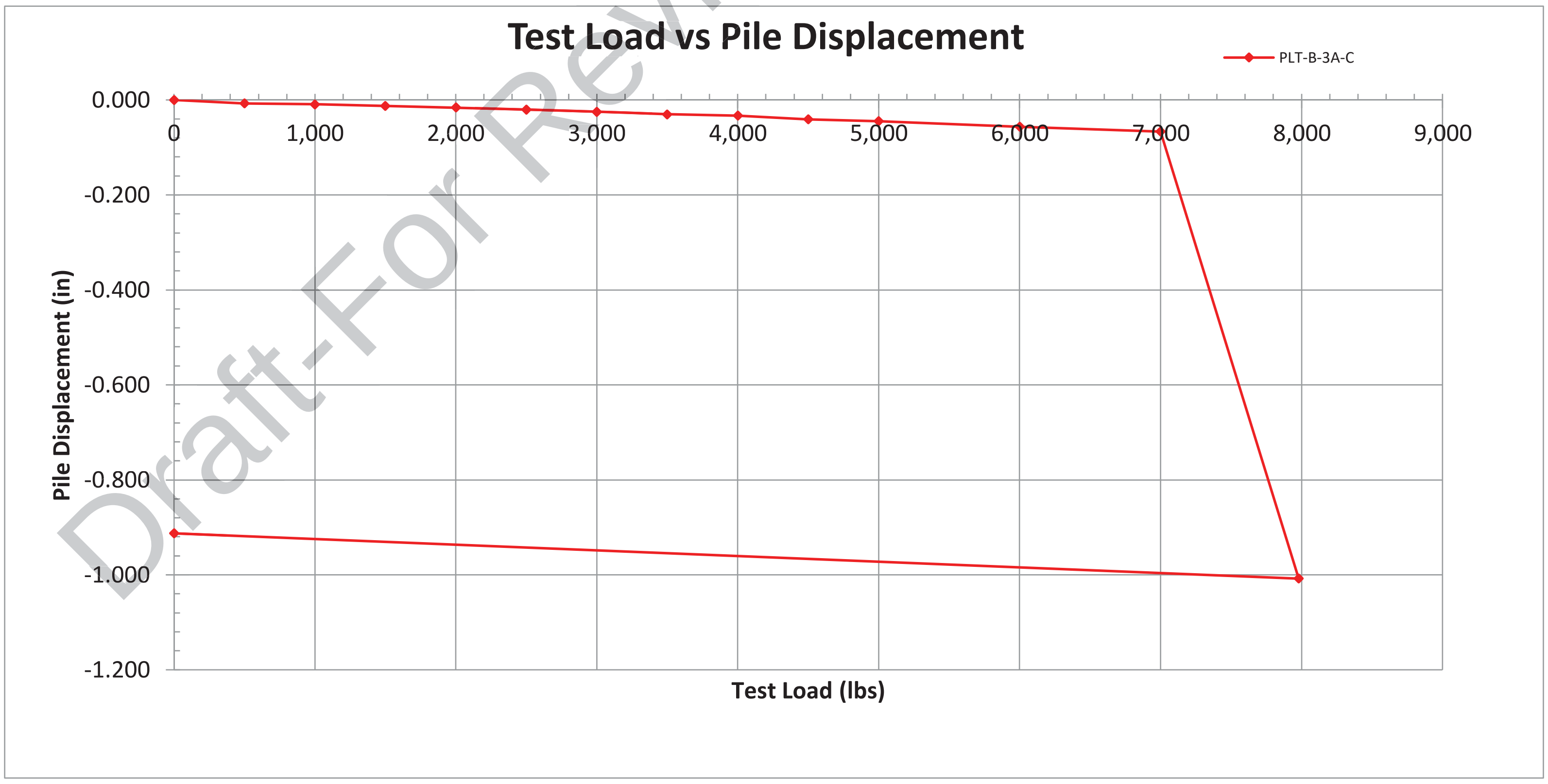
* Re-installed piles and re-tested to ensure the quality of the collected data, since the pile testing equipment initially used in those locations was not meeting high compression loads requirements.

Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-3A-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	75.7	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.008	0.007	-0.008	-0.007	-0.008	
1,000	1 min	0.012	0.007	-0.012	-0.007	-0.009	
1,500	1 min	0.019	0.008	-0.019	-0.008	-0.013	
2,000	1 min	0.020	0.012	-0.020	-0.012	-0.016	
2,500	1 min	0.022	0.019	-0.022	-0.019	-0.020	
3,000	1 min	0.023	0.027	-0.023	-0.027	-0.025	
3,500	1 min	0.028	0.033	-0.028	-0.033	-0.030	
4,000	1 min	0.031	0.036	-0.031	-0.036	-0.033	
4,500	1 min	0.038	0.044	-0.038	-0.044	-0.041	
5,000	1 min	0.042	0.048	-0.042	-0.048	-0.045	
6,000	1 min	0.057	0.056	-0.057	-0.056	-0.057	
7,000	1 min	0.062	0.072	-0.062	-0.072	-0.067	
7,980	1 min	1.005	1.011	-1.005	-1.011	-1.008	Failure
0	1 min	0.911	0.915	-0.911	-0.915	-0.913	

Notes:

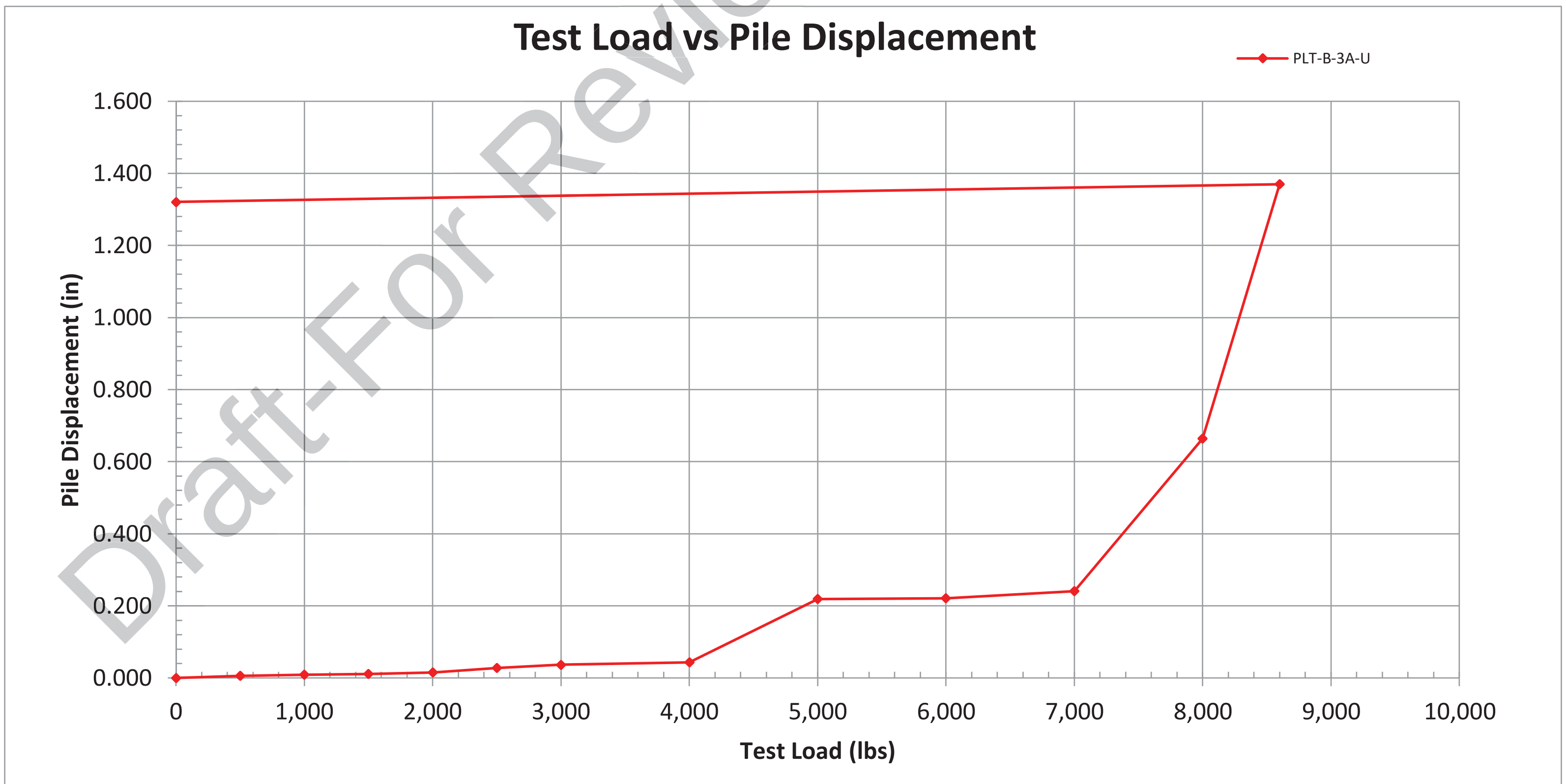


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-3A-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	75.7	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.010	-0.002	0.010	0.002	0.006	
1,000	1 min	-0.012	-0.006	0.012	0.006	0.009	
1,500	1 min	-0.014	-0.009	0.014	0.009	0.011	
2,000	1 min	-0.018	-0.013	0.018	0.013	0.016	
2,500	1 min	-0.031	-0.025	0.031	0.025	0.028	
3,000	1 min	-0.040	-0.033	0.040	0.033	0.037	
4,000	1 min	-0.046	-0.041	0.046	0.041	0.043	
5,000	1 min	-0.224	-0.215	0.224	0.215	0.219	
6,000	1 min	-0.226	-0.217	0.226	0.217	0.221	
7,000	1 min	-0.244	-0.238	0.244	0.238	0.241	
8,000	1 min	-0.668	-0.661	0.668	0.661	0.664	
8,600	1 min	-1.379	-1.361	1.379	1.361	1.370	Failure
0	1 min	-1.333	-1.309	1.333	1.309	1.321	

Notes:

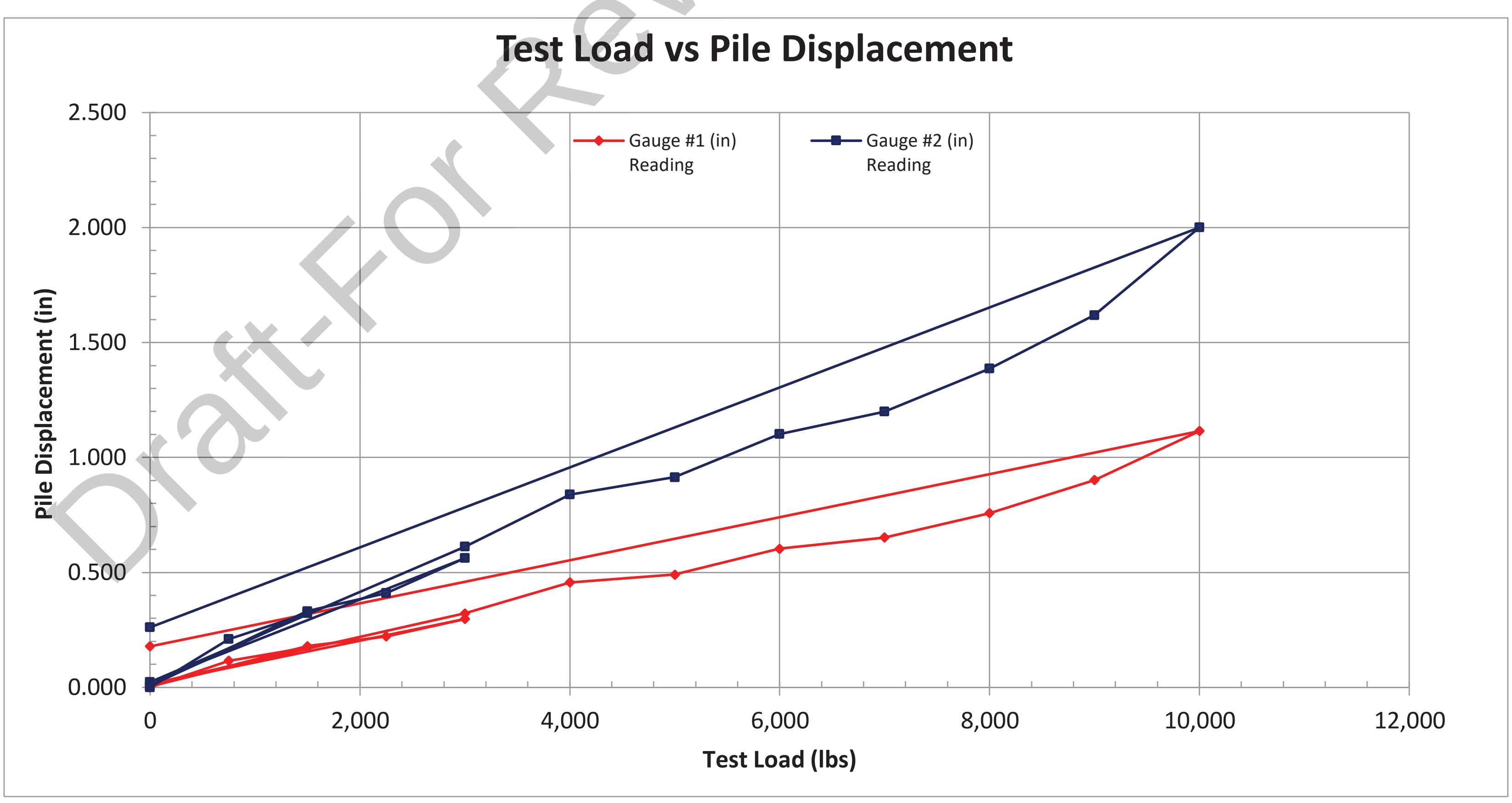


Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-3A-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	75.7	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.116	-0.210	0.116	0.210	
1,500	1	-0.174	-0.322	0.174	0.322	
0	0	-0.004	-0.008	0.004	0.008	
1,500	1	-0.180	-0.332	0.180	0.332	
2,250	1	-0.222	-0.411	0.222	0.411	
3,000	1	-0.298	-0.562	0.298	0.562	
0	0	-0.015	-0.023	0.015	0.023	
3,000	1	-0.322	-0.613	0.322	0.613	
4,000	1	-0.456	-0.839	0.456	0.839	
5,000	1	-0.491	-0.914	0.491	0.914	
6,000	1	-0.603	-1.102	0.603	1.102	
7,000	1	-0.652	-1.199	0.652	1.199	
8,000	1	-0.758	-1.386	0.758	1.386	
9,000	1	-0.901	-1.619	0.901	1.619	
10,000	1	-1.115	-2.000	1.115	2.000	Failure
0	0	-0.179	-0.261	0.179	0.261	

Notes:

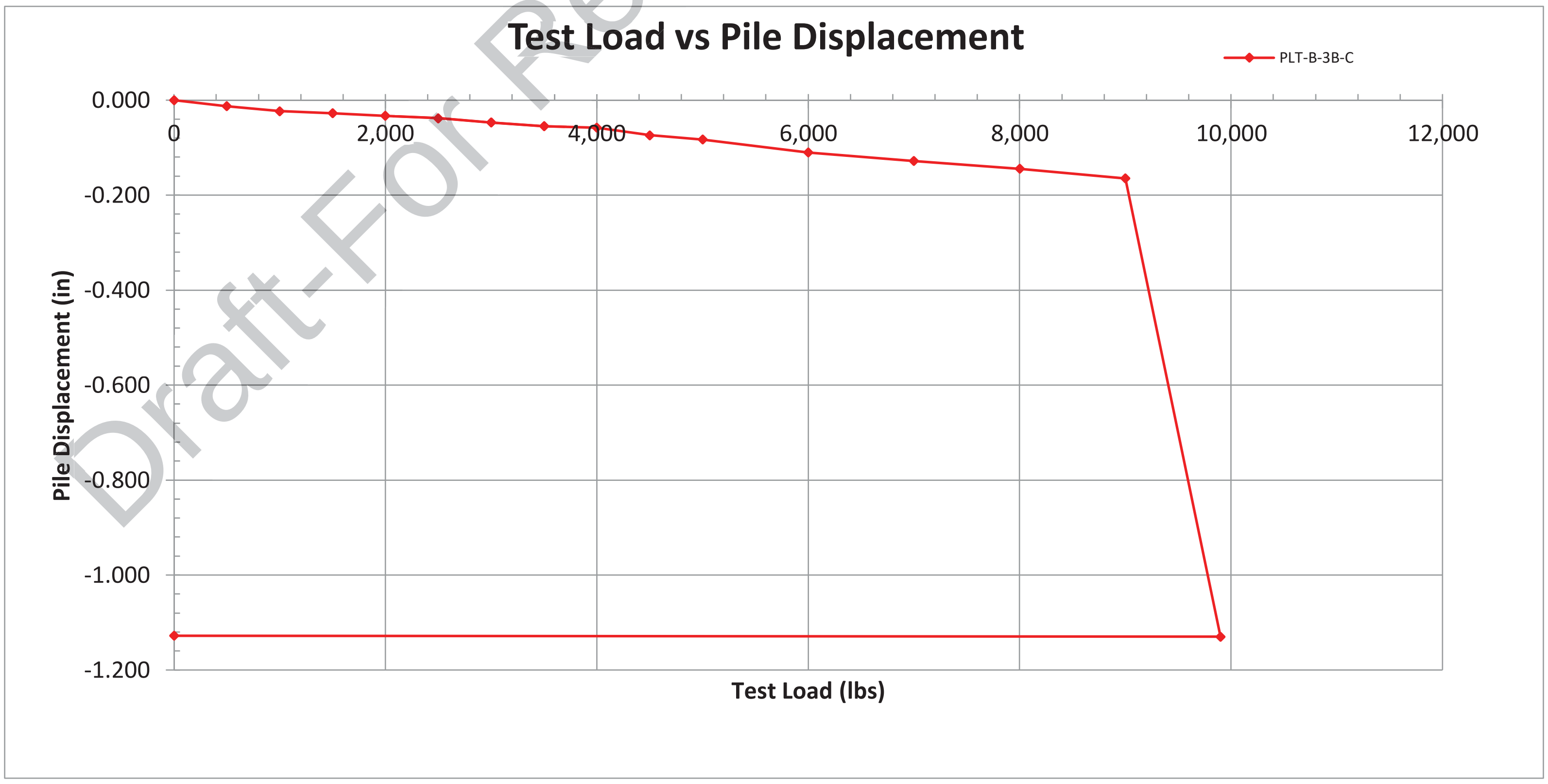


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	dmb
Pile No.	PLT-B-3B-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	137.7	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.022	0.005	-0.022	-0.005	-0.013	
1,000	1 min	0.025	0.022	-0.025	-0.022	-0.023	
1,500	1 min	0.029	0.027	-0.029	-0.027	-0.028	
2,000	1 min	0.038	0.029	-0.038	-0.029	-0.033	
2,500	1 min	0.045	0.032	-0.045	-0.032	-0.038	
3,000	1 min	0.050	0.044	-0.050	-0.044	-0.047	
3,500	1 min	0.055	0.055	-0.055	-0.055	-0.055	
4,000	1 min	0.060	0.057	-0.060	-0.057	-0.058	
4,500	1 min	0.082	0.067	-0.082	-0.067	-0.074	
5,000	1 min	0.092	0.074	-0.092	-0.074	-0.083	
6,000	1 min	0.116	0.105	-0.116	-0.105	-0.110	
7,000	1 min	0.135	0.122	-0.135	-0.122	-0.128	
8,000	1 min	0.154	0.136	-0.154	-0.136	-0.145	
9,000	1 min	0.165	0.165	-0.165	-0.165	-0.165	
9,900	1 min	1.027	1.233	-1.027	-1.233	-1.130	Failure
0	1 min	1.025	1.230	-1.025	-1.230	-1.128	

Notes:

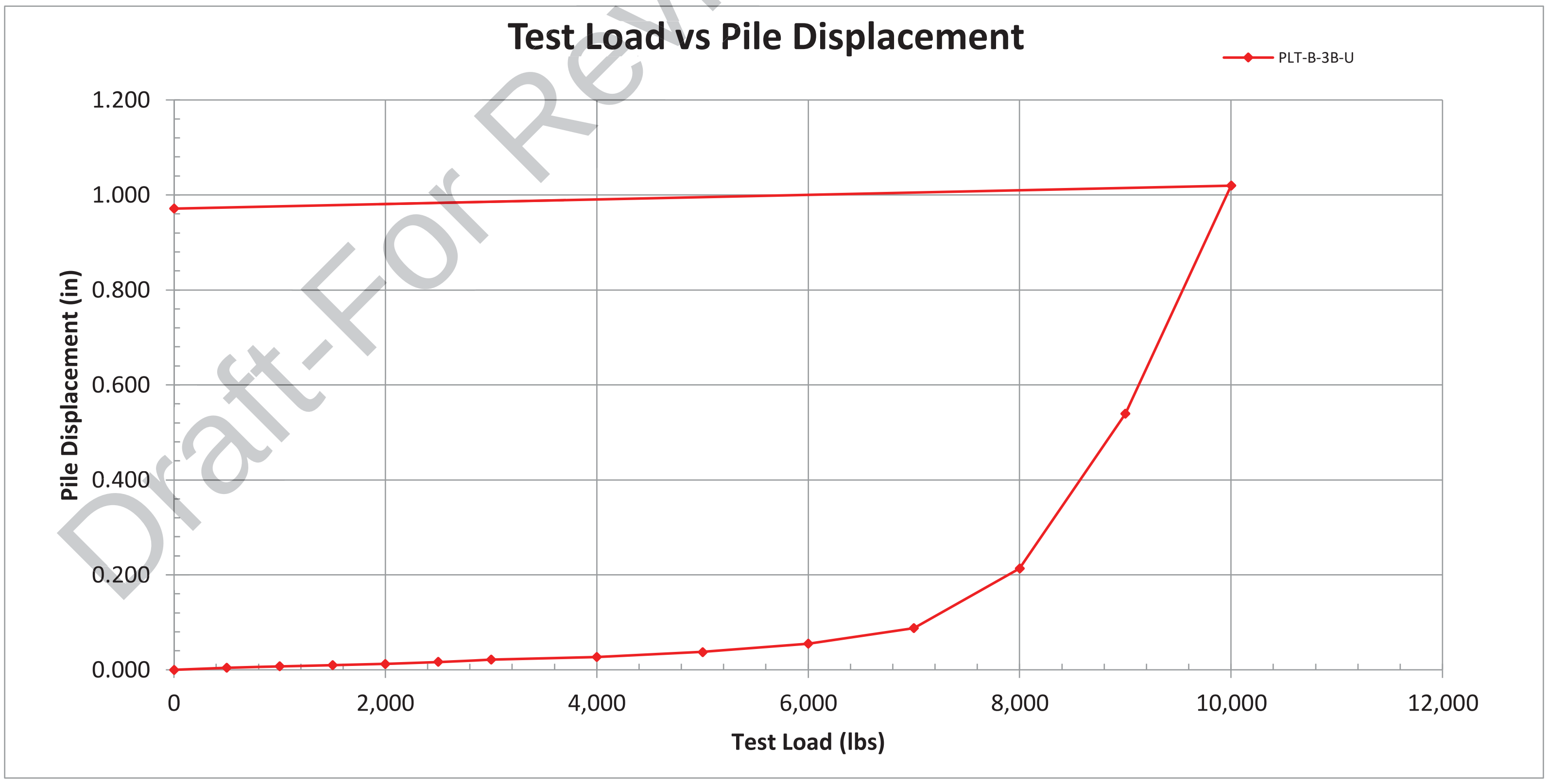


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-3B-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	137.7	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.004	-0.005	0.004	0.005	0.004	
1,000	1 min	-0.007	-0.009	0.007	0.009	0.008	
1,500	1 min	-0.010	-0.011	0.010	0.011	0.010	
2,000	1 min	-0.012	-0.013	0.012	0.013	0.012	
2,500	1 min	-0.017	-0.017	0.017	0.017	0.017	
3,000	1 min	-0.022	-0.022	0.022	0.022	0.022	
4,000	1 min	-0.028	-0.027	0.028	0.027	0.027	
5,000	1 min	-0.038	-0.037	0.038	0.037	0.038	
6,000	1 min	-0.057	-0.054	0.057	0.054	0.055	
7,000	1 min	-0.089	-0.086	0.089	0.086	0.088	
8,000	1 min	-0.215	-0.212	0.215	0.212	0.213	
9,000	1 min	-0.534	-0.545	0.534	0.545	0.539	
10,000	1 min	-1.034	-1.005	1.034	1.005	1.020	Failure
0	1 min	-0.971	-0.972	0.971	0.972	0.971	

Notes:

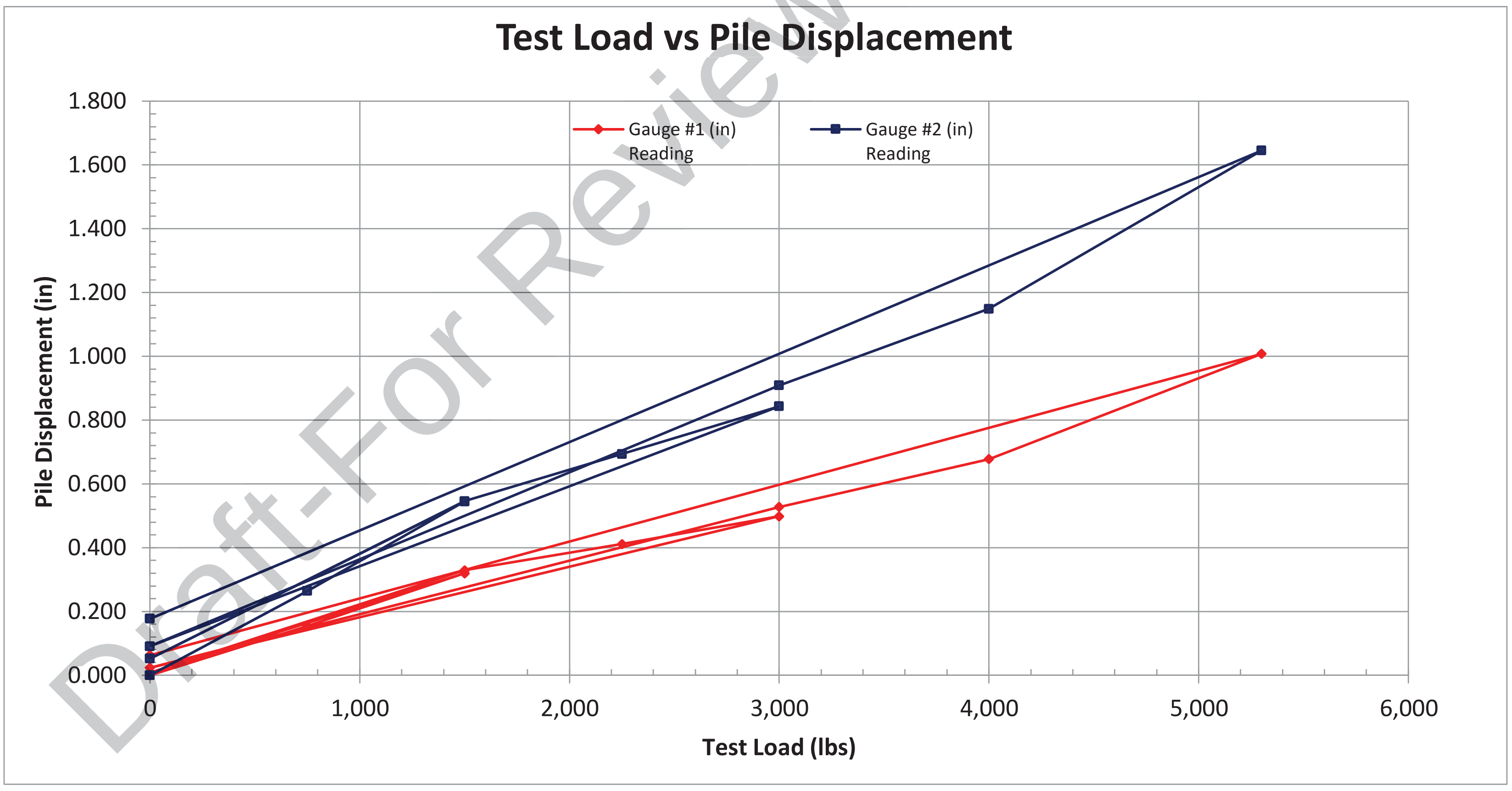


Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-3B-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	137.7	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.155	-0.264	0.155	0.264	
1,500	1	-0.320	-0.545	0.320	0.545	
0	0	-0.009	-0.052	0.009	0.052	
1,500	1	-0.329	-0.546	0.329	0.546	
2,250	1	-0.412	-0.694	0.412	0.694	
3,000	1	-0.499	-0.843	0.499	0.843	
0	0	-0.024	-0.091	0.024	0.091	
3,000	1	-0.528	-0.909	0.528	0.909	
4,000	1	-0.678	-1.149	0.678	1.149	
5,300	1	-1.008	-1.645	1.008	1.645	Failure
0	0	-0.063	-0.177	0.063	0.177	

Notes:

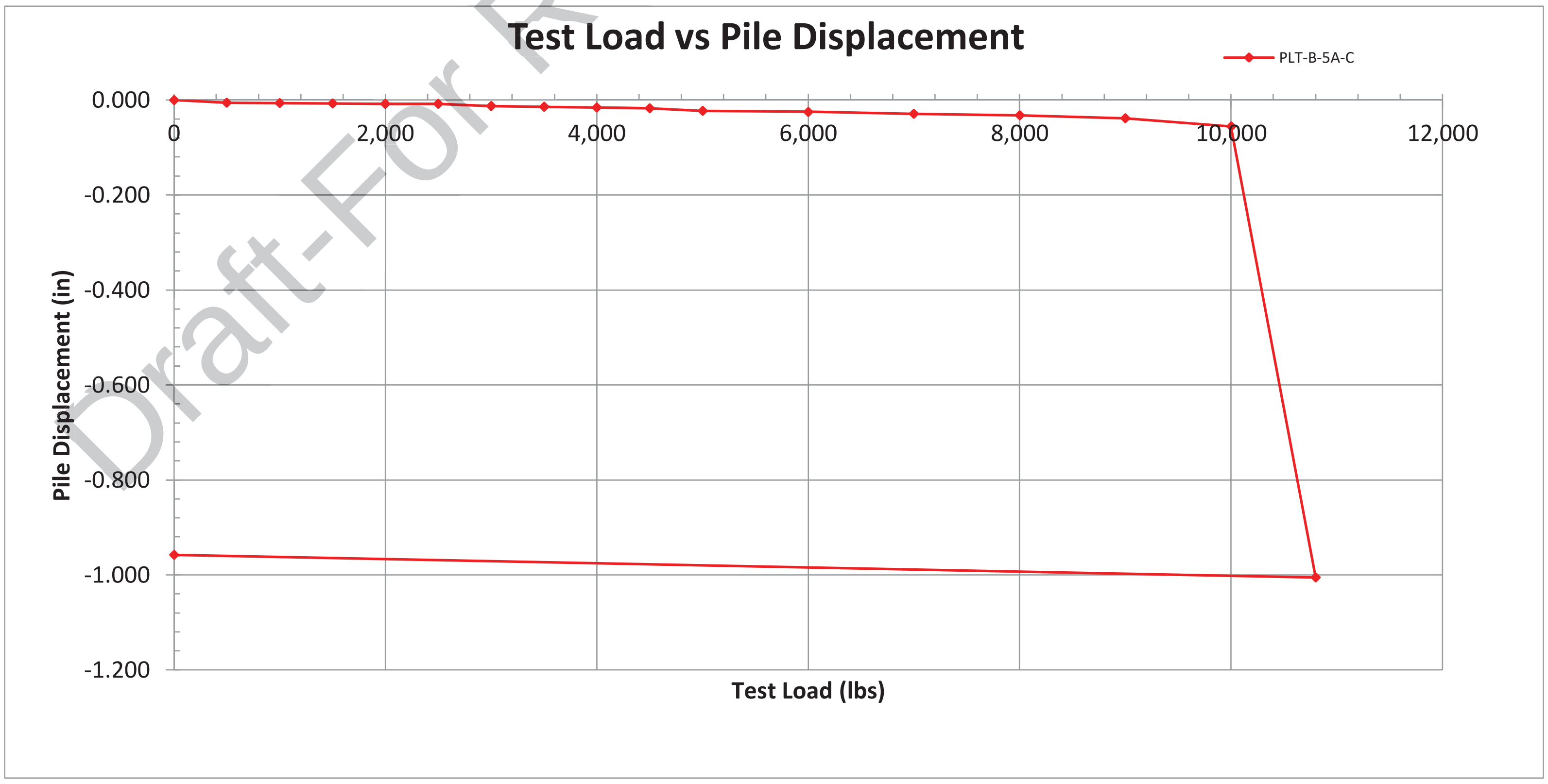


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5A-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	6.0
Pile Stickup Ht (ft):	4.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	41.2	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.006	0.006	-0.006	-0.006	-0.006	
1,000	1 min	0.006	0.007	-0.006	-0.007	-0.007	
1,500	1 min	0.007	0.008	-0.007	-0.008	-0.008	
2,000	1 min	0.008	0.008	-0.008	-0.008	-0.008	
2,500	1 min	0.008	0.009	-0.008	-0.009	-0.008	
3,000	1 min	0.014	0.012	-0.014	-0.012	-0.013	
3,500	1 min	0.015	0.014	-0.015	-0.014	-0.015	
4,000	1 min	0.016	0.016	-0.016	-0.016	-0.016	
4,500	1 min	0.017	0.018	-0.017	-0.018	-0.017	
5,000	1 min	0.022	0.024	-0.022	-0.024	-0.023	
6,000	1 min	0.024	0.026	-0.024	-0.026	-0.025	
7,000	1 min	0.028	0.031	-0.028	-0.031	-0.029	
8,000	1 min	0.031	0.035	-0.031	-0.035	-0.033	
9,000	1 min	0.037	0.041	-0.037	-0.041	-0.039	
10,000	1 min	0.056	0.056	-0.056	-0.056	-0.056	
10,800	1 min	1.006	1.005	-1.006	-1.005	-1.006	Failure
0	1 min	0.963	0.953	-0.963	-0.953	-0.958	

Notes:

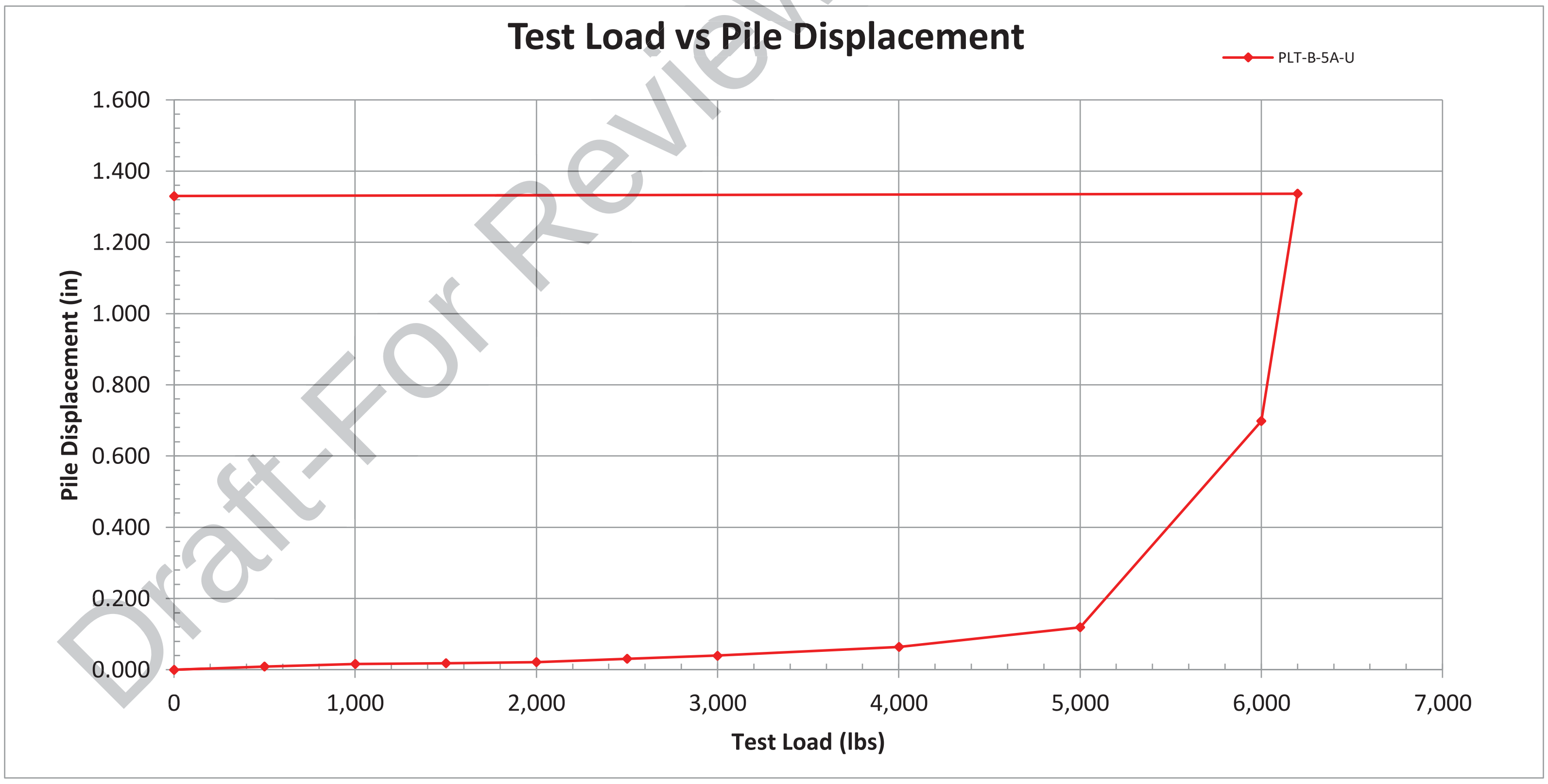


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5A-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	6.0
Pile Stickup Ht (ft):	4.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	41.2	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.011	-0.008	0.011	0.008	0.009	
1,000	1 min	-0.018	-0.015	0.018	0.015	0.016	
1,500	1 min	-0.020	-0.017	0.020	0.017	0.018	
2,000	1 min	-0.022	-0.022	0.022	0.022	0.022	
2,500	1 min	-0.031	-0.031	0.031	0.031	0.031	
3,000	1 min	-0.039	-0.040	0.039	0.040	0.040	
4,000	1 min	-0.062	-0.066	0.062	0.066	0.064	
5,000	1 min	-0.115	-0.123	0.115	0.123	0.119	
6,000	1 min	-0.693	-0.704	0.693	0.704	0.698	
6,200	1 min	-1.335	-1.338	1.335	1.338	1.337	Failure
0	1 min	-1.329	-1.331	1.329	1.331	1.330	

Notes:

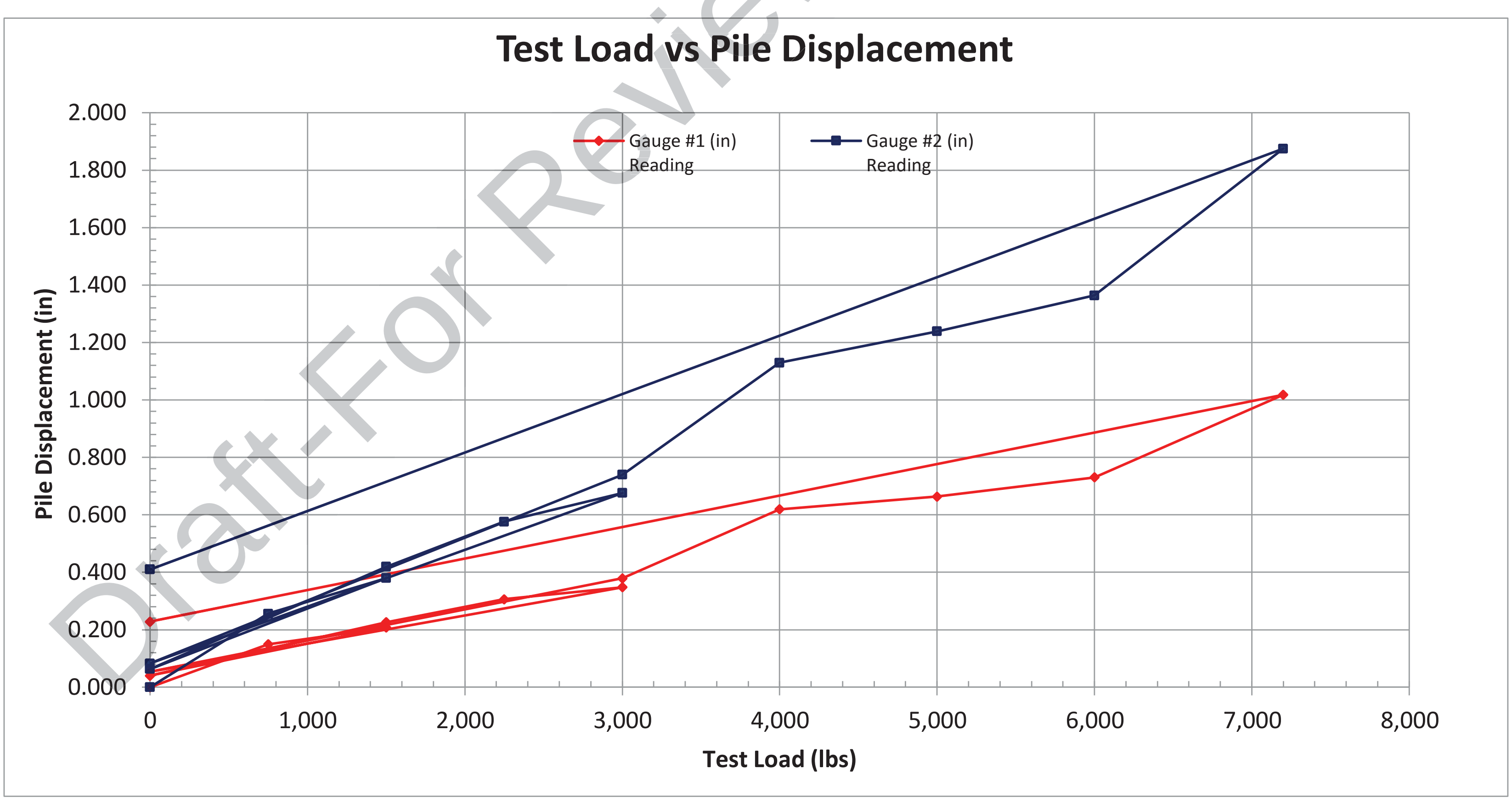


Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/4/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5A-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	6.0
Pile Stickup Ht (ft):	4.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	41.2	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.149	-0.256	0.149	0.256	
1,500	1	-0.208	-0.380	0.208	0.380	
0	0	-0.040	-0.063	0.040	0.063	
1,500	1	-0.227	-0.420	0.227	0.420	
2,250	1	-0.306	-0.576	0.306	0.576	
3,000	1	-0.348	-0.676	0.348	0.676	
0	0	-0.054	-0.083	0.054	0.083	
3,000	1	-0.379	-0.740	0.379	0.740	
4,000	1	-0.619	-1.130	0.619	1.130	
5,000	1	-0.664	-1.239	0.664	1.239	
6,000	1	-0.731	-1.364	0.731	1.364	
7,200	1	-1.018	-1.875	1.018	1.875	Failure
0	0	-0.228	-0.410	0.228	0.410	

Notes:

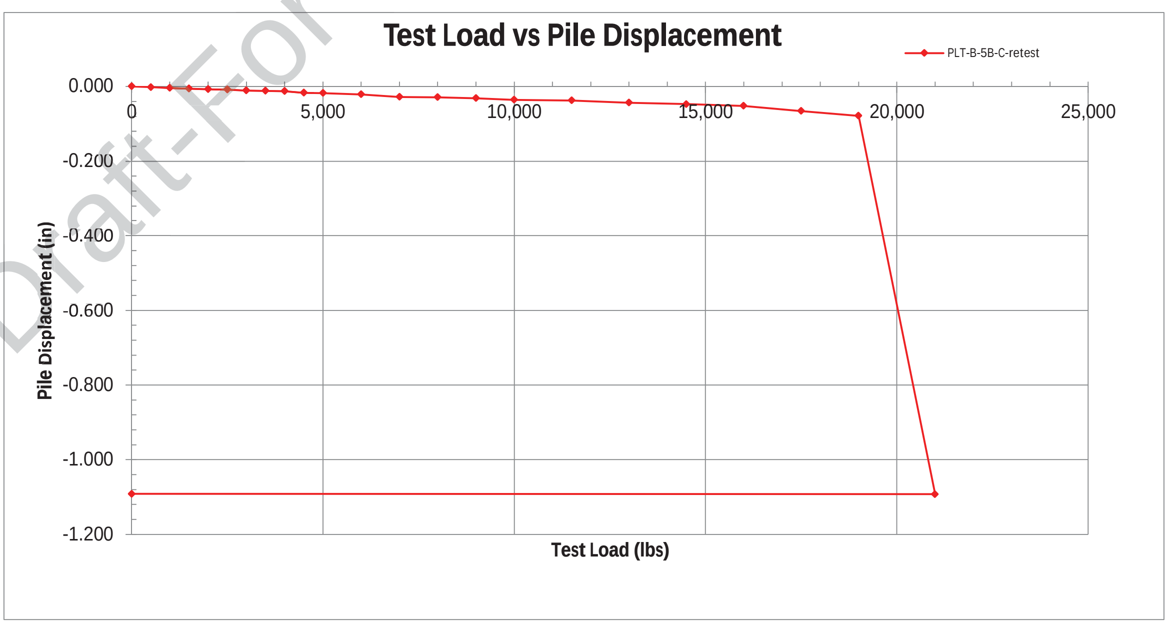


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/9/2023
Project No.:	GE2306031	Pile Test Date:	10/11/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5B-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	8.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	86.9	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.002	0.002	-0.002	-0.002	-0.002	
1,000	1 min	0.005	0.003	-0.005	-0.003	-0.004	
1,500	1 min	0.007	0.005	-0.007	-0.005	-0.006	
2,000	1 min	0.008	0.007	-0.008	-0.007	-0.007	
2,500	1 min	0.008	0.009	-0.008	-0.009	-0.009	
3,000	1 min	0.012	0.010	-0.012	-0.010	-0.011	
3,500	1 min	0.013	0.010	-0.013	-0.010	-0.012	
4,000	1 min	0.014	0.012	-0.014	-0.012	-0.013	
4,500	1 min	0.018	0.017	-0.018	-0.017	-0.017	
5,000	1 min	0.018	0.018	-0.018	-0.018	-0.018	
6,000	1 min	0.020	0.022	-0.020	-0.022	-0.021	
7,000	1 min	0.028	0.028	-0.028	-0.028	-0.028	
8,000	1 min	0.029	0.029	-0.029	-0.029	-0.029	
9,000	1 min	0.031	0.033	-0.031	-0.033	-0.032	
10,000	1 min	0.037	0.035	-0.037	-0.035	-0.036	
11,500	1 min	0.038	0.037	-0.038	-0.037	-0.038	
13,000	1 min	0.044	0.044	-0.044	-0.044	-0.044	
14,500	1 min	0.047	0.048	-0.047	-0.048	-0.047	
16,000	1 min	0.051	0.054	-0.051	-0.054	-0.052	
17,500	1 min	0.065	0.067	-0.065	-0.067	-0.066	
19,000	1 min	0.080	0.078	-0.080	-0.078	-0.079	
21,000	1 min	1.135	1.050	-1.135	-1.050	-1.092	Failure
0	1 min	1.134	1.049	-1.134	-1.049	-1.092	

Notes:

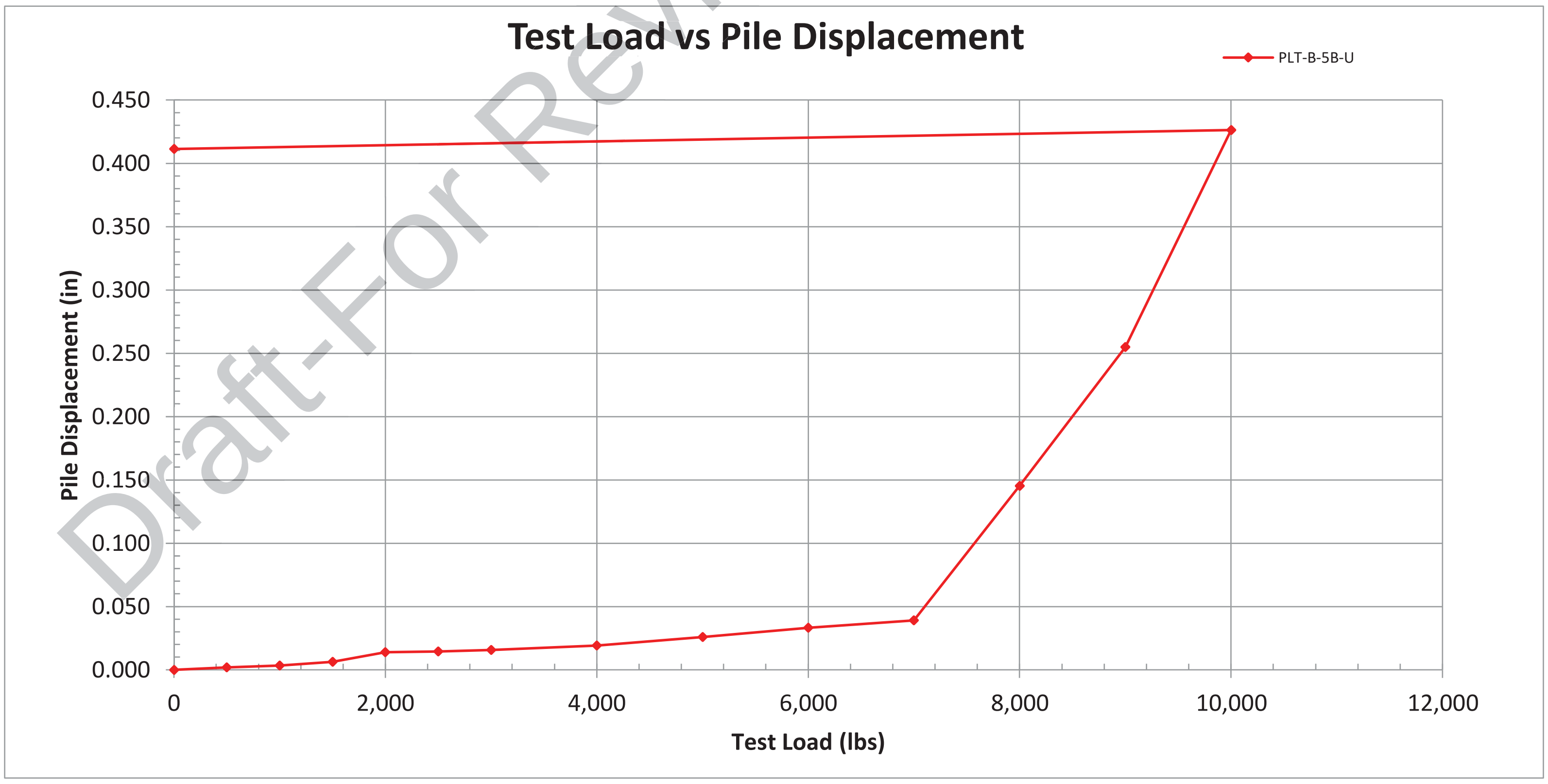


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5B-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	8.0
Pile Stickup Ht (ft):	5.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	98.7	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.001	-0.003	0.001	0.003	0.002	
1,000	1 min	-0.003	-0.004	0.003	0.004	0.004	
1,500	1 min	-0.005	-0.008	0.005	0.008	0.006	
2,000	1 min	-0.012	-0.016	0.012	0.016	0.014	
2,500	1 min	-0.013	-0.017	0.013	0.017	0.015	
3,000	1 min	-0.014	-0.018	0.014	0.018	0.016	
4,000	1 min	-0.017	-0.022	0.017	0.022	0.019	
5,000	1 min	-0.023	-0.030	0.023	0.030	0.026	
6,000	1 min	-0.030	-0.037	0.030	0.037	0.033	
7,000	1 min	-0.036	-0.043	0.036	0.043	0.039	
8,000	1 min	-0.137	-0.154	0.137	0.154	0.145	
9,000	1 min	-0.248	-0.262	0.248	0.262	0.255	
10,000	1 min	-0.419	-0.434	0.419	0.434	0.426	
0	1 min	-0.409	-0.414	0.409	0.414	0.411	

Notes:

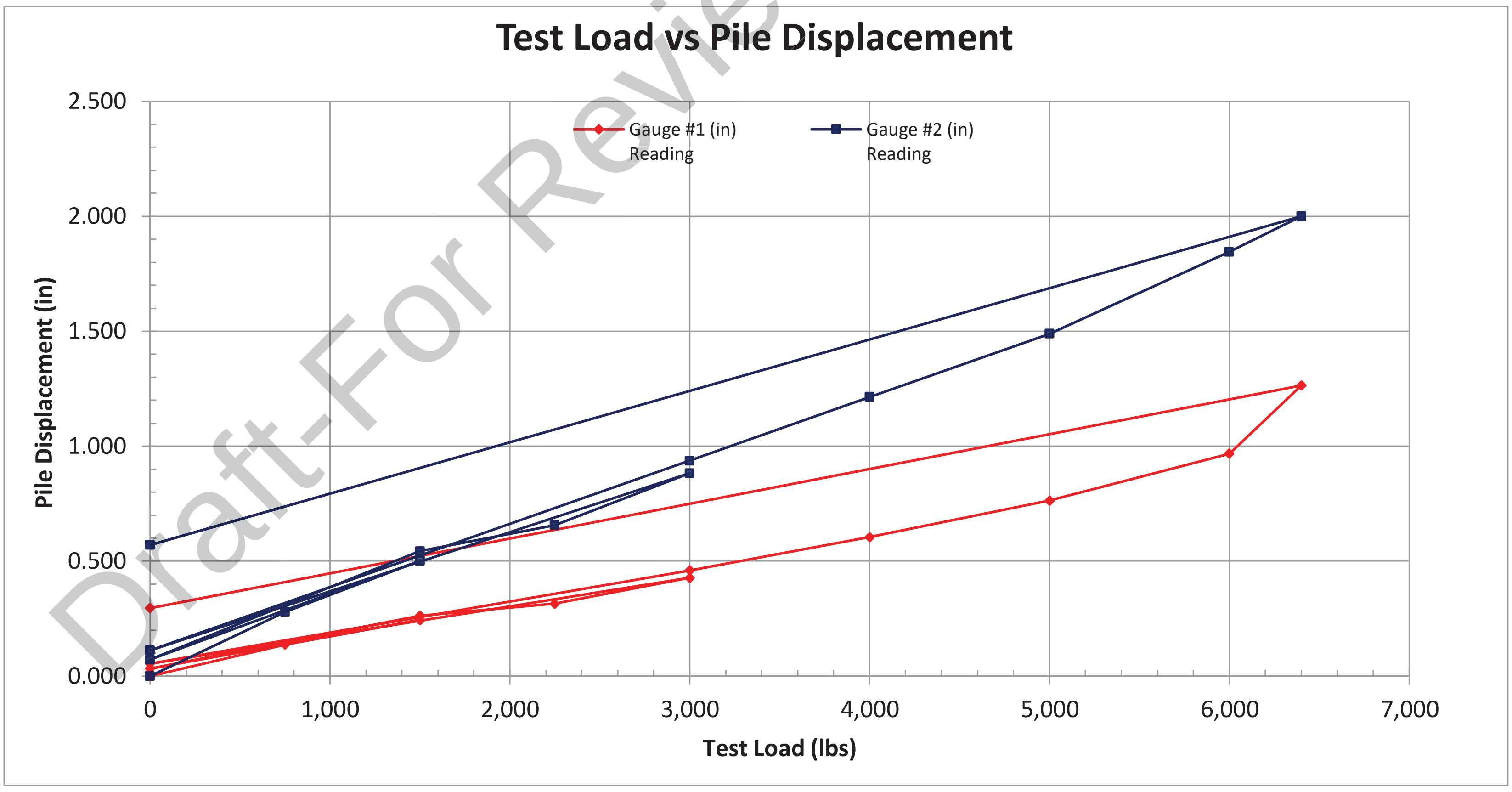


Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-5B-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	8.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	86.9	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.137	-0.279	0.137	0.279	
1,500	1	-0.244	-0.502	0.244	0.502	
0	0	-0.033	-0.072	0.033	0.072	
1,500	1	-0.263	-0.543	0.263	0.543	
2,250	1	-0.315	-0.657	0.315	0.657	
3,000	1	-0.428	-0.883	0.428	0.883	
0	0	-0.055	-0.112	0.055	0.112	
3,000	1	-0.460	-0.937	0.460	0.937	
4,000	1	-0.605	-1.215	0.605	1.215	
5,000	1	-0.763	-1.489	0.763	1.489	
6,000	1	-0.968	-1.846	0.968	1.846	
6,400	1	-1.264	-2.000	1.264	2.000	Failure
0	0	-0.296	-0.571	0.296	0.571	

Notes:

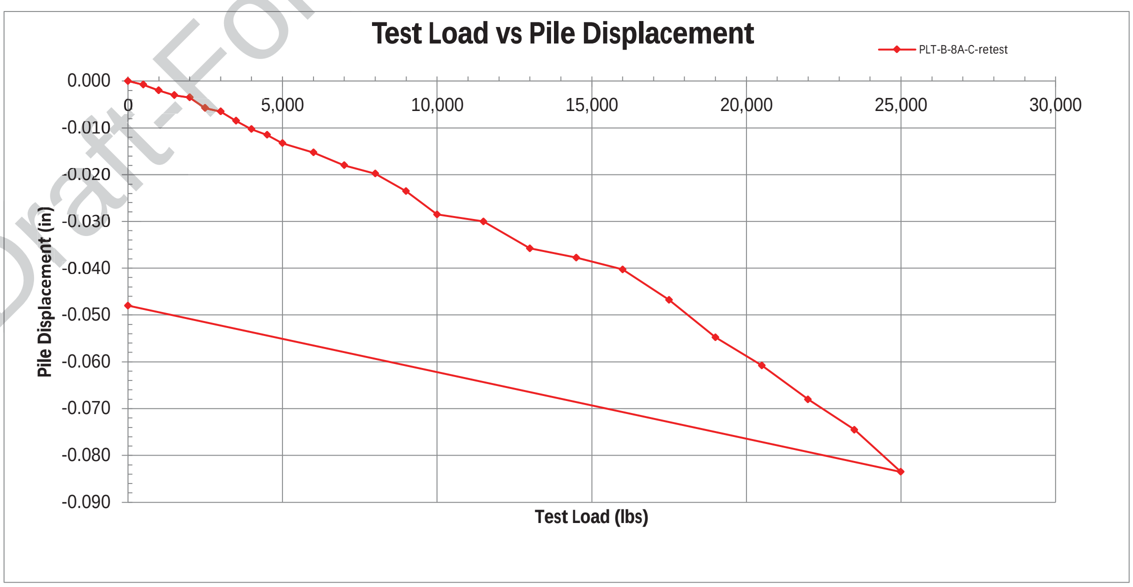


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/9/2023
Project No.:	GE2306031	Pile Test Date:	10/11/2023
Client:	NextEra Energy	Tested by:	dmb
Pile No.	PLT-B-8A-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	81.3	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.001	0.001	-0.001	-0.001	-0.001	
1,000	1 min	0.002	0.002	-0.002	-0.002	-0.002	
1,500	1 min	0.003	0.003	-0.003	-0.003	-0.003	
2,000	1 min	0.003	0.004	-0.003	-0.004	-0.004	
2,500	1 min	0.006	0.006	-0.006	-0.006	-0.006	
3,000	1 min	0.007	0.006	-0.007	-0.006	-0.007	
3,500	1 min	0.009	0.008	-0.009	-0.008	-0.009	
4,000	1 min	0.011	0.010	-0.011	-0.010	-0.010	
4,500	1 min	0.011	0.012	-0.011	-0.012	-0.012	
5,000	1 min	0.014	0.013	-0.014	-0.013	-0.013	
6,000	1 min	0.016	0.015	-0.016	-0.015	-0.015	
7,000	1 min	0.018	0.019	-0.018	-0.019	-0.018	
8,000	1 min	0.019	0.021	-0.019	-0.021	-0.020	
9,000	1 min	0.022	0.026	-0.022	-0.026	-0.024	
10,000	1 min	0.027	0.031	-0.027	-0.031	-0.029	
11,500	1 min	0.028	0.032	-0.028	-0.032	-0.030	
13,000	1 min	0.037	0.035	-0.037	-0.035	-0.036	
14,500	1 min	0.039	0.037	-0.039	-0.037	-0.038	
16,000	1 min	0.040	0.041	-0.040	-0.041	-0.040	
17,500	1 min	0.047	0.047	-0.047	-0.047	-0.047	
19,000	1 min	0.054	0.056	-0.054	-0.056	-0.055	
20,500	1 min	0.061	0.061	-0.061	-0.061	-0.061	
22,000	1 min	0.068	0.068	-0.068	-0.068	-0.068	
23,500	1 min	0.074	0.076	-0.074	-0.076	-0.075	
25,000	1 min	0.085	0.083	-0.085	-0.083	-0.084	
0	1 min	0.056	0.040	-0.056	-0.040	-0.048	

Notes:

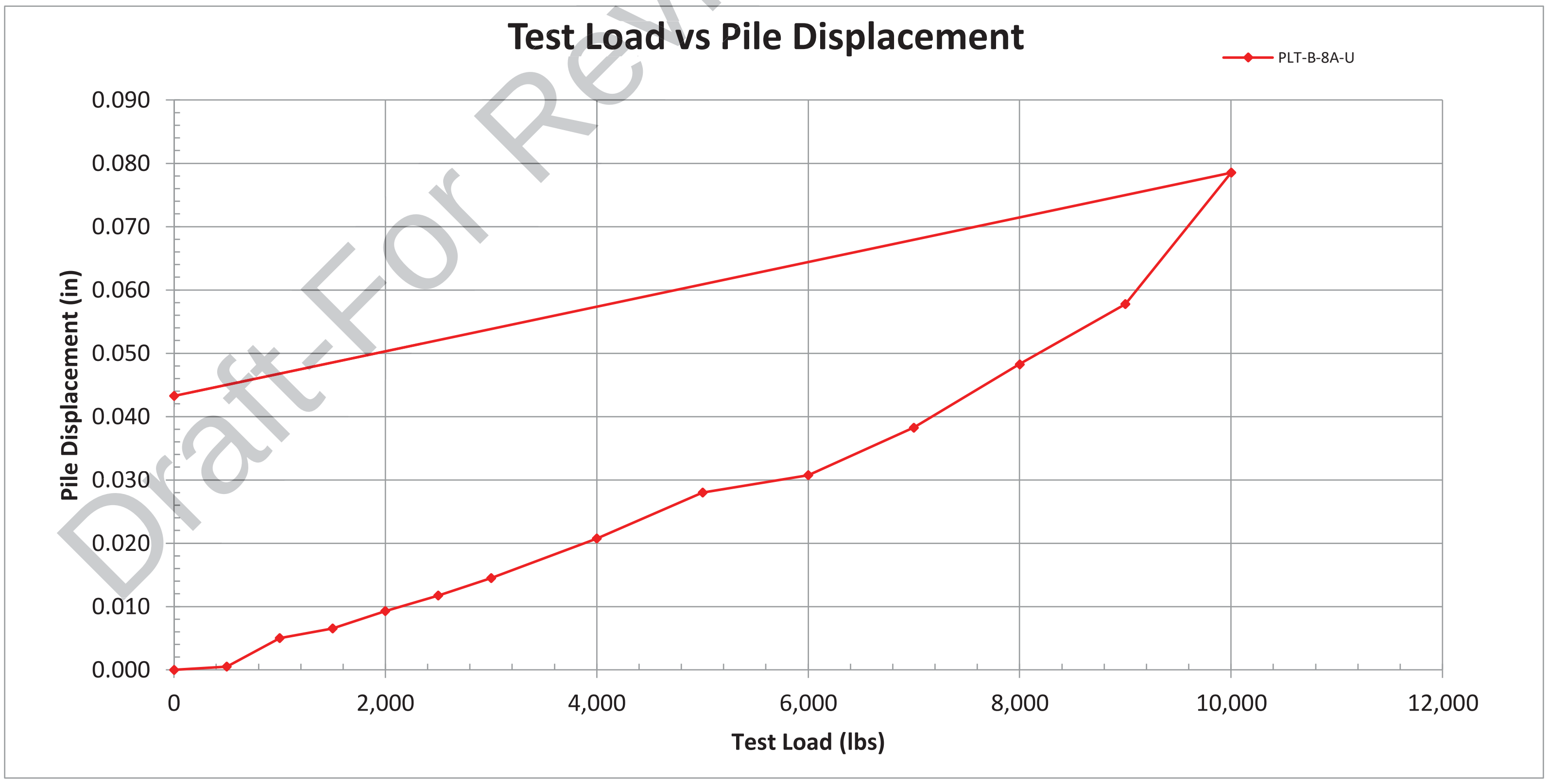


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-8A-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	108.5	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.001	-0.001	0.001	0.001	0.001	
1,000	1 min	-0.005	-0.005	0.005	0.005	0.005	
1,500	1 min	-0.007	-0.007	0.007	0.007	0.007	
2,000	1 min	-0.010	-0.009	0.010	0.009	0.009	
2,500	1 min	-0.012	-0.012	0.012	0.012	0.012	
3,000	1 min	-0.015	-0.014	0.015	0.014	0.015	
4,000	1 min	-0.022	-0.020	0.022	0.020	0.021	
5,000	1 min	-0.029	-0.027	0.029	0.027	0.028	
6,000	1 min	-0.032	-0.030	0.032	0.030	0.031	
7,000	1 min	-0.039	-0.038	0.039	0.038	0.038	
8,000	1 min	-0.049	-0.048	0.049	0.048	0.048	
9,000	1 min	-0.059	-0.057	0.059	0.057	0.058	
10,000	1 min	-0.079	-0.079	0.079	0.079	0.079	
0	1 min	-0.043	-0.044	0.043	0.044	0.043	

Notes:



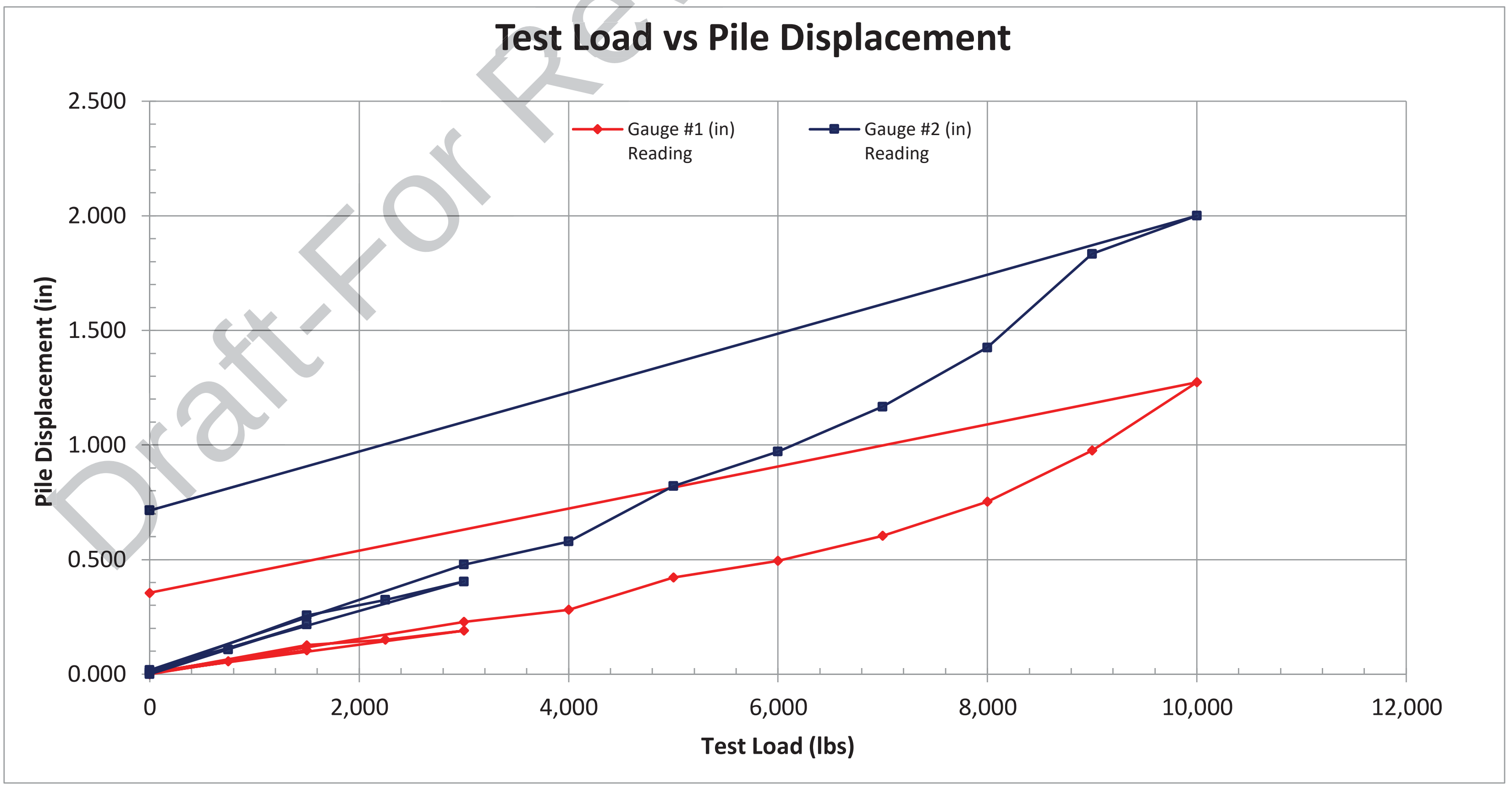
RRC

Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-8A-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	108.5	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.057	-0.108	0.057	0.108	
1,500	1	-0.104	-0.218	0.104	0.218	
0	0	-0.004	-0.008	0.004	0.008	
1,500	1	-0.127	-0.256	0.127	0.256	
2,250	1	-0.151	-0.325	0.151	0.325	
3,000	1	-0.191	-0.405	0.191	0.405	
0	0	-0.007	-0.018	0.007	0.018	
3,000	1	-0.229	-0.479	0.229	0.479	
4,000	1	-0.281	-0.579	0.281	0.579	
5,000	1	-0.422	-0.821	0.422	0.821	
6,000	1	-0.494	-0.972	0.494	0.972	
7,000	1	-0.604	-1.167	0.604	1.167	
8,000	1	-0.753	-1.425	0.753	1.425	
9,000	1	-0.976	-1.834	0.976	1.834	
10,000	1	-1.274	-2.000	1.274	2.000	Failure
0	0	-0.355	-0.714	0.355	0.714	

Notes:

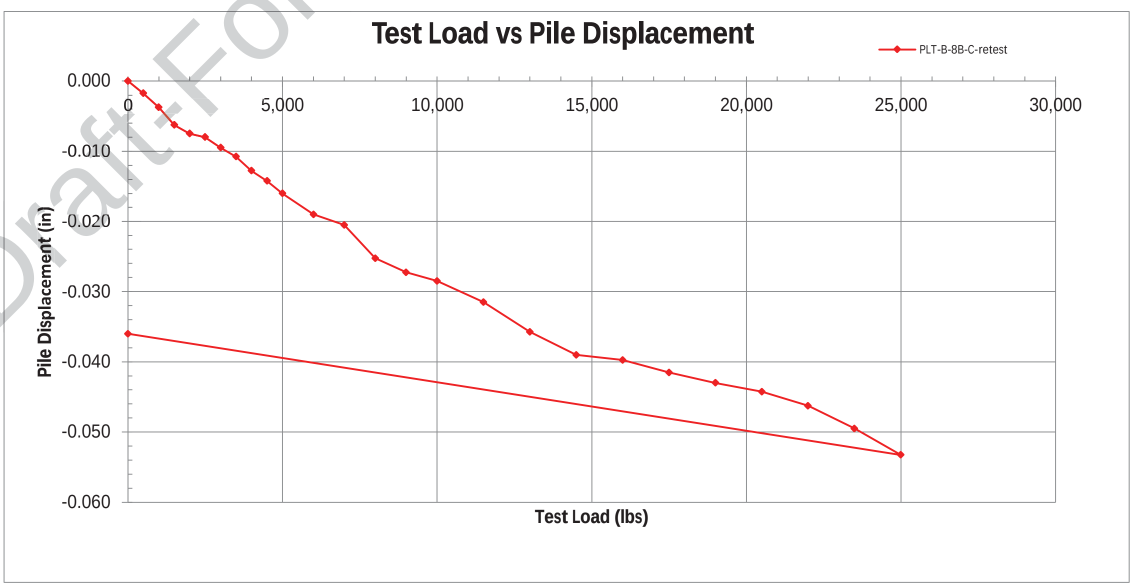


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/9/2023
Project No.:	GE2306031	Pile Test Date:	10/11/2023
Client:	NextEra Energy	Tested by:	dmb
Pile No.	PLT-B-8B-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	216.3	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.002	0.002	-0.002	-0.002	-0.002	
1,000	1 min	0.004	0.004	-0.004	-0.004	-0.004	
1,500	1 min	0.005	0.008	-0.005	-0.008	-0.006	
2,000	1 min	0.006	0.009	-0.006	-0.009	-0.008	
2,500	1 min	0.007	0.010	-0.007	-0.010	-0.008	
3,000	1 min	0.008	0.011	-0.008	-0.011	-0.010	
3,500	1 min	0.010	0.012	-0.010	-0.012	-0.011	
4,000	1 min	0.012	0.014	-0.012	-0.014	-0.013	
4,500	1 min	0.013	0.016	-0.013	-0.016	-0.014	
5,000	1 min	0.016	0.017	-0.016	-0.017	-0.016	
6,000	1 min	0.019	0.019	-0.019	-0.019	-0.019	
7,000	1 min	0.021	0.021	-0.021	-0.021	-0.021	
8,000	1 min	0.026	0.025	-0.026	-0.025	-0.025	
9,000	1 min	0.028	0.027	-0.028	-0.027	-0.027	
10,000	1 min	0.029	0.028	-0.029	-0.028	-0.029	
11,500	1 min	0.031	0.032	-0.031	-0.032	-0.032	
13,000	1 min	0.038	0.034	-0.038	-0.034	-0.036	
14,500	1 min	0.040	0.038	-0.040	-0.038	-0.039	
16,000	1 min	0.041	0.039	-0.041	-0.039	-0.040	
17,500	1 min	0.043	0.041	-0.043	-0.041	-0.042	
19,000	1 min	0.044	0.042	-0.044	-0.042	-0.043	
20,500	1 min	0.046	0.043	-0.046	-0.043	-0.044	
22,000	1 min	0.048	0.045	-0.048	-0.045	-0.046	
23,500	1 min	0.052	0.048	-0.052	-0.048	-0.050	
25,000	1 min	0.055	0.052	-0.055	-0.052	-0.053	
0	1 min	0.035	0.037	-0.035	-0.037	-0.036	

Notes:



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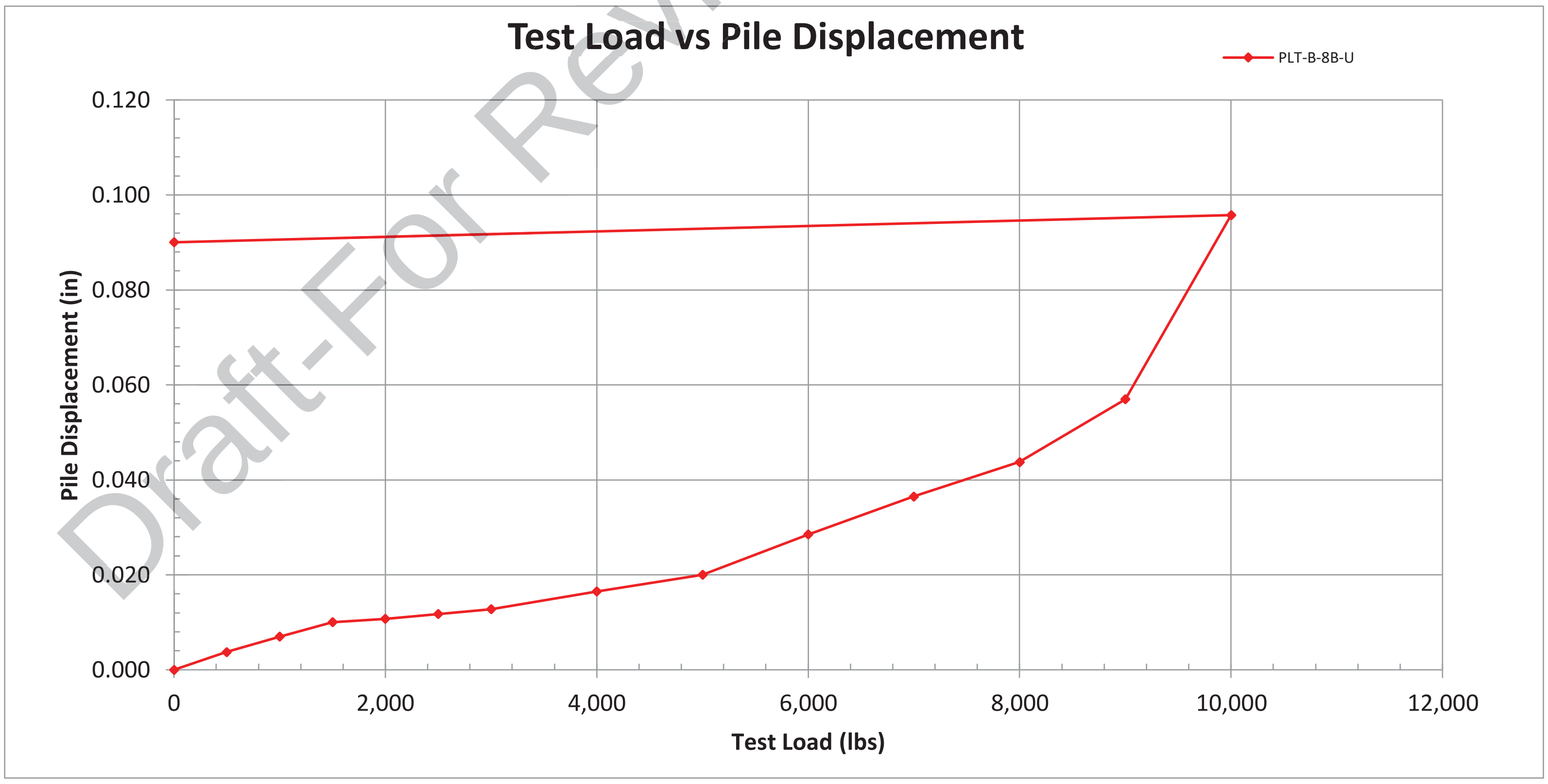
Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-8B-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	213.8	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.005	-0.003	0.005	0.003	0.004	
1,000	1 min	-0.010	-0.005	0.010	0.005	0.007	
1,500	1 min	-0.012	-0.008	0.012	0.008	0.010	
2,000	1 min	-0.013	-0.009	0.013	0.009	0.011	
2,500	1 min	-0.014	-0.010	0.014	0.010	0.012	
3,000	1 min	-0.015	-0.011	0.015	0.011	0.013	
4,000	1 min	-0.019	-0.014	0.019	0.014	0.017	
5,000	1 min	-0.022	-0.018	0.022	0.018	0.020	
6,000	1 min	-0.031	-0.027	0.031	0.027	0.029	
7,000	1 min	-0.038	-0.035	0.038	0.035	0.037	
8,000	1 min	-0.046	-0.042	0.046	0.042	0.044	
9,000	1 min	-0.059	-0.056	0.059	0.056	0.057	
10,000	1 min	-0.097	-0.095	0.097	0.095	0.096	
0	1 min	-0.097	-0.083	0.097	0.083	0.090	

Notes:

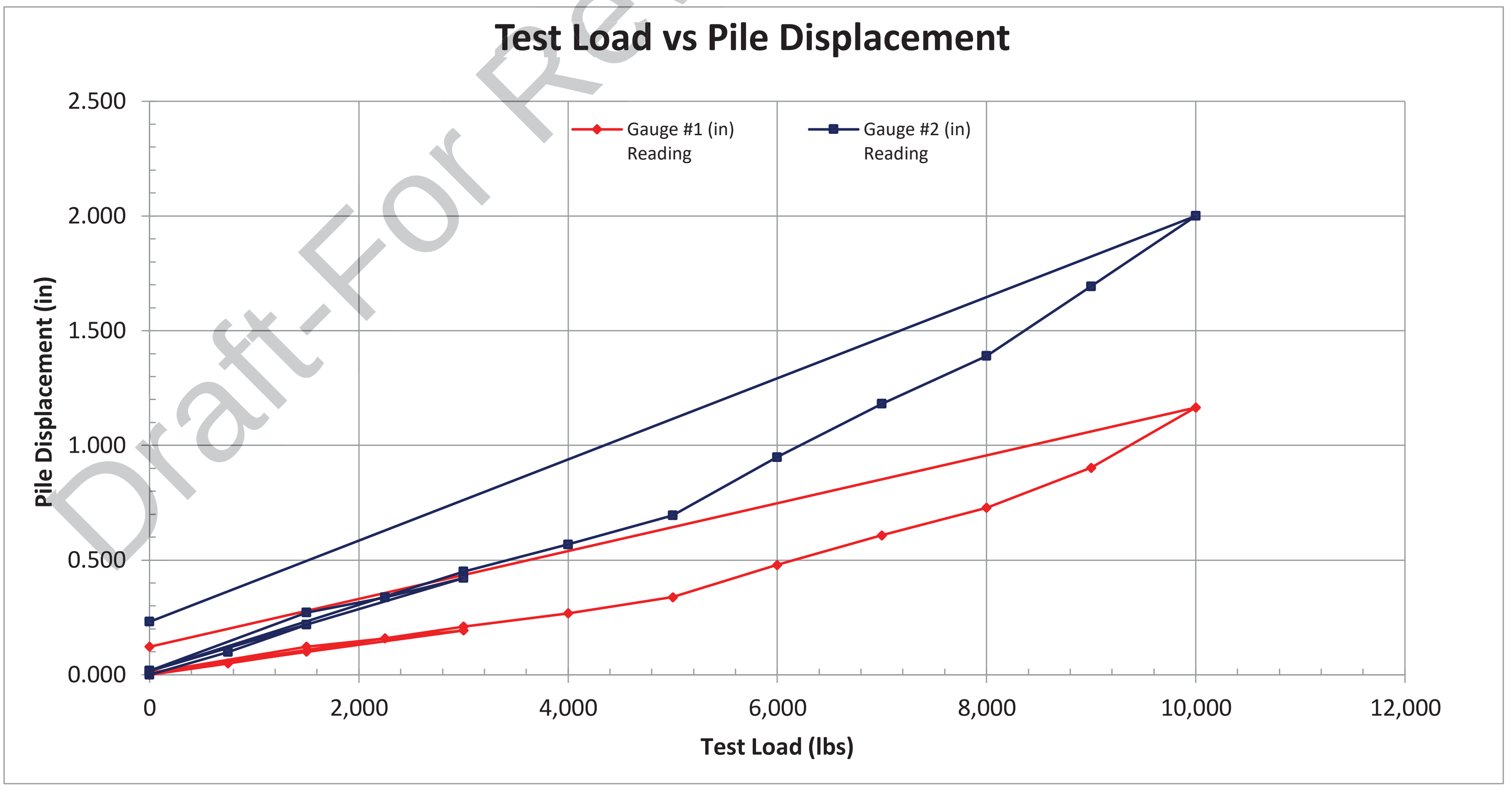


Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/5/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-8B-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	213.8	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.049	-0.099	0.049	0.099	
1,500	1	-0.102	-0.219	0.102	0.219	
0	0	-0.008	-0.018	0.008	0.018	
1,500	1	-0.123	-0.272	0.123	0.272	
2,250	1	-0.158	-0.338	0.158	0.338	
3,000	1	-0.193	-0.422	0.193	0.422	
0	0	-0.007	-0.016	0.007	0.016	
3,000	1	-0.210	-0.450	0.210	0.450	
4,000	1	-0.268	-0.568	0.268	0.568	
5,000	1	-0.338	-0.695	0.338	0.695	
6,000	1	-0.479	-0.949	0.479	0.949	
7,000	1	-0.608	-1.181	0.608	1.181	
8,000	1	-0.728	-1.390	0.728	1.390	
9,000	1	-0.903	-1.694	0.903	1.694	
10,000	1	-1.165	-2.000	1.165	2.000	Failure
0	0	-0.123	-0.232	0.123	0.232	

Notes:

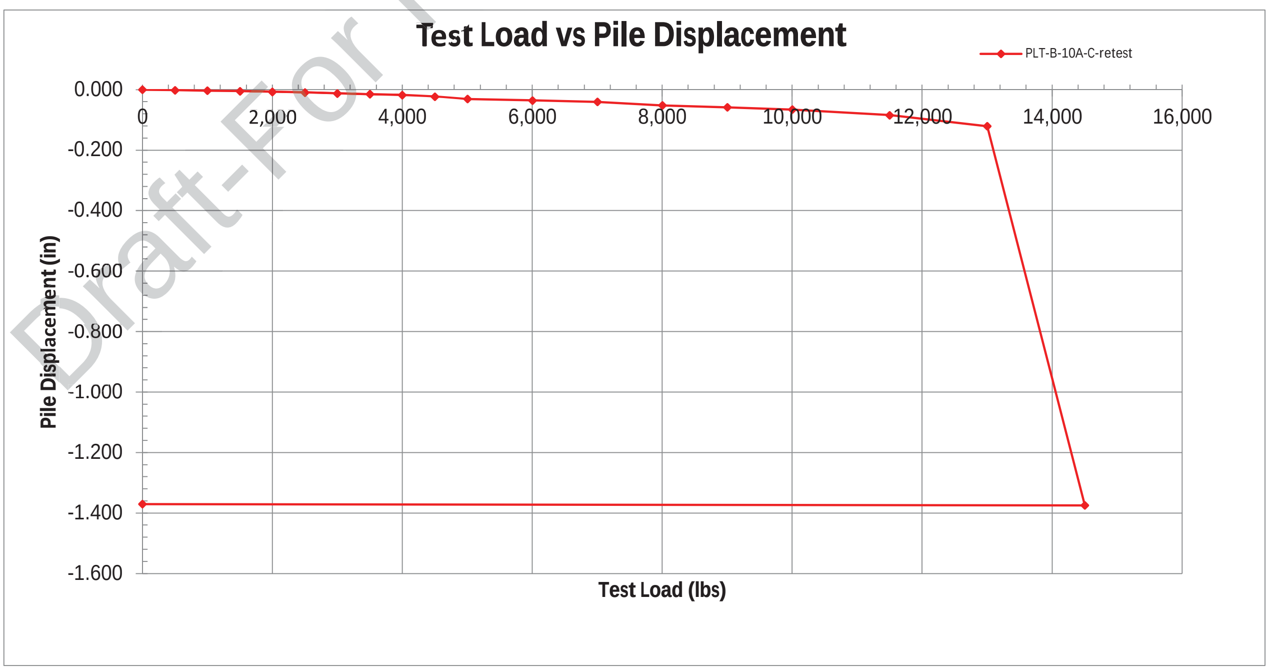


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/9/2023
Project No.:	GE2306031	Pile Test Date:	10/10/2023
Client:	NextEra Energy	Tested by:	dmb
Pile No.	PLT-B-10A-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	84.1	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.002	0.002	-0.002	-0.002	-0.002	
1,000	1 min	0.003	0.004	-0.003	-0.004	-0.004	
1,500	1 min	0.005	0.005	-0.005	-0.005	-0.005	
2,000	1 min	0.007	0.007	-0.007	-0.007	-0.007	
2,500	1 min	0.009	0.010	-0.009	-0.010	-0.009	
3,000	1 min	0.012	0.013	-0.012	-0.013	-0.013	
3,500	1 min	0.014	0.016	-0.014	-0.016	-0.015	
4,000	1 min	0.018	0.018	-0.018	-0.018	-0.018	
4,500	1 min	0.022	0.024	-0.022	-0.024	-0.023	
5,000	1 min	0.033	0.029	-0.033	-0.029	-0.031	
6,000	1 min	0.036	0.036	-0.036	-0.036	-0.036	
7,000	1 min	0.040	0.041	-0.040	-0.041	-0.040	
8,000	1 min	0.050	0.054	-0.050	-0.054	-0.052	
9,000	1 min	0.058	0.059	-0.058	-0.059	-0.058	
10,000	1 min	0.063	0.069	-0.063	-0.069	-0.066	
11,500	1 min	0.081	0.088	-0.081	-0.088	-0.084	
13,000	1 min	0.114	0.128	-0.114	-0.128	-0.121	
14,500	1 min	1.371	1.377	-1.371	-1.377	-1.374	Failure
0	1 min	1.371	1.370	-1.371	-1.370	-1.371	

Notes:

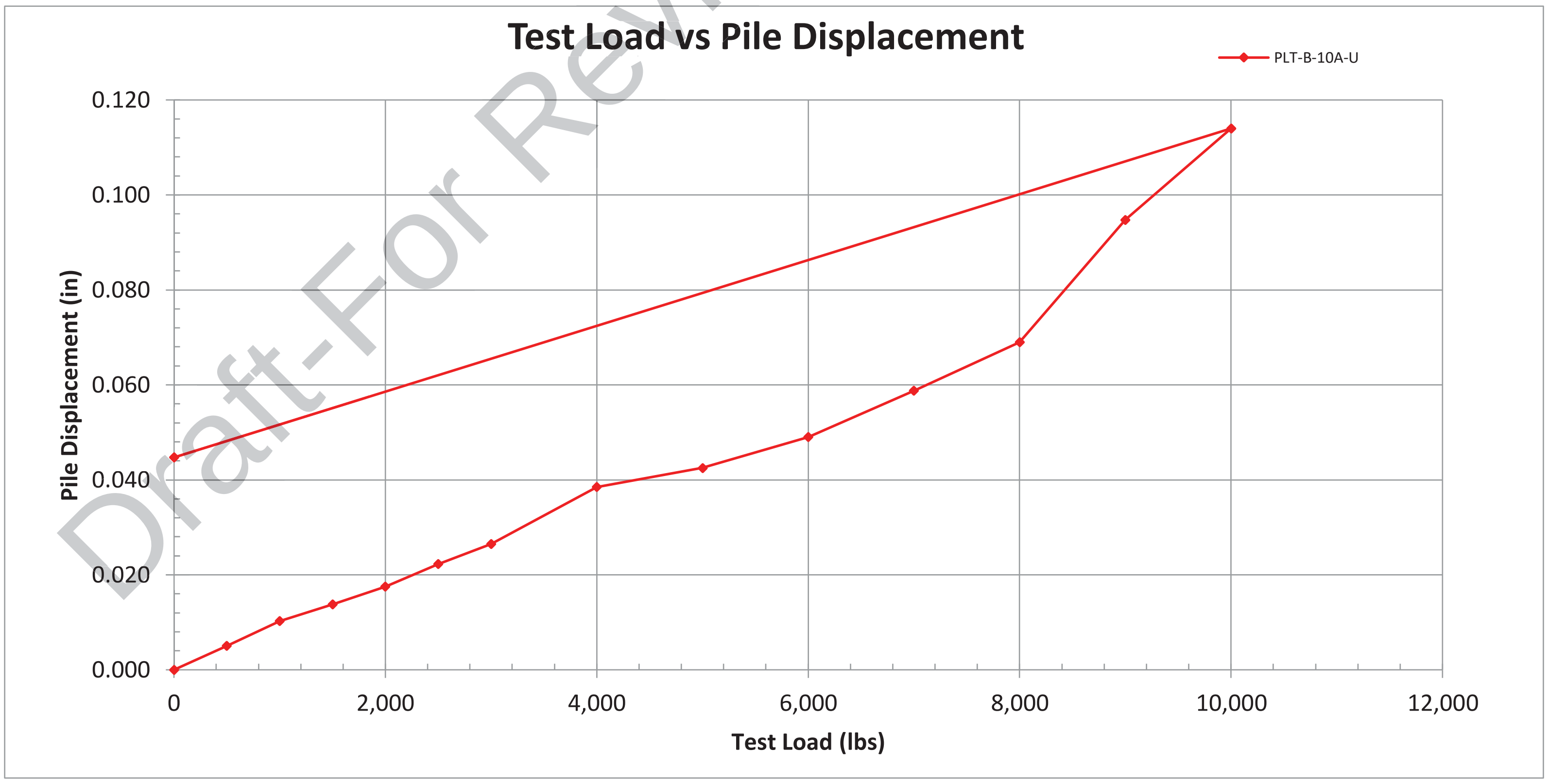


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-10A-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	79	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.005	-0.005	0.005	0.005	0.005	
1,000	1 min	-0.011	-0.010	0.011	0.010	0.010	
1,500	1 min	-0.015	-0.013	0.015	0.013	0.014	
2,000	1 min	-0.019	-0.017	0.019	0.017	0.018	
2,500	1 min	-0.024	-0.021	0.024	0.021	0.022	
3,000	1 min	-0.029	-0.024	0.029	0.024	0.027	
4,000	1 min	-0.043	-0.034	0.043	0.034	0.039	
5,000	1 min	-0.048	-0.038	0.048	0.038	0.043	
6,000	1 min	-0.056	-0.042	0.056	0.042	0.049	
7,000	1 min	-0.067	-0.051	0.067	0.051	0.059	
8,000	1 min	-0.079	-0.060	0.079	0.060	0.069	
9,000	1 min	-0.111	-0.079	0.111	0.079	0.095	
10,000	1 min	-0.131	-0.098	0.131	0.098	0.114	
0	1 min	-0.047	-0.043	0.047	0.043	0.045	

Notes:



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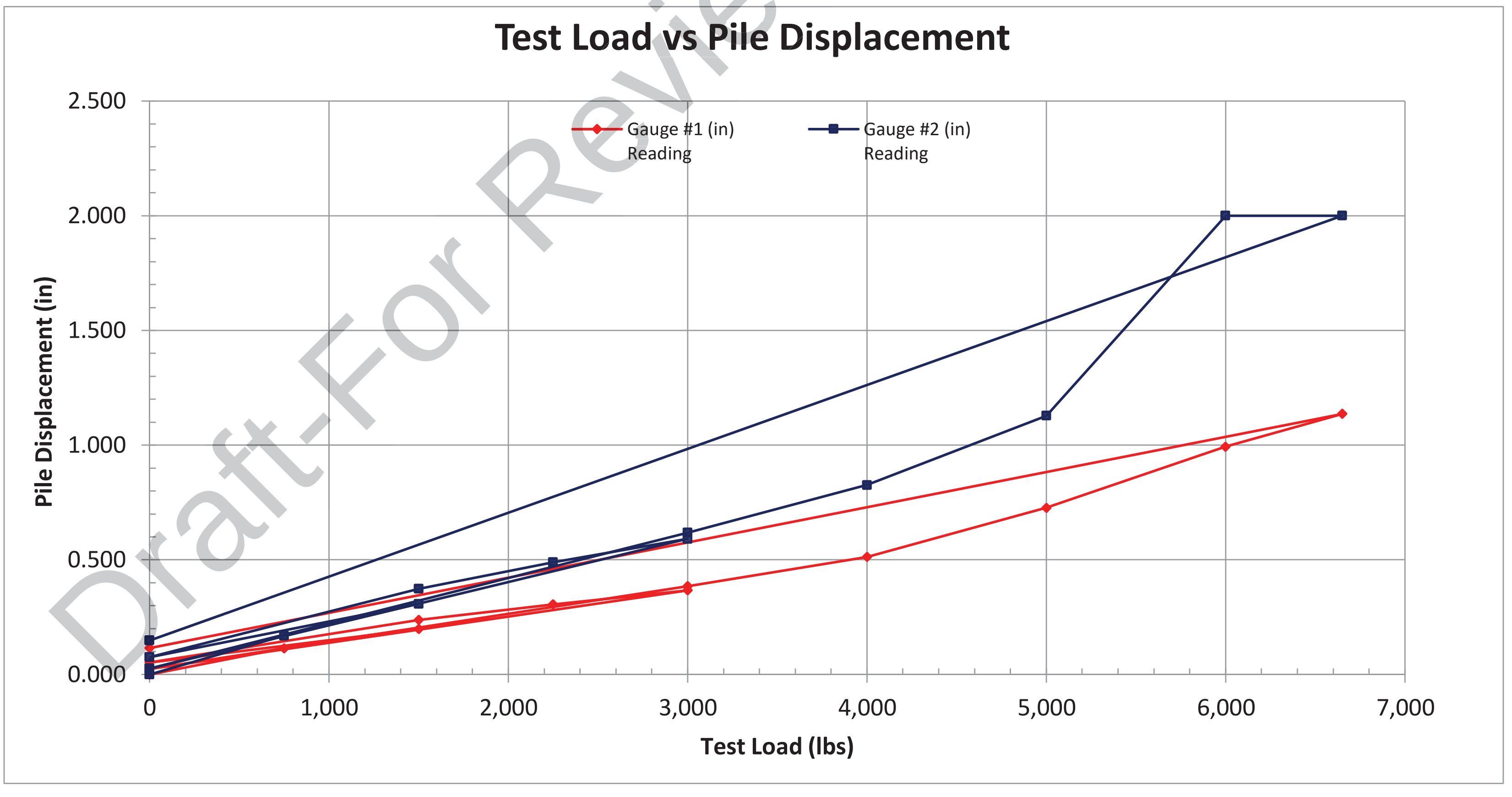
Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-10A-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	79	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.115	-0.170	0.115	0.170	
1,500	1	-0.198	-0.309	0.198	0.309	
0	0	-0.053	-0.076	0.053	0.076	
1,500	1	-0.238	-0.374	0.238	0.374	
2,250	1	-0.306	-0.489	0.306	0.489	
3,000	1	-0.367	-0.591	0.367	0.591	
0	0	-0.025	-0.026	0.025	0.026	
3,000	1	-0.385	-0.619	0.385	0.619	
4,000	1	-0.513	-0.826	0.513	0.826	
5,000	1	-0.727	-1.128	0.727	1.128	
6,000	1	-0.994	-2.000	0.994	2.000	
6,650	1	-1.136	-2.000	1.136	2.000	Failure
0	0	-0.116	-0.149	0.116	0.149	

Notes:

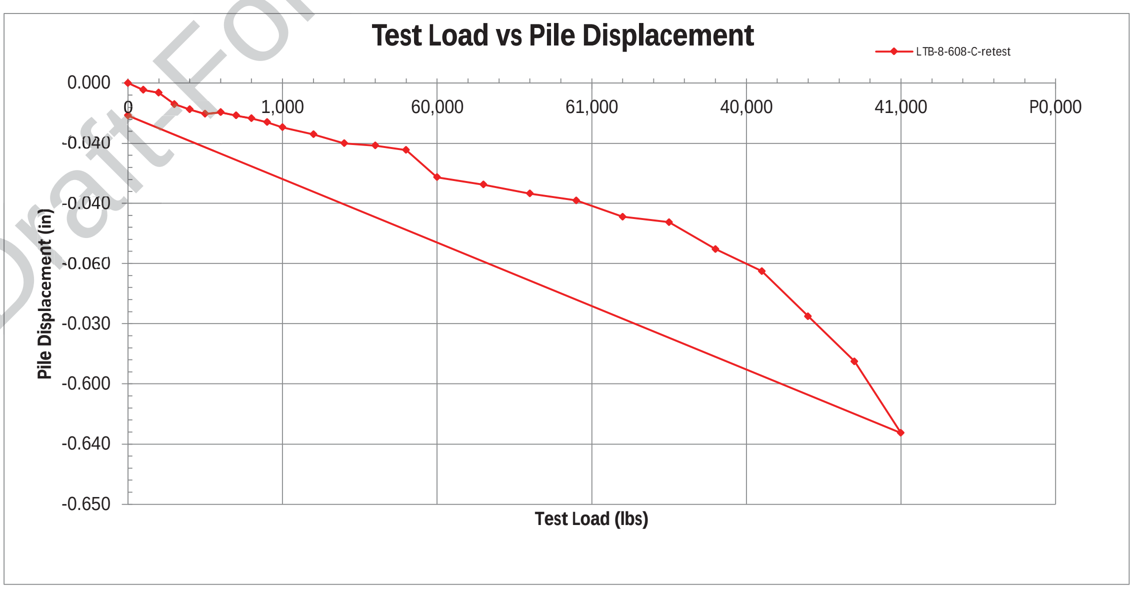


Pile Load Test - Compression

Project Name:	Corby BESS	Pile Install Date:	10/9/2023
Project No.:	GE2306031	Pile Test Date:	10/10/2023
Client:	NextEra Energy	Tested by:	dmb
Pile No.	PLT-B-10B-C	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	7.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	191.5	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	0.003	0.002	-0.003	-0.002	-0.002	
1,000	1 min	0.005	0.002	-0.005	-0.002	-0.003	
1,500	1 min	0.007	0.007	-0.007	-0.007	-0.007	
2,000	1 min	0.009	0.009	-0.009	-0.009	-0.009	
2,500	1 min	0.010	0.011	-0.010	-0.011	-0.010	
3,000	1 min	0.010	0.010	-0.010	-0.010	-0.010	
3,500	1 min	0.011	0.011	-0.011	-0.011	-0.011	
4,000	1 min	0.011	0.013	-0.011	-0.013	-0.012	
4,500	1 min	0.012	0.014	-0.012	-0.014	-0.013	
5,000	1 min	0.014	0.016	-0.014	-0.016	-0.015	
6,000	1 min	0.016	0.018	-0.016	-0.018	-0.017	
7,000	1 min	0.020	0.021	-0.020	-0.021	-0.020	
8,000	1 min	0.020	0.022	-0.020	-0.022	-0.021	
9,000	1 min	0.021	0.024	-0.021	-0.024	-0.022	
10,000	1 min	0.032	0.031	-0.032	-0.031	-0.031	
11,500	1 min	0.034	0.034	-0.034	-0.034	-0.034	
13,000	1 min	0.038	0.036	-0.038	-0.036	-0.037	
14,500	1 min	0.039	0.040	-0.039	-0.040	-0.039	
16,000	1 min	0.042	0.047	-0.042	-0.047	-0.045	
17,500	1 min	0.045	0.048	-0.045	-0.048	-0.046	
19,000	1 min	0.053	0.058	-0.053	-0.058	-0.055	
20,500	1 min	0.062	0.063	-0.062	-0.063	-0.063	
22,000	1 min	0.079	0.076	-0.079	-0.076	-0.078	
23,500	1 min	0.095	0.090	-0.095	-0.090	-0.093	
25,000	1 min	0.109	0.124	-0.109	-0.124	-0.116	
0	1 min	0.017	0.005	-0.017	-0.005	-0.011	

Notes:

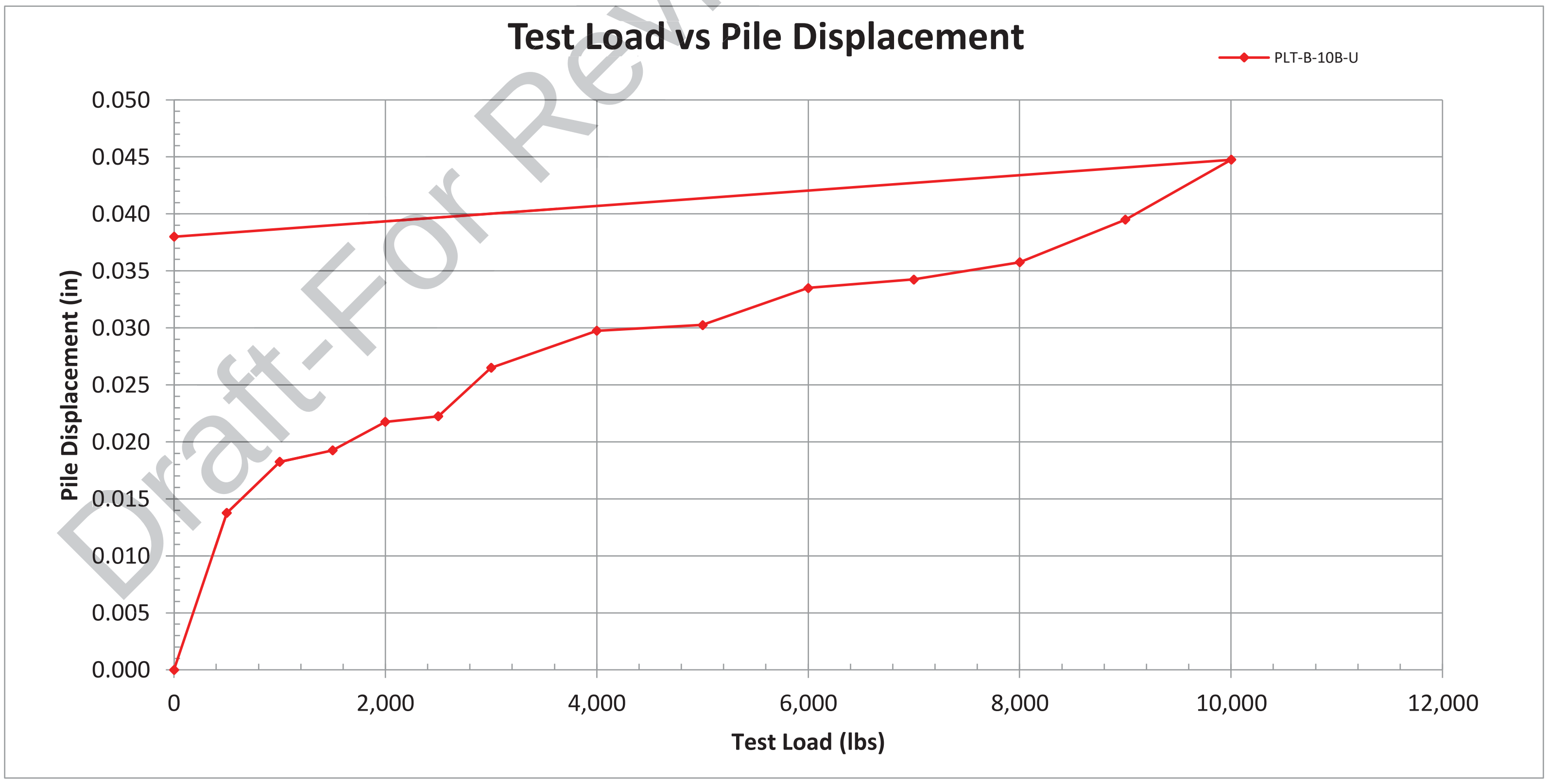


Pile Load Test - Uplift/Tension

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-10B-U	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 Ht above Ground (in):	6
Pile Drive Time (sec):	198.3	Gauge#2 Ht above Ground (in):	6

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Ave. Gauge Displacement (in)	Notes
		Gauge #1 Reading (in)	Gauge #2 Reading (in)	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement		
0		0.000	0.000	0.000	0.000	0.000	
500	1 min	-0.019	-0.009	0.019	0.009	0.014	
1,000	1 min	-0.025	-0.012	0.025	0.012	0.018	
1,500	1 min	-0.026	-0.013	0.026	0.013	0.019	
2,000	1 min	-0.031	-0.013	0.031	0.013	0.022	
2,500	1 min	-0.031	-0.014	0.031	0.014	0.022	
3,000	1 min	-0.037	-0.017	0.037	0.017	0.027	
4,000	1 min	-0.040	-0.020	0.040	0.020	0.030	
5,000	1 min	-0.040	-0.021	0.040	0.021	0.030	
6,000	1 min	-0.042	-0.025	0.042	0.025	0.034	
7,000	1 min	-0.043	-0.026	0.043	0.026	0.034	
8,000	1 min	-0.045	-0.027	0.045	0.027	0.036	
9,000	1 min	-0.047	-0.033	0.047	0.033	0.040	
10,000	1 min	-0.050	-0.040	0.050	0.040	0.045	
0	1 min	-0.050	-0.027	0.050	0.027	0.038	

Notes:



Pile Load Test - Lateral

Project Name:	Corby BESS	Pile Install Date:	10/3/2023
Project No.:	GE2306031	Pile Test Date:	10/4/2023
Client:	NextEra Energy	Tested by:	Dmb
Pile No.	PLT-B-10B-L	Weather:	Sunny
Pile Type:	W6x9	Pile Embedment Depth (ft):	10.0
Pile Stickup Ht (ft):	3.0	Gauge#1 and #2 Ht above Ground (in):	3 and 24
Pile Drive Time (sec):	198.3	Load application above Ground (in):	24

Load (lbs)	Hold Time (min)	Dial Gauge Reading		Dial Gauge Displacement		Note
		Gauge #1 (in) Reading	Gauge #2 (in) Reading	Gauge #1 (in) Displacement	Gauge #2 (in) Displacement	
0	0	0.000	0.000	0.000	0.000	
750	1	-0.111	-0.174	0.111	0.174	
1,500	1	-0.172	-0.284	0.172	0.284	
0	0	-0.018	-0.020	0.018	0.020	
1,500	1	-0.178	-0.296	0.178	0.296	
2,250	1	-0.246	-0.414	0.246	0.414	
3,000	1	-0.301	-0.522	0.301	0.522	
0	0	-0.021	-0.029	0.021	0.029	
3,000	1	-0.319	-0.546	0.319	0.546	
4,000	1	-0.431	-0.738	0.431	0.738	
5,000	1	-0.557	-0.940	0.557	0.940	
6,000	1	-0.768	-1.274	0.768	1.274	
7,000	1	-0.843	-1.397	0.843	1.397	
8,100	1	-1.038	-2.000	1.038	2.000	Failure
0	0	-0.113	-0.148	0.113	0.148	

Notes: