EXHIBIT H
GEOLOGY AND SEISMICITY

OAR 345-021-0010(1)(h)

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ATTACHMENT

H.1 INTRODUCTION

OAR 345-021-0010(1)(h) *Information from reasonably available sources regarding the geological and soil stability within the analysis area, providing evidence to support findings by the Council as required by OAR 345-022-0020, including:*

Response:

Archway Solar Energy LLC (Applicant) proposes to construct the Archway Solar Energy Facility (Facility) in Lake County, Oregon, with generating capacity of up to 400 megawatts (MW). The Facility may also contain a battery energy component with storage capacity of up to 400 MW and discharge capacity of up to 1,600 megawatt-hours. This Exhibit presents an analysis of the Facility geology and seismicity, as required by OAR 345-021-0010(1)(h).

Section H.2 defines the analysis area of the Facility. Sections H.3 through H.9 provide information from reasonably available sources regarding the geological and soil stability within the analysis area. Section H.10 provides a summary of Exhibit H findings.

H.2 ANALYSIS AREA

The analysis area for structural standards (Exhibit H) is the 4,470-acre area within the site boundary. "Site boundary" as defined in OAR 345-001-0010(55) means "*the perimeter of the site of a proposed energy facility, its related or supporting facilities, all temporary laydown and staging areas, and all corridors and micrositing corridors proposed by the applicant*." In this Exhibit, the Applicant equates the term "site boundary" with the analysis area.

H.3 GEOLOGIC REPORT AND SUMMARY OF CONSULTATION WITH OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES

OAR 345-021-0010(1)(h)(A) *A geologic report meeting the Oregon State Board of Geologist Examiners geologic report guidelines. Current guidelines must be determined based on consultation with the Oregon Department of Geology and Mineral Industries, as described in paragraph (B) of this subsection;*

OAR 345-021-0010(1)(h)(B) *A summary of consultation with the Oregon Department of Geology and Mineral Industries regarding the appropriate methodology and scope of the seismic hazards and geology and soil-related hazards assessments, and the appropriate site-specific geotechnical work that must be performed before submitting the application for the Department to determine that the application is complete;*

Response: Topographic and geologic conditions/hazards within the Facility site boundary were evaluated by reviewing available reference materials (such as topographic maps, geologic maps, and aerial photographs), including the *Preliminary Geotechnical Engineering Report* (Terracon 2020) provided as Attachment H-1. The following sections describe the review findings. During the final analysis, a field reconnaissance will be conducted by a Jacobs geologist and geotechnical engineer. Additional subsurface explorations, testing, and engineering analysis may be necessary prior to design and construction; this will be determined during final analysis.

Before the final submittal of this Application for Site Certificate, a Jacobs geotechnical engineer will contact the Oregon Department of Geology and Mineral Industries (DOGAMI) and Oregon Department of Energy (ODOE). Once a formal meeting is held, the discussion will focus on site characteristics, DOGAMI points of concern, perceived site hazards, and Facility resiliency. Jacobs will update the record when discussion with DOGAMI (and ODOE, if necessary) is complete.

H.3.1 Topographic Setting

The Facility is located in the Fort Rock-Christmas Lake Valley Basin in south-central Oregon. The site is approximately 10 miles east of Christmas Valley, Oregon, and 75 miles southeast of Bend, Oregon. The Fort Rock-Christmas Lake Valley is a paleo lake basin that historically contained an inland sea up to 200 feet in maximum depth. The basin is approximately 25 miles wide and 40 miles long.

The Christmas Valley Highway borders the northern part of the site and Fandango Canyon is approximately 4.5 miles to the west. The site is flat, generally grading to the north with less than a 1 percent slope. Thus, drainage is in a generally northward direction. Elevations within the Facility site boundary range from approximately 4,300 feet to 4,400 feet above mean sea level.

H.3.2 Regional Geologic Setting

The Fort Rock-Christmas Lake Valley Basin is located within the Great Basin section of the Basin and Range physiographic province. The area is characterized by basins that have closed or partially closed drainage systems and are separated by north- to south-trending fault-block ranges or escarpments.

The Great Basin is typically defined as having the following geological traits: (1) it is a region of drainage basins that have no outlet to a sea or ocean; and (2) it is a region of north to south oriented mountain ranges, separated by flat valleys or basins.

A geologic map of the Facility site vicinity, adapted using geographic information systems (GIS) and DOGAMI resources (Franczyk et al. 2021) is presented in Figure H-1.

H.3.3 Site Geologic Setting

The following descriptions of the geologic units found in the area are summarized from Franczyk et al. (2021) and Diggles et al. (1990) and from the Facility-specific preliminary geotechnical work performed by Terracon and included as Attachment H-1 (Terracon 2020).

H.3.3.1 Surficial Geologic Units

As described by Terracon (2020), the majority of the Facility site is mapped as Quaternary sediments (Qs), known as diatomaceous earth, which can be further defined as a soft, crumbly, porous sedimentary deposit formed from the fossil remains of diatoms (see Attachment H-1). Diatoms are a type of algae found at the bottom of a body of water, which is consistent for an area historically covered by an inland sea. The Terracon report further states that diatomaceous soil is often characterized by high water contents, low unit weights, and susceptibility to crumbling.

The geologic map also indicates that minor portions of the southern and western boundaries of the Facility site include Tertiary basalt (Tb) and a southeastern portion of the site as Quaternary alluvium and surficial aeolian deposits (Qal). Based on Terracon’s site explorations, the mapped Qal includes wind-blown sand that overlies the Quaternary sediments described above. Based on their Facility-specific explorations, Terracon believes the soils are generally consistent with these descriptions

H.3.3.2 Bedrock Geologic Units

The geologic map also indicates that minor portions of the southern and western boundaries of the Facility site are underlain by Tertiary basalt flows (Tb). This basalt is described as primarily plagioclase-olivine basalt with minor interbeds of tuff.

H.3.3.3 Structural Geology

As previously noted, the site located within the northern Basin and Range province, which is characterized by sometimes active, north-trending normal faults. Potentially active faults in the vicinity (within a 50-mile radius) include the Abert Rim fault, two fault sections of the Winter Rim fault system, Paulina Marsh faults, and Southeast Newberry fault zone. None of these faults are mapped within the proposed site boundary (Figure H-2), although the Southeast Newberry fault zone is less than 5 miles away. Section H.6.2 and Table H-2 describe the potentially active faults.

H.3.3.4 Groundwater/Springs

Groundwater in the vicinity is used primarily for irrigation and stock watering. Based on a well log search (OWRD 2022), several wells have been drilled adjacent to the west side of the site to provide irrigation water. These are large wells drilled to several hundred feet deep that are screened at deep intervals; thus these do not reflect the shallowest aquifer. Post-completion water levels are as shallow as 15 feet below ground surface (bgs) (indicating some artesian conditions in the deep wells). The depth to “first water” is more than 50 feet bgs.

As described by Terracon (2020), although no groundwater was encountered during their subsurface testing (which only extended as much as 4 to 6 feet bgs), the Christmas Valley area is known for seasonal, shallow ponds, and lakes. Therefore, areas of ponding should be anticipated during wet-weather conditions. Groundwater level fluctuations are likely occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the Terracon borings were performed.

H.4 SITE-SPECIFIC GEOTECHNICAL WORK

OAR 345-021-0010(1)(h)(C) *A description and schedule of site-specific geotechnical work that will be performed before construction for inclusion in the site certificate as conditions.*

Response:

H.4.1 Geotechnical Review

Existing published information was reviewed and used to characterize the current geologic conditions and potential seismic hazards in the vicinity of the Facility site. These materials included local, state, and federal government aerial photography, site photographs, published geologic maps, and the Facility-specific preliminary geotechnical report included as Attachment H-1 (Terracon 2020). This preliminary geotechnical report was completed for the Facility site in 2020, and provides a summary of subsurface soil and groundwater conditions (based on drilling, cone penetrometer tests, and backhoe test pits), a basic review of the Facility’s geologic setting, and a summary of various soil laboratory testing. The report also provides preliminary design and construction recommendations, and a summary of axial and lateral foundation testing that was completed on test piles installed at the Facility.

For this Application for Site Certificate, a preliminary seismic hazard assessment was conducted to characterize seismicity in the vicinity of the Facility site and evaluate potential seismic impacts. This work was based on the potential for regional and local seismic activity as described in the existing scientific literature, and on subsurface soil and groundwater conditions within the Facility site boundary based on geotechnical subsurface investigations. The preliminary seismic hazard assessment included the following tasks:

1. Cursory review of literature and databases
2. Review of existing subsurface data obtained for the Facility site; these data were used to select a subsurface profile for seismic site class selection

3. Identification of the potential seismic events appropriate for the site and characterization of those events in terms of a series of design events

Before final analysis, a more detailed seismic characterization will be performed to facilitate preparation of conclusions and recommendations that include:

a) Specific seismic events that might have a significant effect on the area within the Facility site boundary

b) The potential for seismic energy amplification within the Facility site boundary

c) A site-specific acceleration response spectrum for the area within the Facility site boundary

d) Evaluation of the potential for earthquake-induced fault displacement, landslides, liquefaction, settlement, and subsidence. For this phase of the Application for Site Certificate, the anticipated potential for these risks has been described, but will be revisited and confirmed during the final analysis.

Josh Butler, P.E., and Greg Warren, P.G. (Jacobs) conducted work for this Exhibit. Mr. Butler and Mr. Warren have prepared numerous Energy Facility Siting Council and industrial siting applications for energy facilities throughout Oregon, Washington, Utah, Wyoming, California, and Colorado. In addition, they have conducted many geotechnical investigations and evaluations, and have prepared data and design reports for various energy facilities (including wind, solar, and geothermal projects).

H.4.2 Additional Geotechnical Work

At an appropriate stage in the development, additional geotechnical work must be completed to confirm the anticipated soil conditions and provide final design recommendations. The final design geotechnical investigation will consist primarily of the following tasks:

- Reviewing available data from previous geotechnical explorations in the vicinity of the Facility site. Database searches will be repeated to see if new references have become available. This will include searching Oregon Water Resources Department (OWRD) monitoring well records and the Oregon Department of Transportation’s (ODOT) database for highway construction in the vicinity.
- Reviewing available geologic information from published sources. Existing publications and references reviewed for this ASC are provided in Section H.11 (References); before additional geotechnical work begins, the databases provided by OWRD, ODOT, DOGAMI, and others will be queried to determine if new material is available. This will include DOGAMI’s open file report database and published state and regional reports on seismicity in the Facility vicinity. DOGAMI’s database will be visited again, along with contacting DOGAMI staff, to check for updates.
- Conducting additional geotechnical field exploration within the Facility site boundary, including soil borings, test pits, infiltration tests, and possibly geophysical testing.
- Collecting soil samples for classification and conducting laboratory tests on selected soil samples, likely to include moisture content, grain size analyses, index testing, in situ field testing of soil strength, moisture-density relationship of soils, and possibly rock durability and quality testing (if encountered)

Geotechnical analyses will be used to calculate bearing capacity of the soils, conduct stability analyses, evaluation of corrosion potential, and provide engineering recommendations for earthwork and construction of the Facility’s structures.

H.5 TRANSMISSION LINES AND PIPELINES

OAR 345-021-0010(1)(h)(D) *For all transmission lines, and for all pipelines that would carry explosive, flammable or hazardous materials, a description of locations along the proposed route where the applicant proposes to perform site specific geotechnical work, including but not limited to railroad crossings, major road crossings, river crossings, dead ends (for transmission lines), corners (for transmission lines), and portions of the proposed route where geologic reconnaissance and other site specific studies provide evidence of existing landslides, marginally stable slopes or potentially liquefiable soils that could be made unstable by the planned construction or experience impacts during the facility's operation.*

Response: Power generated by the Facility will be transmitted to the power grid via a 500-kilovolt overhead transmission line from the Facility substation to the point of interconnection. The Facility's transmission line will be approximately 4 miles long, depending on the routing to the Bonneville Power Administration line-tap location. The overhead transmission line will be on steel tangent H-frame pole-structures. The pole-structures will be approximately 110 feet in height and will be spaced approximately 1,000 feet apart, depending on the specific pole type chosen and site conditions.

Subsurface explorations consisting of test pit excavations and/or borings will occur at a representative sample of pole-structure locations along the proposed gen-tie transmission line route. The corresponding laboratory testing of these subsurface explorations will form the basis of geotechnical work in the gen-tie transmission line corridor. Given its placement and design, the gen-tie transmission line will not create new or exacerbate existing geologic hazards. In addition, the proposed transmission line will not cross over any major roadways, railroads, or rivers. The transmission line corridor occupies flat terrain with no slope stability hazards. Standard-of-practice geotechnical design efforts and ground improvement measures are anticipated to be sufficient to identify and mitigate potentially liquefiable soils within the transmission corridor.

H.6 SEISMIC HAZARD ASSESSMENT

OAR 345-021-0010(1)(h)(E) *An assessment of seismic hazards, in accordance with standard-of-practice methods and best practices, that addresses all issues relating to the consultation with the Oregon Department of Geology and Mineral Industries described in paragraph (B) of this subsection, and an explanation of how the applicant will design, engineer, construct, and operate the facility to avoid dangers to human safety and the environment from these seismic hazards. Furthermore, an explanation of how the applicant will design, engineer, construct and operate the facility to integrate disaster resilience design to ensure recovery of operations after major disasters. The applicant must include proposed design and engineering features, applicable construction codes, and any monitoring and emergency measures for seismic hazards, including tsunami safety measures if the site is located in the DOGAMI-defined tsunami evacuation zone; and*

H.6.1 Maximum Considered Earthquake Ground Motion

Response: The 2022 U.S. Geological Survey (USGS) National Seismic Hazard Mapping project (USGS 2022a) developed ground motion models using a probabilistic seismic hazard analysis that covered the area within the Facility site boundary. Though these models are not considered site-specific, they provide a reasonable estimate of the ground motions within the Facility site boundary. Based on the USGS uniform hazard model, the 500-year and 5,000-year earthquakes have bedrock peak ground accelerations of 0.12g and 0.38g, respectively, where "g" is the acceleration of gravity.

For new construction, the site should be designed for the maximum considered earthquake, according to the International Building Code (International Code Council 2018; referenced as IBC) as amended by the Oregon Structural Specialty Code (International Code Council and State of Oregon 2019; OSSC). The 2019 code was adopted effective October 1, 2019. The next code update is anticipated to be adopted in October 2022. This code adheres to the 2015 National Earthquake Hazards Reduction Program Seismic Design Provisions (Federal Emergency Management Agency 2015), and the latest USGS Seismic Hazard Maps (USGS 2022a). This maximum considered earthquake event (or MConE) has a 2 percent probability of exceedance in 50 years (or an approximately 2,475-year return period). For the Facility, this event has an estimated peak ground acceleration (PGA) of 0.22g at the bedrock surface based on the 2015 NEHRP Seismic Design Maps (Federal Emergency Management Agency 2015). This value of PGA on rock is an average representation of the acceleration for all potential seismic sources (crustal, intraplate, or subduction) mapped as active at the time of the study (USGS 2022a).

Seismic design parameters were developed in accordance with the IBC. Based on existing subsurface information (including a preliminary review of boring logs and geologic mapping), the Facility will be conservatively designed for Site Class D (S_D; stiff soil profile), according to IBC requirements. Once site-specific geotechnical subsurface information is collected, the actual site class determination may improve or worsen. Final site class determination cannot be made until further site exploration is performed. Table H-1 summarizes the current recommended seismic design parameters for the Maximum Considered Earthquake (MConE) event.

Table H-1. Seismic Design Parameters—Maximum Considered Earthquake

Site Class	Controlling Earthquake Magnitude	Peak Horizontal Ground Acceleration on Bedrock	Soil Amplification Factor, Fa	Peak Horizontal Ground Acceleration at Ground Surface
S_B (475-year return)	7.1	0.08g	1.40	0.11g
S_B (2,475-year return)	7.1	0.22g	1.40	0.31g

Notes: Earthquake magnitude in this table is a mean representation of all known seismic sources. The peak ground acceleration is assumed to be roughly 40 percent of the 0.2-second spectral acceleration, following the recommendations of the IBC.

Fa = sail amplification factor

g = acceleration from gravity

10 Percent Exceedance in 50 Years (475-Year Return Interval):

- Short period (0.2-second) spectral response acceleration at the ground surface, S_{MS} = 0.366g for Site Class S_D
- 1-second period spectral response acceleration at the ground surface, S_{M1} = 0.166g for Site Class S_D

2 Percent Exceedance in 50 Years (2,475-Year Return Interval):

- Short period (0.2-second) spectral response acceleration at the ground surface, S_{MS} = 0.695g for Site Class S_D
- 1-second period spectral response acceleration at the ground surface, S_{M1} = 0.453g for Site Class S_D

The design spectral response accelerations, S_{DS}, for both the short period and the 1-second period (S_{DS} and S_{p1}, respectively) are determined by multiplying the spectral response accelerations (S_{MS} and S_{M1}) by a factor of 2/3.

H.6.2 Earthquake Sources

Response: The potential seismic hazards in the vicinity of the Facility site result from three seismic sources: Cascadia Subduction Zone (CSZ) interplate events, CSZ intraslab events, and crustal events (Geomatrix 1995).

Two of the potential seismic sources, interplate and intraslab events, are related to the subduction of the Juan de Fuca plate beneath the North American plate. Interplate events are caused by the frictional interface between these two tectonic plates. Intraslab events, which originate within the subducting Juan de Fuca plate, are generally associated with normal faulting that results from bending stresses built up within the plate as it is subducted beneath the North American plate. The combination of these factors is often referred to as the CSZ source mechanism. The CSZ is located beneath western Oregon, Washington, and British Columbia. The two source mechanisms associated with the CSZ are currently thought to be capable of producing maximum earthquakes with moment magnitudes of approximately 9.0 and 7.2 for the interplate and intraslab events, respectively (Geomatrix 1995; USGS 2022a, 2022b).

Earthquakes caused by movements along crustal faults, generally in the upper 10 to 15 miles of the earth's crust, result in the third seismic source mechanism. In the vicinity of the Facility site, earthquakes occur within the crust of the North American tectonic plate when built-up stresses near the surface are released through fault rupture.

No potentially active faults are mapped within the Facility site boundary (Figure H-2). A number of late-Quaternary-age faults are mapped in the general vicinity of the Facility site, as shown in Figure H-2.

The **Abert Rim** fault is a north-northeast-trending, high-angle normal fault forms the eastern margin of the half-graben that confines the Lake Abert Basin. The fault has produced escarpments up to 2,600 feet high in Pliocene and Miocene volcanic rocks. The Abert Rim fault is divided into two sections, primarily based on recency of movement—the southern section, the Lake Abert section, most of which exhibits evidence of Holocene displacement. Fault-scarp profiles along this section show scarps are 12 to 16 feet high on Holocene debris flows and as much as 25 feet high on latest Pleistocene deposits. These fault scarps on post-pluvial lake deposits, steep scarp-slope angles, and the presence of a scarp free face in places along the fault support a Holocene age of most-recent movement on the Abert Rim fault.

The **Winter Rim fault system, which includes the Winter Rim and eSlide Mountain sections,** is a northwest trending, high-angle, down-to-the-east normal fault system that forms the western margin of a large graben that confines the Chewaucan-Summer Lake Basin. The fault is marked by prominent escarpments (Winter Rim) in Miocene volcanic and volcaniclastic sedimentary rocks. The Winter Rim fault system is divided into three sections, and all sections show evidence of latest Quaternary displacement. This fault exhibits intermittent 20- to 25-foot-high fault scarps on latest Pleistocene pluvial lake deposits and younger (Holocene) deposits. Fault-trenching investigations revealed at least two units of post-lacustrine colluvium that suggest multiple late Quaternary events. An alluvial-fan deposit that buried the fault scarp at Kelley Creek yielded a radiocarbon age on charcoal of 2,130±90 years Before Present. Fresh fault scarps that offset latest Pleistocene pluvial shorelines and deposits support a Holocene age of most-recent movement on the Slide Mountain section of the Winter Rim fault system.

The **Southeast Newberry fault zone** is a northwest-trending fault zoned is a group of relatively short, mostly normal faults that form small escarpments and fault scarps on Plio-Pleistocene volcanic rocks and Pleistocene and Holocene sediments on the floor of Fort Rock Valley. The most-recent events on at least two faults in the zone, the Viewpoint and Crack-In-The-Ground faults, occurred in the Holocene. Individual faults in the Southeast Newberry fault zone form small escarpments and fault scarps on Plio-Pleistocene volcanic rocks and late Quaternary alluvial and lacustrine deposits on the floor of Fort Rock-Christmas Lake Valley Basin.

The **Paulina Marsh** faults are northwest-trending faults are located in and along the margins of Paulina Marsh, a large wetland occupying an internally drained basin in the southwestern corner of the Fort Rock Valley, that is underlain by Pleistocene and Holocene alluvial and lacustrine deposits. Most faults in the zone offset Miocene to Pliocene volcanic rocks in uplands around the marsh, but the Paulina Marsh fault is marked on the floor of the marsh by a less than 2-meter-high, down-to-the-southwest fault scarp on deposits that may contain Holocene Mazama ash.

Table H-2 summarizes information about local potentially seismic faults.

Table H-2. Summary of Potentially Active Faults

Fault	Distance to Facility (miles)a	Fault Length	Most Recent Movement (years before present)	Slip-Rate Category
Abert Rim Fault, Lake Abert Section	35	77 km	<15,000	Between 0.2 and 1.0 mm/yr
Winter Rim Fault Slide Mountain Section	36	33 km	<15,000	0.4–0.6 mm/yr
Winter Rim Fault Winter Rim Section	20	26 km	latest Quaternary (<15 ka)	Between 0.2 and 1.0 mm/yr
Paulina Marsh Faults	35	31 km	latest Quaternary (<15 ka)	Between 0.2 and 1.0 mm/yr
Southeast Newberry Fault Zone	3	58 km	<15,000	0.1–0.5 mm/yr[a]

[a] Closest mapped distance to Facility.

Notes:

km = kilometer(s)

mm/yr = millimeter(s) per year

The PGA within the Facility site boundary resulting from a seismic event on one of these source mechanisms was estimated using information the USGS developed in its seismic hazard mapping database (USGS 2022a). This information includes estimated PGA at a theoretical soft rock/stiff soil interface for different probabilities of exceedance. The USGS database also provides the seismic deaggregation information for the seismic hazard, including estimates of the mean earthquake moment magnitude and mean epicentral distance associated with a given probability of exceedance at a given location.

The Maximum Probable Earthquake (MPE) is considered to be an earthquake that has a 10 percent probability of exceedance in 50 years (a nominal 475-year recurrence interval). The MConE is considered to be an earthquake with a nominal 2,475-year recurrence interval (a 2 percent probability of exceedance in 50 years). Figures H-3 and H-4 show the probabilistic seismic hazard deaggregation for the MPE and MConE events, respectively.

The Maximum Credible Earthquake (MCE), is the maximum event that each source is believed to be capable of producing. To provide an estimate of the MCE events from each principal source mechanism, the maximum moment magnitude for each fault was estimated using the relationship developed by Wells and Coppersmith (1994), which relates magnitude to fault length (USGS 2022a) and distance from the Facility site boundary. The USGS also provides a range of magnitude in their database for some fault sources (USGS 2014). These analysis parameters were summarized for the potentially active faults near the Facility (shown in Table H-2). In addition to these estimated magnitudes for crustal faults, Table H-3 summarizes the magnitudes for the random, unnamed crustal event from the USGS gridded hazard and from the CSZ intraslab and interplate events.

Table H-3. Maximum Considered Earthquake Source Characterization Parameters

Earthquake Source	Maximum Moment Magnitude	Epicentral Distance (miles [km])
Random Hazard (Shallow Gridded WUS)	6.0	7 [11]
Crustal	6.7 to 7.3	10 to 34 [16 to 54]
Intraslab	7.2	>150 [>250]
Interplate	9.0 to 9.2	>100 [>160]

Notes:

The magnitudes for all crustal events are determined from the fault length/distance by Wells and Coppersmith (1994).

WUS = Western United States gridded (random) crustal source

H.6.3 Recorded Earthquakes

Response: Figure H-2 displays the location, approximate magnitude, and year of recorded earthquakes within 50 miles of the Facility site boundary. These historical seismic events have been grouped by magnitude, and are displayed using different-sized icons based on the strength of the event. Additional database searches will be performed during the final analysis of historical earthquakes.

Figure H-2 provides a summary of all recorded earthquakes known to have caused Modified Mercalli Intensity (MMI) III shaking intensity or greater within the Facility site boundary, regardless of epicentral distance from the Facility site boundary. For reference, an intensity of MMI III is associated with shaking that is “noticeable indoors, but may not be recognized as an earthquake.” An intensity of MMI V is “felt by nearly everyone; many awakened” (USGS 2022a). Based on preliminary analysis, the largest recorded earthquakes within 50 miles (80 km) of the Facility site boundary was a magnitude 4-4.9 events that occurred in 1959, approximately 50 miles (80 km) northeast of the Facility site boundary. Before the final submittal of this Application for Site Certificate, additional evaluation of historical earthquakes will be performed.

H.6.4 Median Ground Response Spectrum

Response: Prior to the final analysis, a spectral response evaluation will be completed to compare design spectral accelerations with spectral accelerations modeled for the known earthquake sources in Table H-3.

H.6.5 Seismic Hazards Expected to Result from Seismic Events

Response: For facilities designed to the current IBC and OSSC guidelines for Site Class B, the design seismic event will have a 2 percent chance of exceedance in the next 50 years (or an event with an approximate 2,475-year recurrence interval). For this event, the Facility will be designed for no life-threatening structural damage from either the vibrational response of the structure or from secondary hazards associated with ground movement or failure (such as landslides, lateral spreading, liquefaction, fault displacement, or subsidence). It is generally assumed that if significant structural damage can be prevented, the risk to human safety will be minimal.

Seismic hazards associated with a design seismic event could potentially include ground shaking and instability from landslides or subsurface movement. Impacts on the Facility from these hazards are anticipated to be low, as discussed below.

Potential for Fault Displacements. The probability of a fault displacement within the Facility site boundary is considered to be nonexistent because of the absence of known or mapped potentially active faults in the immediate area and, particularly, within the Facility site boundary. Unknown faults could exist, or new fault ruptures could form during a significant seismic event, but the likelihood of either occurrence is low based on the lack of active faults identified during previous geologic investigations.

A search for recently available LiDAR imagery was conducted in order to identify features that could indicate the potential for previously-unmapped faults (e.g. linear fault scarps). No LiDAR coverage is available for the project site (https://gis.dogami.oregon.gov/maps/lidarviewer/)

Potential for Ground Shaking. Ground shaking is expected within the Facility site boundary given the seismic setting. However, the probability of damage to structures from ground shaking is considered to be low because the seismic hazard potential is relatively low and, based on preliminary information, the area within the Facility site boundary is likely classified as Site Class D (International Code Council 2018). Facility components will be designed for the seismic potential of the area. Little or no structural damage is anticipated from MMI III intensity shaking, which is the predominant level of ground shaking anticipated within the Facility site boundary based on the historical record. Higher intensity shaking (MMI IV or MMI V) is not anticipated to cause significant damage to the Facility components. For comparison, MMI VII shaking is considered to result in “negligible damage in buildings of good design and construction.” The final analysis for ground shaking will provide a thorough evaluation of anticipated ground shaking at the Facility, based on historical events. However based on experience from nearby projects, and evaluation of the historical record from 1700 to present, no earthquakes within the Facility site boundary are anticipated to have resulted in MMI VII intensity shaking.

Liquefaction Potential. Liquefaction potential has not been completely characterized at all Facility locations. The geotechnical exploration performed by Terracon in 2020 (Terracon 2020) was limited to the upper 20 feet of the subsurface and does not provide comprehensive information on soil strength and groundwater conditions for the Facility. Zones of loose sand and medium stiff silt that were encountered may will be susceptible to liquefaction during loading from seismic events if they are near the groundwater table. The overall liquefaction potential of individual sites will depend on the thickness and specific characteristics of these deposits, including their density/consistency, fine-grained component, and plasticity. Further evaluation of liquefaction potential is necessary.

Behavior of Subsurface Materials. Risk of landslides or seismically induced landslides within the Facility site boundary is anticipated to be low because of the flat terrain of the site and lack of shallow groundwater. Slopes within the Facility site boundary are generally less than 1 percent. No landslides have been mapped or identified within the Facility site boundary.

Tsunami and Sech Hazard. The Facility is located inland and is not proximal to the ocean or any large surface waterbody. Therefore the hazard potential for tsunami or sech at the Facility is nonexistent.

Conclusion. Because of the potential for seismic-induced hazards within the Facility site boundary, mitigation measures to address these hazards in the siting, design, and construction of the Facility are necessary in order to protect against ground shaking and instability. The design of the Facility components will need to accommodate the level of seismic energy described in Section H.6.4, Median Ground Response Spectrum.

H.7 NONSEISMIC HAZARD ASSESSMENT

OAR 345-021-0010(1)(h)(F) *An assessment of geology and soil-related hazards which could, in the absence of a seismic event, adversely affect or be aggravated by the construction or operation of the facility, in accordance with standard-of-practice methods and best practices, that address all issues relating to the consultation with the Oregon Department of Geology and Mineral Industries described in paragraph (B) of this subsection. An explanation of how the applicant will design, engineer, construct and operate the facility to adequately avoid dangers to human safety and the environment presented by these hazards, as well as…:*

Response: Nonseismic geologic hazards in the Fort Rock-Christmas Lake Valley Basin could potentially include volcanic eruptions, collapsing soils, wind and water erosion potential of soils, and collapsing/shrinking-swelling/frost-heaving soils. The area within the Facility site boundary consists of flat-lying, unconsolidated sediments. The solar array, roads, and transmission line will be constructed on extremely flat terrain, without slopes or drainages that could potentially be subject to landslides and soil creep. A discussion of potential geologic hazards is presented below.

H.7.1 Landslides

DOGAMI released a publication series called Statewide Landslide Information Database for Oregon (SLIDO), which is a compilation of landslides in Oregon that have been identified on published maps (DOGAMI 2020). The database contains only landslides that have been located on these maps. Many landslides have not yet been located or are not on these maps and therefore are not in this database.

The primary purpose of SLIDO is to provide the best currently available mapping of landslide features throughout Oregon. This database serves as useful tool for differentiating broad areas of higher and lower hazards and as a starting point for more detailed study. This spatial information is basic to emergency management and land-use applications, including:

- Identify vulnerable areas that may require planning considerations
- Estimate potential losses from specific hazard events (before or after a disaster hits)
- Decide how to allocate resources for most effective and efficient response and recovery
- Prioritize mitigation measures that need to be implemented to reduce future losses

Release SLIDO-4.2 supersedes all previous releases of SLIDO (DOGAMI 2020). SLIDO-4.2 contains extensive updates to the landslide inventory dataset (deposits, scarp flanks, and scarps) and historic landslide points. These updates reflect the most recent published studies as well as contributions from other Oregon government agencies.

The only existing landslide mapped in the vicinity of the facility is located in the steep-sided plateau more than 2 miles east of the facility boundary (DOGAMI 2020).

H.7.2 Volcanic Eruptions

The Pacific Northwest region is home to a large number of active volcanoes along the Cascade Mountain Range. Table H-4 summarizes the closest volcanoes to the Facility with distances from each mountain to the Facility site boundary, and also the basic characteristics of the volcano:

Table H-4. Summary of Potentially Dangerous Volcanoes

Volcano	Distance to Facility Site Boundary	Volcano Type[a]	Composition[(1)]	Most Recent Eruption (Years Before Present):	Threat Potential[b]:

Medicine Lake volcano	120 mi	Composite/shield	basalt to rhyolite	950	High
Three Sisters	90 mi	Stratovolcano	Andesite to Rhyolite	2,000	Very High
Newberry Volcano	50 mi	broad shield	Basalt to Rhyolite	1,300	Very High
Crater Lake	83 mi	Collapse caldera/stratovolcano	andesite, dacite	6,600	Very High
Notes: [a] Volcano type and Composition are important because high-viscosity, stratovolcanoes (e.g. rhyolite composition) tend to erupt explosively and produce large ash/tephra whereas low-viscosity shield volcanoes (e.g., basalt composition) are more prone to producing lava flows [b] Overall volcanic threat potential according to USGS (2022c). Note that this does not directly apply to the Lakeview area.					

Because of the Facility's large distance from potentially eruptive volcanoes, no direct impacts (such as mudflows, lava flows, flooding) are expected. These are typically confined to the immediate vicinity of the volcanoes. Thus, impacts on a solar Facility from volcanic eruptions would be indirect and would most likely consist of ash/tephra fall.

Relatively small ash/tephra particles can rise more than 30,000 feet, and be carried downwind and blanket areas for distances of tens to hundreds of miles. Tephra plumes can create tens of minutes to hours of darkness as they pass overhead, and tephra fall can reduce visibility. In addition, deposits of tephra can short-circuit or break electric transformers and power lines, especially if the tephra is wet, as well as cause roofs of buildings to collapse. In several historical examples, accumulation of more than 10 centimeters (4 inches) of wet tephra caused roofs to collapse. Tephra can clog filters and increase wear on vehicle engines. Tephra clouds also commonly generate lightning that can interfere with electrical and communication systems and start fires.

The USGS Volcano Hazards program (USGS 2022c) monitors and studies active and potentially active volcanoes to assess their hazards, and conducts research on how volcanoes work in order for the USGS to issue "timely warnings" of potential volcanic hazards to emergency-management professionals and the public. Thus, in addition to collecting and interpreting the best possible scientific information, the program works to effectively communicate its scientific findings and volcanic activity alerts to authorities and the public.

Volcano updates include both a Volcano Alert Level and an Aviation Color Code. In most cases, the alert level and aviation-specific color code will move together (e.g., Normal and Green; Advisory and Yellow; Watch and Orange; Warning and Red).

As of June 2022, Cascade Range and California volcanoes were at a "normal" [Green] alert levels.

Because of the distance to potentially active volcanoes, no direct or indirect impacts of volcanic activity are expected to occur within the Facility site boundary, due to the distance to the volcanoes. Impacts are usually restricted to within 50 miles of the erupting volcano. However, depending on the prevailing wind direction at the time of a volcanic eruption and the source of the eruption; ash fallout in the region surrounding the Facility may occur.

H.7.3 Soil Erosion Potential

The soils within the Facility site boundary could be subject to wind and water erosion, particularly when the vegetation is removed. *The predominant site soils are the* Flagstaff complex, 0 to 1 percent slopes, and the Flagstaff-Playas complex, 0 to 1 percent slopes. These are ashy very fine sandy loam surface formed in Lacustrine deposits derived from volcanic ash and are saline. Frequency of flooding is rated “None” and Frequency of ponding is rated “Occasional”

Despite the nearly flat slopes, most of the site soils are categorized as **high** water erosion hazard (K factor of 0.55 to 0.64) because of the silty ash parent material and lack of soil cohesion.

Wind Erodibility Groups (WEGs) consist of soils that have similar properties (primarily textural classes) that affect their resistance to soil blowing if cultivated or disturbed. The groups are used to predict the susceptibility of soil to blowing and the amount of soil lost as a result of blowing. The predominant site soils are assigned to a WEG of 1 to 2, which means these soils are expected to have high wind erosion potential.

Overall, soil data indicate that the potential for wind and water erosion within the Facility site boundary is generally high. Because of steady, relatively high wind speeds, and brief but intense rainfall events, areas of vegetation removal could potentially expose soils to accelerated water and wind erosion during construction until they are stabilized.

Excavations for roads or other Facility structures could also temporarily expose the excavated spoils to wind and water erosion during construction. Mitigation measures to account for the high wind and water erosion are described in Section H.9.

H.7.4 Frost Action; Shrink-Swell; Corrosion

The site soils are rated as LOW for frost action, LOW for shrink-swell, and HIGH for corrosion.

H.7.5 Collapsing Soils/Piping

Silty soils with little or no plasticity can be subject to collapsing or piping when they are wetted. Loess in the vicinity of the Facility site is typically silty in composition, and therefore it could be subject to piping or collapse. Piping can have a detrimental effect on embankments or foundations constructed on loess.

Diatomaceous soils appear to be prevalent at the Facility, based on observations in the site geotechnical report (Terracon 2020). These soils are susceptible to collapse when wetted or loaded, which can result in large strains. Examples of loading could be Facility construction or seismic events. The magnitude of collapse or settlement can be estimated based on the magnitude of anticipated load, and layer thickness. Evaluation of collapse of diatomaceous soils must be completed during final design of the Facility and mitigated if necessary.

H.7.6 Suitability for Solar Arrays

The site soils are rated as very limited for ballast anchor systems for soil arrays, because of ponding and depth to saturated zone (a clayey subsurface layer may cause ponding).

The site soils are rated as very limited for Soil-penetrating anchoring systems because of ponding and depth to saturated zone (a clayey subsurface layer may cause ponding).

H.7.7 Future Climate Conditions

Either greater-intensity rainfall events or an overall reduction in annual precipitation coupled with warmer average annual temperatures could result in some negligible increase in the potential for geologic hazards. Specifically, increased deviation from climatic norms (including either wetter or drier conditions) could impact erosion. Warmer and drier periods can increase fire hazards in forested areas, which could lead to increased erosion and debris flows in steep drainages adjacent to the Facility. Dust during periods of dry weather and high wind can also result in deposition of windborne sediments at the Facility. Wetter periods with higher than normal precipitation can increase flooding hazards in the drainages.

H.7.8 Adverse Effects from Groundwater or Surface Water

The Facility site lies in a flat basin, with groundwater estimated to be at least 50 feet bgs. In the extremely rare event of a large seismic shift, the basin could potentially subside which could alter shallow groundwater depth. The probability of this is considered exceedingly rare, however. No adverse effects from groundwater or surface water specifically related to seismicity are expected to impact facility design and construction.

The 100-year floodplain boundaries are described in Exhibit XXX of this Application for Site Certificate.

In the areas where drilling has been performed (Terracon 2020), no groundwater was identified within the Facility site boundary. No perennial streams are on or within the Facility boundary and no flood hazard exists. However, according to Terracon, Christmas Valley area is known for seasonal, shallow ponds, and lakes. Therefore, areas of ponding should be anticipated during wet-weather conditions. Groundwater level fluctuations are likely to occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the Terracon borings were performed.

H.8 PROPOSED SEISMIC HAZARD MITIGATION

OAR 345-021-0010(1)(h)(F)(i) *An explanation of how the applicant will design, engineer, construct and operate the facility to integrate disaster resilience design to ensure recovery of operations after major disasters; and*

Response: The State of Oregon uses 2018 IBC (International Code Council 2018), with current amendments by the OSSC and local agencies. Pertinent design codes as they relate to geology, seismicity, and near-surface soil are contained in IBC Chapter 16, Section 1613, with slight modifications by the current amendments of the State of Oregon and local agencies. The Facility will be designed to meet or exceed the minimum standards required by these design codes.

The flat terrain within the Facility site boundary are not expected to be prone to seismically induced landslides. No structures will be built on steep slopes that could be prone to instability, thus avoiding potential impacts.

H.9 PROPOSED NONSEISMIC HAZARD MITIGATION

OAR 345-021-0010(1)(h)(F)(ii) *An assessment of future climate conditions for the expected life span of the proposed facility and the potential impacts of those conditions on the proposed facility.*

Response: Nonseismic geologic hazards and impacts are anticipated to be minimal. Typical mitigation measures for nonseismic hazards include the following:

- Avoiding potential hazards
- Conducting subsurface investigations to characterize the soils to adequately plan and design appropriate mitigation measures
- Creating detailed geologic hazard maps to aid in laying out facilities
- Providing warnings in the event of hazards
- Ensuring that nonseismic geologic events are contemplated under *force majeure* provisions in any relevant Facility contracts

The subsequent sections discuss specific mitigation measures and best management practices (BMPs) for potential nonseismic geologic and soil hazards.

H.9.1 Landslide Mitigation

The site is flat (generally less than 1 percent; thus no landslide hazards are present. The solar modules and roads, including the access road and service roads, will be situated on flat-lying areas.

The engineering properties of the soils will be characterized to design proper trench slope laybacks and cut slopes, if needed.

H.9.2 Volcanic Eruption Mitigation

The USGS has established a Volcano Hazards Program Notification Service that consists of advisories, watches, and warnings (USGS 2022d; Stovall et al. 2016). The alert-notification system has been standardized and the goals are to accomplish the following:

1. Communicate a volcano's status clearly to nonvolcanologists.
2. Help emergency response organizations determine proper mitigation measures.
3. Prompt people and businesses at risk to seek additional information and take appropriate actions.

In the event of a volcanic eruption that could damage or affect Facility components, the Facility will be shut down until safe operating conditions returned. If an eruption occurred during construction, a temporary shutdown will most likely be required to protect equipment and human health.

H.9.3 Soil Erosion Mitigation

To reduce the potential for soil erosion, a detailed construction stormwater pollution prevention plan (SWPPP) will be developed for the Facility. The SWPPP will include both structural and nonstructural BMPs. Examples of structural BMPs include the installation of silt fences or other physical controls to divert flows from exposed soils, or otherwise limit runoff and pollutants from exposed areas within the Facility site boundary. Examples of nonstructural BMPs include management practices such as implementation of materials handling, disposal requirements, and spill prevention methods.

Because roads, solar modules, and other Facility components will be engineered, they will be subject to the requirements of a National Pollutant Discharge Elimination System (NPDES) stormwater construction permit. The Applicant's application for a NPDES stormwater construction permit is attached to Exhibit I and includes an erosion and sediment control plan.

In addition, Exhibit I contains a comprehensive list of mitigation measures to avoid wind and water erosion and soil impacts.

H.9.4 Collapsing Soils/Piping/Corrosion Mitigation

If localized areas of soils with collapsing or settling potential are identified during construction, these soils will be mitigated by overexcavating the soils and replacing them with compacted structural fill; placing impermeable material around the foundations to prevent wetting or saturation Mitigation of collapse potential of diatomaceous soils can also be addressed by distributing loads from Facility components and minimizing their magnitude in susceptible soil layers.

Testing of specific soils for corrosion potential (e.g. resistivity, pH) will conducted in order to identify specific on-site soils that will require mitigation potential corrosion of the steel post foundations. Mitigation for corrosion in surficial soils will be addressed by soil improvements, over-excavation and replacement by non-corrosive backfill. Drainage structures to prevent saturation and keep foundation soils dry will be constructed.

H.9.5 Disaster Resilience

The Facility will be designed to meet or exceed the minimum standards required by the IBC design code and OSSC 2019 in order to maintain core operations without interruption from a design basis earthquake. Critical structures will be designed for continued occupation and operation for a MConE; noncritical structures will require assessment following the MConE. The Applicant will evaluate the *Oregon Resilience Plan* (Oregon Seismic Safety Policy Advisory Commission 2013) during design of Facility components, and design for appropriate operation and operation recovery times.

The flat terrain that underlies the area within the Facility site boundary is not expected to be prone to seismically induced landslides.

As discussed in this Exhibit, nonseismic geologic hazards could include soil erosion, collapsed loess potential, and volcanic eruptions. Typical mitigation measures for nonseismic hazards include but are not limited to avoidance of potential hazards, creation of detailed geologic hazard maps to aid in laying out facilities, characterization of the subsurface soils to determine soil strength and foundation conditions, and provision of warnings in the event of hazards.

A detailed geotechnical exploration of the Facility will be conducted prior to construction, as discussed in Section H.4.2. The exploration will assess subsurface soil and geologic conditions, and provide information that will be used to identify geological or geotechnical hazards and facilitate design of foundations of structures and other supporting facilities. The exploration will also provide data for the installation of underground collector cables. This informed design process will aid in focusing design efforts on solutions that help to increase resiliency during the hazards identified above.

H.9.6 Future Climate Conditions

Future climate conditions should not have a major impact on the geologic, geotechnical, and seismic conditions at the Facility. Sea level rise will not affect Facility construction or operation. While increased rainfall intensity and a long-term increase in precipitation could theoretically lead to an increase in soil erosion compared to historical amounts, the site features relatively stiff cohesive soils, so the potential for erosion is limited. In addition, existing ancient landslides could become reactivated by saturation that occurs as a result of increased annual precipitation; however, no ancient landslides were observed at the site, the vast majority of the Facility is being constructed on extremely flat terrain, which is not prone to landslides.

Drought conditions and the attendant loss of vegetation could also lead to erosion and increase the potential for dust events. However, the onsite soils are relatively cohesive and resistant to wind erosion. Erosion and dust events are expected to be limited and, in any case, will not impact Facility operation (except to the extent that more frequent dust events could necessitate more frequent module cleaning and maintenance in order to maintain Facility output).

Increased potential for wildfire could have an impact on Facility function. Mitigation for this hazard potential includes mowing and limiting development of shrubs and growth of tall grasses within the Facility site boundary. Wildfire hazard is outside of the ability for control on a regional scale, but access to the solar array sites by firefighters, and care and maintenance of fuels on the fenced sites, will mitigate this hazard to the best possible extent.

Finally, the erosion and sediment control plan for the construction of the Facility is likely to specify that the existing vegetation is not to be removed but only scraped at the surface, to ensure that existing root systems remain in place to provide site stability.

H.10 SUMMARY

The risk of seismic hazards to human safety at the Facility is considered low. The Applicant has adequately characterized the area within the Facility site boundary and surrounding vicinity in accordance with OAR 345-022-0020(1)(a) and has considered seismic events and amplification for the Facility's specific subsurface profile. The Facility will consist of components such as new and improved roadways, solar module blocks, an operations and maintenance (O&M) building, a control house, and a transmission line. No facilities other than the O&M building will be continually staffed. The area is sparsely populated and historically has been either vacant or used for occasional cattle grazing. The probability of a large seismic event occurring while the Facility is occupied is low, which results in minimal risk to human safety over the majority of the Facility area and along the transmission line alignment. The risk to human safety is slightly higher at the O&M building, which is required to be designed to current seismic standards for structural safety.

Further, by adhering to IBC requirements, the Applicant has demonstrated that the Facility can be designed, engineered, and constructed to avoid dangers to human safety in case of a design seismic event. These IBC standards require that, for the design seismic event, the factors of safety used in the Facility design exceed certain values. For example, in the case of slope design, a factor of safety of at least 1.1 is normally required during the evaluation of seismic stability. This factor of safety is introduced to account for uncertainties in the design process and to ensure that performance is acceptable. Given the relatively low level of risk for the Facility, adherence to the IBC requirements will ensure that appropriate protection measures for human safety are followed.

The Applicant has provided appropriate site-specific information and demonstrated [in accordance with OAR 345-022-0020(1)(c)] that the construction and operation of the Facility, in the absence of a seismic event, will not adversely affect or aggravate the geological or soil conditions within the Facility site boundary or surrounding vicinity. The risks posed by nonseismic geologic hazards are considered to be low because the Facility can be designed to avoid or minimize the hazards of landslides, rockfall, soil erosion, and volcanic eruptions. Erosion hazards resulting from soil and wind action will be minimized with the implementation of an engineered erosion control plan.

Finally, the Applicant has demonstrated that the Facility can be designed, engineered, and constructed to avoid dangers to human safety resulting from the geological and soil hazards within the Facility site boundary, pursuant to OAR 345-022-0020(1)(d). Accordingly, given the relatively small risks these hazards pose to human safety, standard methods of practice (including implementation of the current IBC) will be adequate for the design and construction of the Facility.

H.11 REFERENCES

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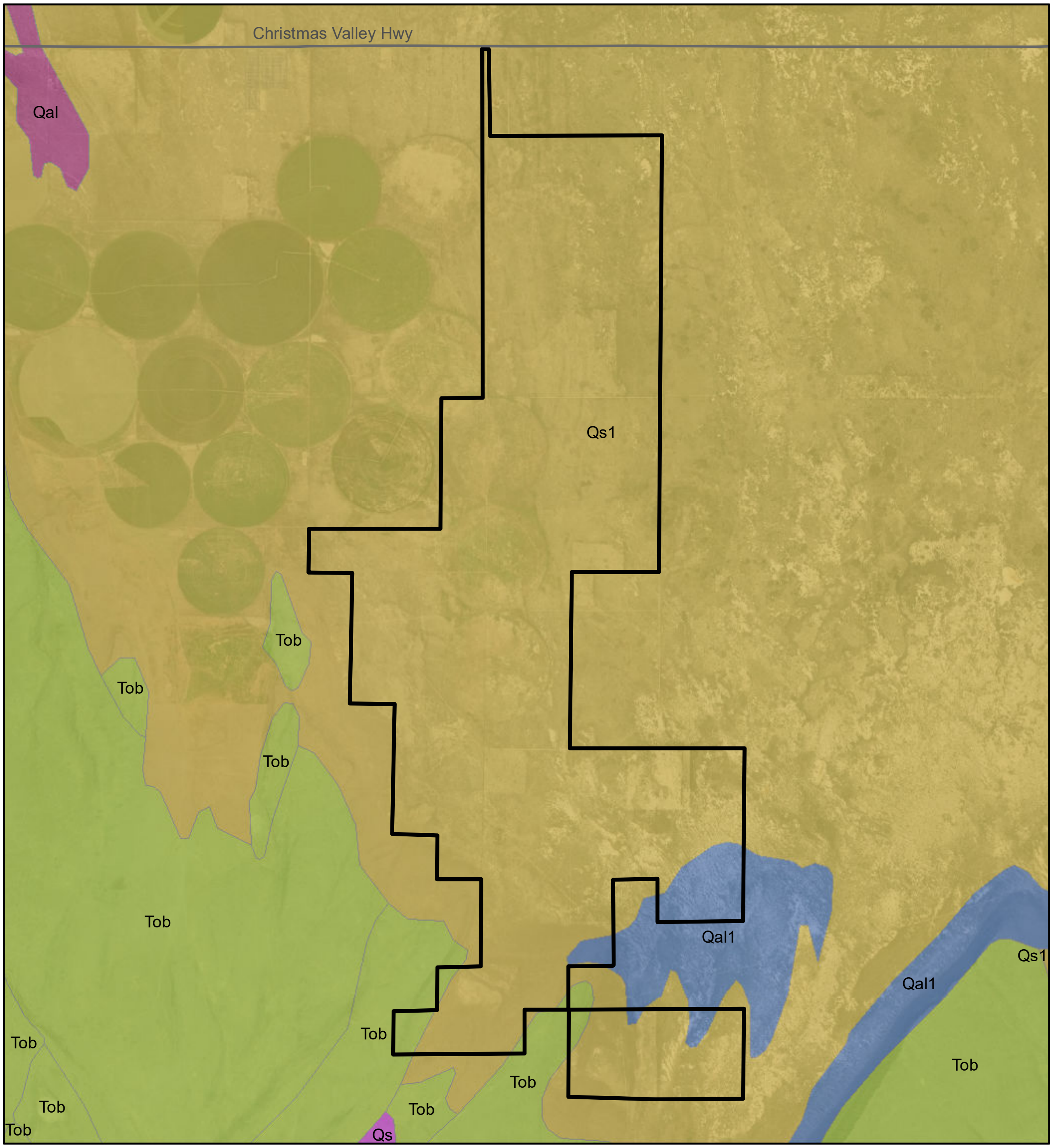
Figures



Legend

Facility Site Boundary

Geology Map Unit Formation

- Qal: Alluvium and surficial aeolian deposits
- Qal1: Sedimentary rocks and basalt flows
- Qs: Sedimentary rocks and basalt flows
- Qs1: Sedimentary rocks and basalt flows
- Tob: Tuffaceous sedimentary rocks, tuffs, and interbedded basalt and andesite flows

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Figure H-1
Geology Map
Application for Site Certificate
Archway Solar Energy Facility
Lake County, Oregon

Jacobs

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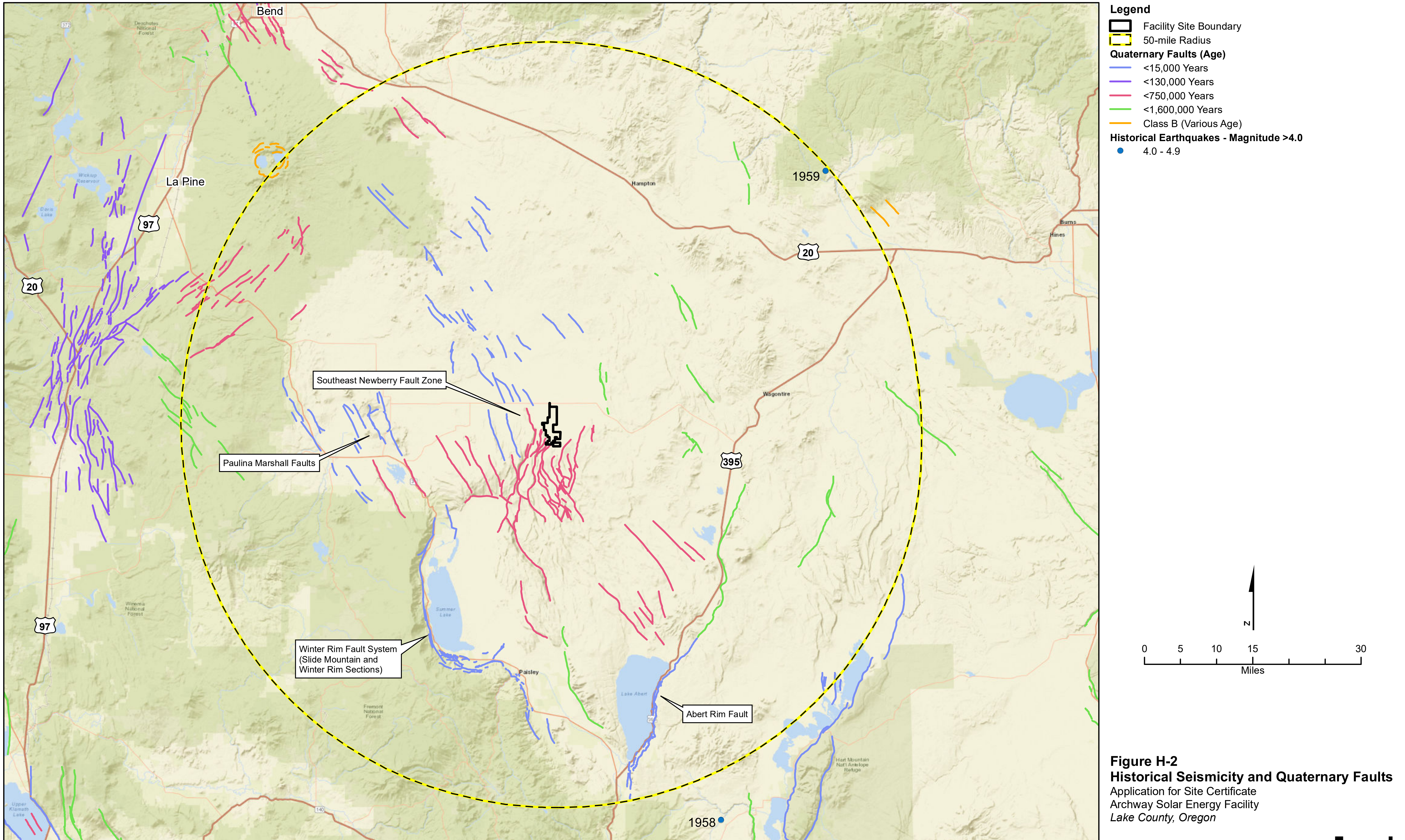

Figure H-2
Historical Seismicity and Quaternary Faults
Application for Site Certificate
Archway Solar Energy Facility
Lake County, Oregon

Jacobs

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PSH Deaggregation on NEHRP BC Rock
Invenergy Archway Solar: 43.200° N, 120.453° W
Mean Return Period: 475 years
Peak Horizontal Ground Acceleration: 0.1250 g

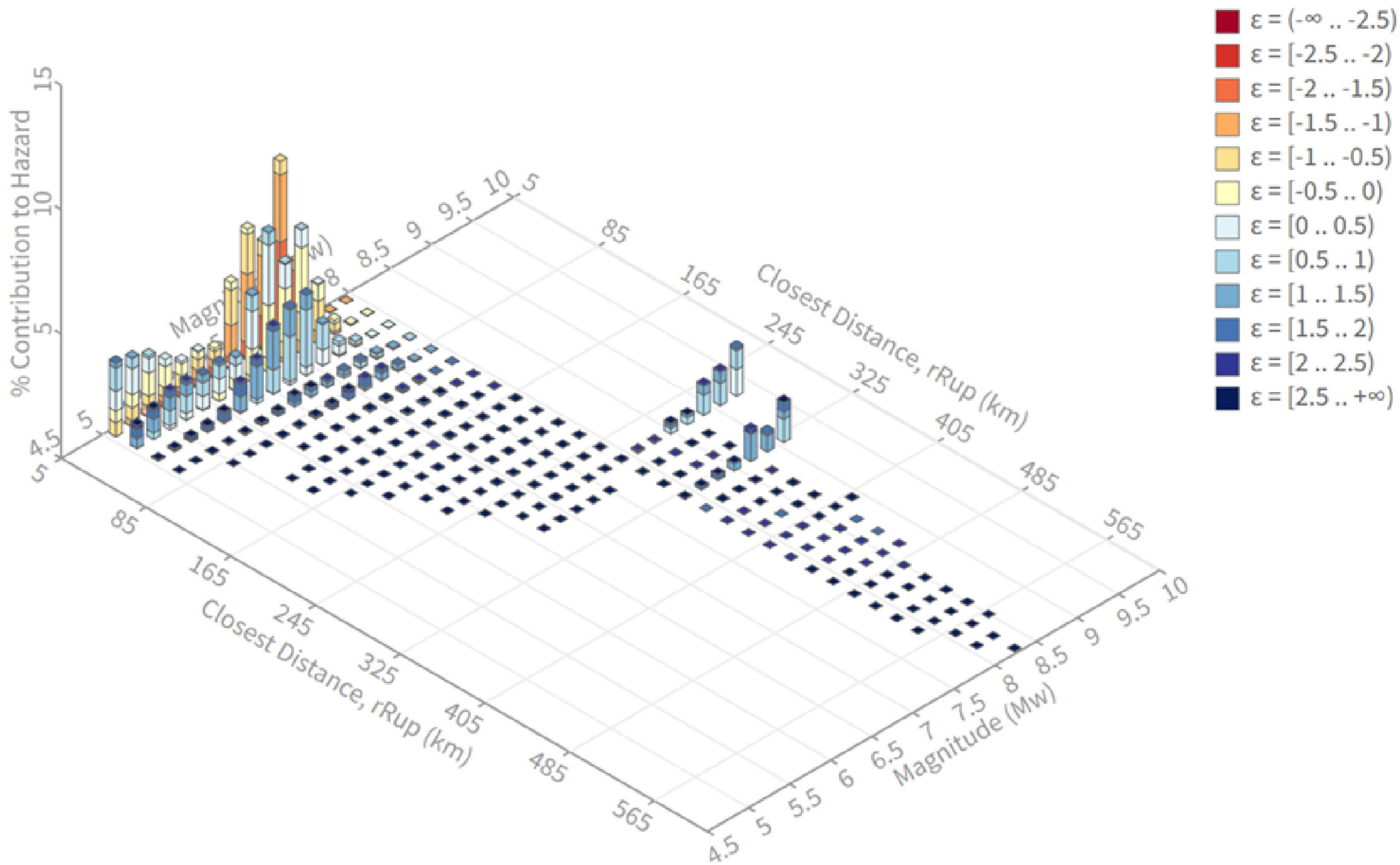


Figure H-3
Probabilistic Seismic Hazard Deaggregation for the Maximum Probable Earthquake Event
Application for Site Certificate
Archway Solar Energy Facility
Lake County, Oregon

Jacobs

PSH Deaggregation on NEHRP BC Rock
Invenergy Archway Solar: 43.200° N, 120.453° W
Mean Return Period: 2,475 years
Peak Horizontal Ground Acceleration: 0.2938 g

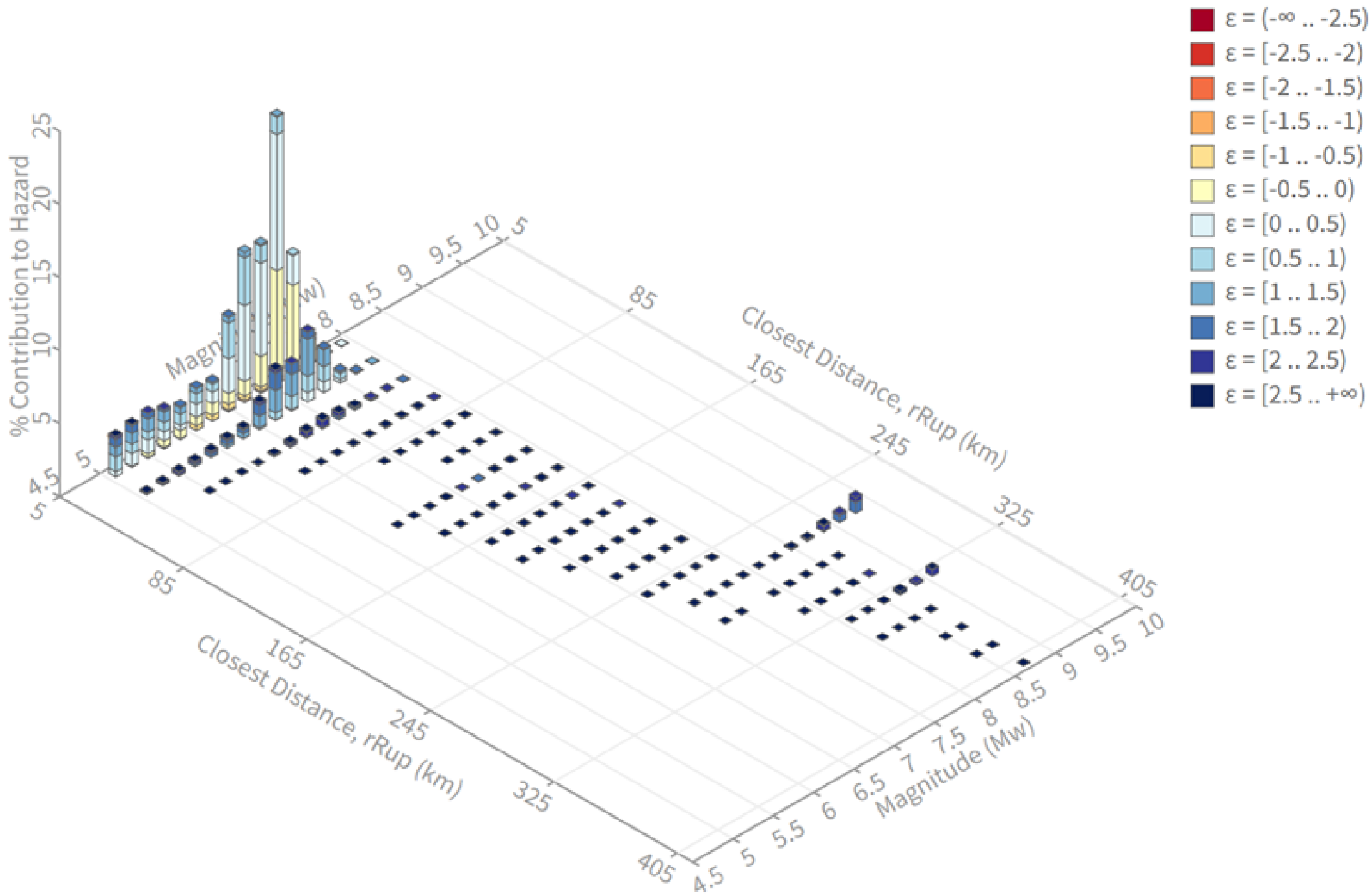


Figure H-4
Probabilistic Seismic Hazard Deaggregation for the Maximum Considered Earthquake Event
Application for Site Certificate
Archway Solar Energy Facility
Lake County, Oregon

Jacobs

Attachment H-1
Preliminary Geotechnical Engineering Report

Preliminary Geotechnical Engineering Report

Archway Solar
Christmas Valley, Lake County, Oregon
February 28, 2020
Terracon Project No. 82185058

Prepared for:
Invenergy Solar Development, LLC
Chicago, Illinois

Prepared by:
Terracon Consultants, Inc.
Portland, Oregon

February 28, 2020

Invenergy Solar Development, LLC
1 S Wacker Drive, Suite 1800
Chicago, Illinois 60606

Attn: Mr. Sydney Eiss – Staff Engineer
P: (312) 638 2893
E: seiss@invenergyllc.com

Re: Preliminary Geotechnical Engineering Report
Archway Solar
3 Mile Road
Christmas Valley, Lake County, Oregon
Terracon Project No. 82185058

Dear Ms. Eiss:

We have completed the Preliminary Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. P82185058 dated November 14, 2019, and in general accordance with Supplement to Agreement for Services Reference Number 82185058 dated February 18, 2020. This report provides a description of our project scope and summarizes the results for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

DRAFT

Brice W. Plouse, PE
Geotechnical Group Leader

Kristopher T. Hauck, PE
Principal | Office Manager

SME Review by: Scott D. Neely, PE, GE (CA)

REPORT TOPICS

Note: This report was originally delivered in a web-based format. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

PILE LOAD TESTING RESULTS

Axial Pile Load Test Results (Exhibit C-1 through C-8)
Compression Pile Load Test Results (Exhibit C-9 through C-12)
Lateral Pile Load Test Results (Exhibit C-13 through C-20)

Note: Refer to each individual Attachment for a listing of contents.

Preliminary Geotechnical Engineering Report

Archway Solar
3 Mile Road
Christmas Valley, Lake County, Oregon
Terracon Project No. 82185058
February 28, 2020

INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed solar project to be located at 3 Mile Road in Christmas Valley, Lake County, Oregon. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Site preparation and earthwork
- Thermal resistivity of trench/backfill
- Pile load test results
- Groundwater conditions
- Seismic considerations
- Electrical resistivity for grounding design
- Foundation design and construction

The geotechnical engineering Scope of Services for this project included the following:

- Fourteen (14) test borings drilled to approximate depths from 1 to 21½ feet below the existing ground surface (bgs);
- Five (5) Cone Penetration Tests (CPT) were explored to approximate depth from 2½ to 20½ feet bgs;
- Four (4) test pits excavated to approximately 10 feet bgs;
- Field soil electrical resistivity testing at four (4) locations to a maximum 'a' spacing of 200 feet;
- Pile load testing at four (4) locations that includes eight (8) axial tensile load tests, eight (8) lateral load tests, and four (4) compression load tests;
- Four (4) laboratory thermal resistivity dry-out curves tested by Geotherm;
- Corrosion testing performed on bulk samples obtained at 18 locations;
- Laboratory testing of soil samples;
- Geotechnical engineering analysis; and
- Preparation of this report

Maps showing the site and exploration locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. A log of each boring, CPT and test pit is included in the **Exploration Results** section of this report. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring and test pit logs and in the **Exploration Results** of this report. The pile load testing results are included in the **Exploration Results** section of this report.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available topographic maps.

Item	Description
Parcel Information	The project is located at 3 Mile Road in Christmas Valley, Lake County, Oregon. The site is approximately 4,100 acres of undeveloped crop land. Approximate center of property Latitude: 43.19208° Longitude: -120.44830° See **Site Location**
Existing Improvements	The majority of the proposed site is undeveloped with three approximate 160-acre (each) crop circles along the western site boundary. Gravel and/or earthen access roads appear spread throughout the property.
Current Ground Cover	The site appears to be covered with dense sage brush and other vegetation.
Existing Topography (from USGS 7.5' Vaughn Well Topo)	The global site is relatively flat, generally grading to the north with less than a 1% slope. However, we anticipate localized slopes to vary at the lacustrine deposit border of the basalt and sand dunes located along the southern property borders.

We also collected photographs at the time of our field exploration program. Representative photos are provided in our **Photography Log**.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Information from 2018: Sean Fallon with Invenergy LLC provided us with a Google Earth (.kmz) file of the approximate development area, as well as the Invenergy Solar Geotechnical Scope of Work document. Information from 2019: Sydney Eiss with Invenergy LLC provided us with Google Earth (.kmz) file of the updated, approximate development area with cultural, archeological and wildlife areas noted. Sydney also shared that we would not need biological, cultural or archeological oversight during the proposed explorations. However, field activities for the geotechnical scope of work would need to avoid areas of concern identified in the surveys.
Project Description (provided by Invenergy LLC)	The proposed solar arrays will be mounted on a single axis tracking racking system or fixed tilt system. The preferred foundations for either racking system would be driven steel piles; however, design loads and site conditions could dictate the need for pre-drilled undersized holes, pre-drilled oversized holes, drilled shafts, helical screw piles, or ballasted systems. Design loads for the racking system will be provided by Invenergy. Electrical equipment could be supported on concrete slabs on grade, spread footings, drilled piers, or driven steel piles at several locations on site. Where possible, it is desirable to minimize grading, without extensive earthwork or treatment of in-situ soils. On-site all-weather access ways are expected along with a gravel site access driveway connected to the nearest existing adjacent county road. It is also expected that each site will be enclosed within a chain link perimeter fence and swing gate at the facility's main entrance. Trenches for electrical conductors for both direct current (DC), alternating current (AC), and communication systems will be proposed throughout the project site.
Proposed Structure	It is our understanding that the Client intends to develop the site as a photovoltaic (PV) electric power plant. Ultimately, the power plant will consist of solar panels installed on steel structures and various other equipment and appurtenances associated with the power plant (e.g. switchgear, transformers, inverters, and overhead and underground electrical conveyance).
Building Construction	The site will include a photovoltaic (PV) solar power plant. We anticipate the PV structures will consist of steel piles, a racking system and PV modules. The steel pile foundations for the solar arrays are anticipated to consist of wide flange steel piles (W6 and W8 sections). We anticipate inverters may also be supported on wide flange steel piles, while transformers, and other appurtenant equipment are anticipated to be supported on shallow spread or mat foundations. We anticipate the probable switchyard/substation structure foundations will include drilled shafts (with diameters of 24 to 36 inches) for the support of pole-mounted equipment, and shallow mat foundations for the support of the transformer, circuit breaker and control house. Typical civil improvements for the project include crushed aggregate roads, AC Pavement approaches, grading, drainage improvements including infiltration basins (where applicable), and erosion control.

Item	Description
Maximum Solar Array Loads (assumed)	Structural loads were not provided, but have been estimated based on our experience on projects using single axis tracking rack systems: ■ Axial Downward (compression): 4 kips ■ Axial uplift (Tensile): 1½ kips; exclusive of frost heave loads ■ Lateral: 3½ kips
Grading	We anticipate minimal grading will occur throughout the site, with maximum changes in grade of up to 2 feet of cut or fill.
Access Road Ways	We anticipate aggregate surfaced roadways will be utilized for access roads, with paved approaches. We understand that access road cross sections used for construction of the project will be the responsibility of the EPC, and that only post construction traffic with an allowable rut depth of 2 inches is what we are to design for in this report. We anticipate low-volume, aggregate-surfaced and native soil access roads will have a maximum vehicle load of 30,000 lbs. and will travel over the access roads only once per week.
Estimated Start of Construction	Summer 2021

SUBSURFACE CONDITIONS

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. Conditions encountered at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** section.

As part of our analyses, we identified the following model layers within the subsurface profile.

Model Layer	Layer Name	General Description
1	**Topsoil**	Rootlet zone
2	**Probable Diatomaceous Earth**	Elastic Silt; Elastic Silt with Sand; Sandy Elastic Silt; Silt with Sand, tan, light brown, brown, medium stiff to hard
3	**Diatomaceous Earth**	Elastic Silt; Elastic Silt with Sand; tan, light brown, low to medium plasticity, medium stiff to hard, moisture content above 50%
4	**Sand**	Silty Sand, brown, fine-grained, loose to very dense
5[1]	**Gravel**	Silty Gravel with Sand, brown and gray, angular, very dense

1. Only encountered in B-13 and B-13A. Explorations B-13 and B-13A were terminated within this stratum.

Site Geology

Based on the Geology and Geologic Map of the East Half of the Crescent Quadrangle of Lake, Deschutes and Crook Counties (Walker, Peterson and Greene, 1967) the majority of the site is mapped as Quaternary sediments (Qs). These sediments are locally known as diatomaceous earth and classified as pleistocene lacustrine, fluviatile, aeolian tuffaceous sedimentary rocks, some ash flow tuff, and unconsolidated ash, pumice, clay, sand, silt and gravel. Locally the unit contains discontinuous layers of poorly consolidated conglomerate and in places mammalian fossils and remains of birds and fish, including some post pluvial aeolian deposits largely composed of silt and pumice fragments.

Diatomaceous earth can be defined as a soft, crumbly, porous sedimentary deposit formed from the fossil remains of diatoms. Diatoms are a type of algae with silicaceous skeletons. When the diatom dies, the skeleton persists, then it typically fills with water and sinks to the bottom of the body of water it occupied. Diatomaceous soil is often characterized by high water contents, low unit weights, and susceptibility to crumbling. The relic skeleton of the diatom creates a soil matrix that can often be described as a microscopic honeycomb type structure. While this structure may display relatively stiff strength parameters in laboratory and field testing, when the soil becomes overstressed it can breakdown the structure causing significant strain related movements.

The map also expresses limited portions of the southern and western boundaries of the site include Tertiary basalt (Tb) and a portion of the southeastern property as Quaternary alluvium and surficial aeolian deposits (Qal). The Tertiary basalt is described as olivine basalt with minor interbeds of tuff. Based on our explorations we believe the mapped Qal to be surficial wind-blown sand underlain by the Quarternary sediments described above. The approximately changes in geology as expressed by the above referenced publication are defined on the **Geologic Plan** in this report.

Based on our explorations we believe he soils encountered are generally consistent with the geologic mapping.

Seismic Hazards

Seismic hazards resulting from earthquake motions can include slope stability, liquefaction, and surface rupture due to faulting or lateral spreading. Liquefaction is the phenomenon wherein soil strength is dramatically reduced when subjected to vibration or shaking.

We reviewed the Statewide Geohazards Viewer (HazVu) published by the Oregon Department of Geology and Mineral Studies (DOGAMI) and available online at https://gis.dogami.oregon.gov/hazvu/. The viewer categorizes the expected earthquake shaking from light, moderate, strong, very strong, severe and violent; and the landslide susceptibility from low, moderate, high, and very high.

Earthquake Liquefaction Hazard: Moderate
Expected Earthquake Shaking: Moderate to strong
Landslide Susceptibility (due to earthquake): Low to moderate

Faults

The United States Geological Survey (USGS) Quaternary Fault and Fold Database of the United States published a report containing descriptions of nearby faults.

Faults North of Summer Lake (Class A) No. 833

Information	Description
Length	50 km
Strike (degrees)	N 10°W
Sense of Movement	Normal
Dip Direction	East
Slip-rate Category	Less than 0.2 mm/yr.
Most recent prehistoric deformation	Middle to late Quaternary <750 Ka
Distance from Fault	Along ridgelines at southern property boundary (see attached **Exploration Plan**)

Southeast Newberry fault zone (Class A) No. 835

Information	Description
Length	58 km
Strike (degrees)	N34°W
Sense of Movement	Normal, Left lateral
Dip Direction	70–90° SW
Slip-rate Category	Between 0.2 and 1.0 mm/yr
Most recent prehistoric deformation	Latest Quaternary (<15 ka)
Distance from Fault	2½ west of site

Based on our review of the available fault information, the depth to bedrock, and the site's proximity to the nearest known faults, it is our opinion that the risk of surface rupture due to ground faulting is low, with the exception of the ridgelines and slopes along the southern property boundary. However, we anticipate solar arrays will be limited in this area due to their north and east facing slopes and shallow bedrock conditions.

Groundwater Conditions

The boreholes were observed while drilling and after completion for the presence and level of groundwater. No groundwater was encountered during the explorations. It should be noted that groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

Additionally, the Christmas Valley area is known for seasonal, shallow ponds and lakes. Therefore, areas of ponding should be anticipated during wet-weather conditions for longer than regular periods due to the low hydraulic conductivity of the site soils.

Long-Term Soil Moisture and Temperature Conditions

We understand this project to be in the Preliminary Design stage and will likely be a year or more before it continues to the construction stage. Therefore, to provide the long-term moisture content for the combined soils within the depth of burial of the power lines we installed two Meter Group TEROS12 sensors that record both moisture and temperature near exploration B-4. The sensors, installed on February 14, 2020 at approximate depths of 2 and 3½ feet below the respective ground surface, are connected to a remote data logger device and collected periodically for a calendar year to provide site specific data. The data will be shared graphically, quarterly with a memorandum to the Preliminary Geotechnical Engineering Report.

We believe temperature and moisture content fluctuation to be valuable information for design of buried power cables due to the high thermal resistivity values of the diatomaceous earth (expressed in the **Thermal Resistivity Laboratory Testing** and **Exploration Results** sections of this report). The long-term moisture content and temperature of the soils can help the electrical designer in selecting an appropriate design thermal resistivity value for cable thickness determination.

Thermal Resistivity Laboratory Testing

Thermal resistivity testing was performed by Geotherm USA on 4 soil samples obtained at depths from just below the topsoil layer to depths of 1 to 4 feet below the respective ground surface at borings B-4, B-7, B-11, and B-13.

The dry-out curves were developed from 4 Shelby tube in-situ samples, and 4 bulk soil samples compacted to 85% and 95% of the maximum density determined in accordance with Standard Proctor criteria (ASTM D698) at the optimum moisture content and at intermediate moisture contents to develop the dry-out curves. The results of the thermal resistivity testing are presented in the **Exploration Results** appendix. The thermal resistivity obtained for the diatomaceous soils (B-4, B-7 and B-11) ranged from 123 to 240°C-cm/W for moist soils and from 480 to 694°C-cm/W for dry soils. The thermal resistivity obtained for the Silty Gravel soils (B-13) ranged from 106 to 127°C-cm/W for moist soils and from 279 to 353°C-cm/W for dry soils.

Upon review of the data we noted the 85% compaction sample for location B-7 and all results for location B-11 had dashed lines for thermal resistivity results for the 0 to 2 percent moisture content range. After questioning Nimesh Patel with Geotherm USA in regards to the dashed results he stated they utilized the dashed lines to express the materials extremely poor thermal resistivity characteristics.

Field Electrical Resistivity Test Results

The field electrical resistivity measurements were performed in general accordance with ASTM Test Method G 57, and IEEE Standard 81, using the Wenner Four-Electrode Method. The approximate soil resistivity test locations are shown in the **Exploration Results** appendix. The soil resistivity measurements were performed using an Ultra Mini-Res, Earth Resistivity and IP Meter, manufactured by L&R Instruments. The Wenner arrangement (equal electrode spacing) was used with the "a" spacing of 1, 2, 4, 8, 15, 25, 50, 75, 100, 150, and 200 at 4 locations within the proposed array area. The testing was performed in two near perpendicular orientations at each location. The "a" spacing is generally considered to be the depth of influence of the test. Results of the field soil resistivity measurements are presented in tabular and graphical format in the **Exploration Results** appendix. The resistivity ranged from as low as 230 ohm-cm to as high as 6880 ohm-cm.

Corrosivity

Samples for corrosion testing were obtained from 18 locations. The samples were obtained from depths of approximately 0 to 5 feet below existing ground surface. The samples were tested for pH, water soluble sulfate, sulfides, chlorides, total salts, Red-Ox potential, and electrical resistivity. The results of the Corrosion Series Testing are presented in the **Exploration Results** appendix.

The degradation of concrete or cement grout can be caused by chemical agents in the soil that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication ACI Building Code Requirements for Structural Concrete (ACI 318-14) provides guidelines for this assessment. The results of the sulfate tests indicate the potential for deterioration of concrete ranges from not applicable to severe. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation.

These test results are provided to assist in determining the type and degree of corrosion protection that may be required. We recommend that a certified corrosion engineer be employed to determine the need for corrosion protection and to design appropriate protective measures, if required.

Seismic Considerations

Based on the results of our site characterization program, we conclude that Site Class D is appropriate for the subject site. The following table provides the seismic design criteria in accordance with the 2019 Oregon Structural Specialty Code (OSSC) at the project site, obtained from the Structural Engineers Association Seismic Design Map (https://seismicmaps.org/) tool:

Site Class[1]	Site Latitude (ºNorth)	Site Longitude (ºEast)	S_s - Spectral Acceleration for a Short Period	S_1 - Spectral Acceleration for a 1-Second Period	F_a - Site Coefficient for a Short Period	F_v - Site Coefficient for a 1-Second Period
D	43.1963	-120.4543	0.529g	0.217g	1.376	1.966

1. The 2019 OSSC requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100-foot soil profile determination. Borings extended to a maximum depth of 21½ feet, and this seismic site class definition considers that similar or better subsurface conditions continue below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be required to change the current seismic site classification.

PRELIMINARY PILE LOAD TESTING (PLT) PROGRAM

We have performed a preliminary pile load testing program that included:

- Directing the installation of a group of three test piles at each of four locations.
- Performing testing under axial tensile loads for two test piles in each group.
- Performing testing under lateral loads for two test piles in each group.
- Performing testing under compression loads for one test pile in each group.

These activities are further described in the following sections.

Pile Location Procedures

The pile load testing locations are indicated on the attached **Exploration Plan**. These locations were established in the field by using a hand-held GPS (accurate to about 20 feet) and existing site features as reference points. Ground surface elevations were not obtained. The mapped test locations should be considered accurate only to the degree implied by the means and methods used to define them.

Test Pile Installation

The test piles consisted of wide-flange, bare steel W6x9 sections. A group of three test piles were installed at each of the four locations across the project site. The test piles have been identified using a location and embedment depth system. The pile identification system for each location begins with "PLT" and is followed by the number corresponding to the test pile group location and embedment depth.

The piles were advanced on January 21, 2020 with a track mounted GAYK Model HRE 4000 equipped with a hydraulic hammer to embedment depths of approximately 5 and 8 feet below the ground surface (bgs). Test piles were removed with a backhoe after testing and backfilled with cuttings from the excavation (compaction was applied to the backfilled materials by tamping bucket and tracking over excavation). The time rate of installation was recorded with a stopwatch. The total time required to advance each pile to its specified embedment depth was recorded and is summarized in the following table:

Pile Location	Pile Size	Actual Embedment Depth (feet)	Drive Time (seconds)
PLT-1	W6x9	5	21
	W6x9	5	29
	W6x9	8	77
PLT-2	W6x9	5	10
	W6x9	5	9
	W6x9	8	21
PLT-3	W6x9	5	9
	W6x9	5	8
	W6x9	8	17
PLT-4	W6x9	5	16
	W6x9	5	14
	W6x9	8	30

Testing Under Axial Tensile ("pull-out") Load

We performed testing under axial tensile load for the piles at each location using the procedures generally outlined below.

Eight (8) piles, two piles at each PLT location, were tested under axial tensile (“pull-out”) load. The “pull-out” load reaction was supported using Terracon’s proprietary 20-kip tripod frame supported at an appropriate lateral distance from the pile.

Axial loads were applied to the test pile using a 5-ton chain hoist. Connections to the test piles were made using a 6-ton plate clamp (vertical) designed for connection to W-sections.

The chain hoist and load cell were connected in series with chains and clevises to the two test piles, and the load was applied by pulling the chain through the chain fall in successive 500 lb increments from 0 lbs to the ultimate tension load of 7,000 lbs for each test pile. The limit of soil capacity during the tension test is defined as movement in excess of ¾-inch. Each load increment was sustained for about 30 seconds and the stabilized deflection reading of both indicator gauges were recorded.

Deflections were measured with Fowler High Precision 1-inch dial indicators, while loads were measured with a Dillon ED Junior Dynamometer 25-kip electronic load cell. The gauges were read and the data was recorded manually by Terracon field personnel.

The following photograph shows a typical axial tensile load test pile arrangement using the tripod system:

Typical Photo of Tripod with Chain Fall, Load Cell, and Dial Gauges.

Testing Under Lateral Load

After testing under axial tensile load, the piles at each location were then tested under lateral load as described below.

Eight (8) piles, two piles at each PLT location, were tested under lateral load. As the test piles were installed in-line with each other, the piles were connected together to provide a reaction for the opposite pile and tested simultaneously in the strong axis direction.

For lateral testing, the pair of piles were pulled toward each other and deflections of each pile were measured. The load for the lateral tests was applied at about 4 feet above the ground surface against the strong axis of the piles. The loads were applied in 500 lb increments in 5 cycles from 0 lbs to the ultimate lateral load of 7,000 lbs or the limits of the soil capacity, whichever occurred first for each test pile. The limit of soil capacity during the lateral test is defined as movement in excess of 1-inch at 6 inches above the ground surface. Each load increment was held for at least 1 minute and the stabilized deflection reading of both indicator gauge was recorded.

Deflections were measured with Fowler High Precision 1-inch dial indicators and Mitutoyo 4-inch dial indicators, while loads were measured with a Dillon ED Junior Dynamometer 25-kip electronic load cell. The gauges were read and the data was recorded manually by Terracon field personnel.

The following photograph shows a typical arrangement of two piles connected together for a lateral load test:

Typical Photo of Load Cell Connected to Two Test Piles.

Testing Under Compression Load

Four (4) piles, one pile in each PLT location, were tested under compression load. The compression load reaction was supported using a 20-ton track-hoe supported at an appropriate distance from the test pile.

Compression loads were applied to the test pile using a compression load cell and a hydraulic jack that sat between the pile and track-hoe. The bottom of the hydraulic jack was clamped to a metal plate which sat above the load cell which sat above the pile head. The top of the hydraulic jack was clamped to metal plate which sat underneath the cab of the track-hoe.

The hydraulic jack was used to apply compression loads at 500 lb increments from 0 lbs to the ultimate compression load of 10,000 lbs for each test pile. The limit of soil capacity during the compression test is defined as movement in excess of ¾-inch. Each load increment was sustained for about 30 seconds and the stabilized deflection reading of both indicator gauges were recorded.

Deflections were measured with Fowler High Precision 1-inch dial indicators, while loads were measured with a 10,000 pound digital compression load cell. The gauges were read and the data was recorded manually by Terracon field personnel.

Summary of Pile Load Test Results

The following table provides a summary of each test pile location, embedment depth, total drive time, compression load at ¼-inch of vertical displacement, uplift load at ¼-inch of vertical displacement, and the lateral load at ½-inch of lateral displacement:

Pile Location	Actual Embedment Depth (feet)	Drive Time (seconds)	Compression Load at ¼-inch Displacement (lbs)	Uplift Load at ¼-inch Displacement (lbs)	Lateral Load at ½-inch Displacement (lbs)
PLT-1	5	21	10,000+	N/A	N/A
	5	29	N/A	4,500+	2,750
	8	77	N/A	4,500+	3,000+
PLT-2	5	10	3,000	N/A	N/A
	5	9	N/A	1,900	2,700
	8	21	N/A	6,600	3,300
PLT-3	5	9	2,300	N/A	N/A
	5	8	N/A	800	2,050
	8	17	N/A	3,400	3,600
PLT-4	5	16	10,000+	N/A	N/A
	5	14	N/A	6,500	2,200
	8	30	N/A	7,000+	3,200

1. N/A = not applicable. Pile not tested in loading direction.

Results of the full Pile Load Testing (PLT) program can be found in the **Pile Load Testing Results** section of this report.

Based on the test results in the above table, we have determined there are two zones for consideration for axial capacity. With regard to the lateral pile load tests, all of the locations performed similarly and thus only one set of design data have been provided for lateral load design.

CONTRIBUTORY RISK COMPONENTS

Item	Description
Supplemental Exploration and Testing	Additional soil test borings should be performed to adequately explore the site as part of a design-level study. Additionally, a full-scale pile load testing (PLT) program should be considered as the project design progresses. The results of a full-scale PLT program in conjunction with soil test CPT/test pit results are often successful in reducing the design embedment depth when compared to designs solely based on explorative results and analytical methods.
Soil Conditions	The majority of the site is underlain by diatomaceous earth, a soft, crumbly, porous sedimentary deposit formed from the fossil remains of diatoms. Diatomaceous soil is often characterized by high water contents, low unit weights, and susceptibility to crumbling. The relic skeleton of the diatom creates a soil matrix that can often be described as a microscopic honeycomb type structure. While this structure may display relatively stiff strength parameters in laboratory and field testing, when the soil becomes overstressed it can breakdown the structure causing significant strain related movements. The strengths of this material is largely variable across the site and has very high thermal resistivity values. The southern portion of the site has basaltic bedrock formations, as well as surficial sand dunes.
Liquefaction	Due to the plasticity and stiffness of the site soils we believe liquefaction risk to be low even through the site lies within a seismically active region. It should also be noted that faults lie within the southern portion of the site. Based on the location of the faults and anticipated movement we believe the risk of faulting to be low.
Access	Diatomaceous soils are known for localized seasonal ponding and lakes. Therefore, wet and loose/soft surface conditions due to rainwater will create access issues for vehicles. The sites climate is relatively arid and receives limited amounts of precipitation throughout the year. The site will generally be more accessible in the summer and early fall due to the improved drying conditions. The site is currently covered in heavy sage brush which creates difficult access prior to site clearing.
Grading	We anticipate very little grading will be required with the exception of grading the dune sand. On-site materials that are used as fill or backfill will likely require drying prior to re-compaction as engineered fill. Alternatively, these materials could be replaced with imported soils or graded dune sands containing an appropriate moisture content. We expect localized areas of unsuitable conditions will be encountered prior to placing fill and within the subgrade for roadways and shallow foundations that are planned. Stabilization measures, such as undercutting/replacement.
Groundwater	Groundwater was not observed in the explorations, however high moisture contents were recorded within the diatomaceous earth soils. Based on our experience in the project area, groundwater levels are much deeper than the anticipate maximum exploration depth of 4 to 6 feet below the ground surface.

Item	Description
Site Drainage	The site soils have poor drainage characteristics. Site drainage should be established early in the project to limit the anticipated localized, seasonal ponding of precipitation.
Corrosion Hazard	The results of our laboratory testing of soil chemical properties are expected to assist a qualified engineer to design corrosion protection for the production piles and other project elements.
Excavation Hazards	Based on the results of our borings, CPTs, test pits, pile load testing and our experience with the geology of the project site, we do not expect that difficult excavation conditions or widespread obstructions to pile driving operations will be encountered during construction, unless arrays are planned on the basalt hill slopes on the southern site boundary. Although high in moisture content we expect general instability in the form of caving, sloughing, and raveling to be encountered in excavations deeper than 4 feet below the ground surface. Excavations will likely require bracing, sloping, and/or other means to create safe and stable working conditions.
Slope Hazards	The site is relatively flat with the exception of the slopes on the southern hill sides. Due to the basalt formation and current slopes we believe the risk of slope failures to be negligible to low.
Anticipated Pile Drivability	With the exception of the southern hill sides and slopes there is a low likelihood of encountering difficulties during pile driving. We do not anticipate pre-drilling to be required.
General Construction Considerations	The near-surface soils are moisture sensitive and subject to degradation with exposure to moisture. To the extent practical, earthwork should be performed during warmer and drier periods of weather to reduce the amount of necessary subgrade remedial measures for soft and unsuitable conditions beneath access roadways, equipment pads, etc.

PHOTOVOLTAIC (PV) SOLAR ARRAY FIELD – PRELIMINARY RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

Geotechnical Considerations

We would expect the PV panels to be supported by driven piles, while inverters embedded in the array field, could be supported on mat foundations and/or driven piles. The proposed structure types and loading information was not available at the time of this report, but we anticipate NEXTracker or Soletec racking and loading will be utilized. Settlement and strength parameters were analyzed using soil compressibility properties derived from the SPT borings, CPT explorations and the results of pile load test results.

Results of the pile load tests indicate that driven steel piles should be suitable for support of the planned solar panels. Piles with embedment depths between 5 and 8 feet should be suitable for support of PV array panels. We have provided geotechnical engineering parameters in this report to assist the designers of production piles.

As part of the overall quality control program, the time rate of installation (seconds per foot of embedment) should be recorded during production post driving. As a direct extension of the design process, additional "proof" testing should be performed on a representative number of production piles that do not meet the minimum installation rate criteria outlined in this report.

Geotechnical engineering recommendations for foundation systems and other earth connected phases of the project are outlined in this report. The recommendations contained in this report are based upon the results of field and laboratory testing, engineering analyses, and our current understanding of the proposed project.

The **General Comments** section provides an understanding of the report limitations.

Preliminary Solar Panel Support Pile Design Recommendations

Axial Capacity Recommendations

The axial uplift capacity of driven piles may be estimated based on skin friction developed along the perimeter of the pile, while the compression capacity may be estimated using the skin friction and end bearing. When determining embedment depths, the perimeter of a wide flange beam should be taken as twice the sum of the flange width and section depth. The upper 12 inches of soil for each pile should be neglected in the axial capacity analyses.

Based on the results of the pile load testing program, we have broken identified the site into two (2) zones. Zone "A" is assigned to the results from Pile Load Testing locations PLT-1 and PLT-4, where we encountered higher skin friction and end bearing loads, and Zone "B" is assigned to results from PLT-2 and PLT-3, where we encountered low skin friction and end bearing loads. Exact boundaries cannot be established at this time due to the sparse amount of exploration data. The ultimate axial capacity of driven steel piles may be calculated using skin friction and end bearing values as presented in the following table for each individual zone:

Zone	Minimum Pile Embedment Depth (ft)	Ultimate Uplift and Compression Unit Skin Friction (psf)	Ultimate End Bearing (lbs)
A	5	550	3,000
	8		
B	5	200	1,400
	8	400	

The ultimate unit skin friction is based on the results of the uplift load testing. The ultimate end bearing values provided are based on the results of the CPT explorations and compressive load testing.

The above skin friction and end bearing values are applicable for piles that are driven for a 5-foot embedment using equipment similar to a GAYK Model HRE 4000 equipped with an Atlas Copco IM400 hydraulic hammer. If a smaller or larger drive hammer is used, we recommend Terracon be consulted to determine the minimum drive time based on the proposed equipment to be used for driving of the piles.

For Allowable Stress Design (ASD) design, we recommend the allowable skin friction and end bearing be determined by applying a minimum factor of safety of at least 1.5 to the ultimate values.

Piles should have a minimum center-to-center spacing of at least 5 times their largest cross-sectional dimension to prevent reduction in the axial capacities due to group effects.

Preliminary Lateral Capacity Recommendations

Lateral load response of pile foundations was calculated using the computer program L-Pile 2019, by Ensoft, Inc. The stiffness of the pile and the stress-strain properties of the surrounding soils determine the lateral resistance of the foundation. We modeled the lateral response of the tested piles to evaluate L-Pile input parameters that can be used for design of the production piles. Recommended L-Pile input parameters for preliminary lateral load analysis for driven pile foundations are shown in the following table:

Zones A and B - Pile Embedment Soil Parameter					
Depth to Bottom of Layer (feet)	**Soil Type**	**Effective Unit Weight (pcf)**	**Effective Friction Angle (Φ')**	**Undrained Cohesion (psf)**	**k Value (pci)**
0 – 8	Sand (Reese)	80	36	N/A	Allow LPILE to choose this value

Based on the pile load testing results and L-Pile input parameters used for design of the production piles, the recommended P-multiplier for different pile embedment depths are shown in the following table.

Embedment Pile (feet)	P Multiplier
5	3.5
8	5.0

L-PILE analyses were performed by applying the field test load that resulted in approximately ½-inch deflection at a point about 6 inches above the ground surface. The shear load was applied at approximately 4 feet above the ground surface. The effective unit weight and friction angle were based on the results of the borings and Cone Penetration Test explorations. The p-multiplier was then adjusted (by trial and error method) such that the applied load resulted with a deflection value that matched the in-situ test results. Please note that this procedure was based on only one discreet set of data determined at about 6 inches from the ground surface during the field load testing. These results should be used for L-PILE analysis only using the 2019 version of L-Pile. These parameters are only applicable to piles embedded between 5 and 8 feet below grade. In our evaluation, the piles were modeled as an elastic section (non-yielding).

The structural engineer should evaluate the moment capacity of the pile as part of their structural evaluation. Piles should have a minimum center-to-center spacing of at least 5 times their largest cross-sectional dimension in the direction of the lateral loads, or the lateral capacities should be reduced due to group effects. If piles will be spaced closer than 5 times their largest cross-sectional dimension we should be notified to provide supplemental recommendations.

Construction Considerations

Based on the field exploration and laboratory testing, it is our opinion that the soils on the site are suitable for direct driving pile installation.

A representative of the geotechnical engineer should observe pile driving operations. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

Preliminary Pile Design Recommendations for Other Structures

Other structures (i.e. inverters and embedded poles) that are planned to be supported on deep foundation systems similar to the solar panels may require piles to be driven to greater depths in order to achieve the required axial capacities.

The table in **Axial Capacity Recommendations** can be used to determine an ultimate skin friction for piles embedded between depths of 5 and 8 feet. When determining embedment depths, the perimeter of a wide flange beam should be taken as twice the sum of the flange width and web depth, and the upper 1-foot of soil for each pile should be neglected.

The ultimate unit end bearing for alternate pile sections should be assumed to be the same as the W6x9 pile tested for this project.

We recommend Terracon be consulted to determine the minimum drive time based on the proposed equipment to be used for driving of the piles.

For allowable strength design, we recommend the allowable skin friction be determined by applying a factor of safety of at least 1.5 to the ultimate values provided in this section for pile embedded greater than 5 feet. We recommend a factor of safety of at least 2 be applied to the end bearing ultimate value provided in this section for piles embedded greater than 5 feet. For a full-scale/final design we believe further pile load testing is required.

Piles should have a minimum center-to-center spacing of at least 5 times their largest cross-sectional dimension to prevent reduction in the axial capacities due to group effects.

Preliminary Driven Pile Embedment Analysis

We have performed preliminary geotechnical and structural analyses for evaluating the embedment depths of driven pile foundations to support the typical Soltec and NEXTracker racking systems when installed in native soils. This analysis is based on the results of our widely-spaced soil explorations, the structural loads as provided by Soltec, NEXTracker, LPILE parameters derived from this preliminary study, and other noted assumptions. Subsequent analyses will be required once design level geotechnical information is available and after other design considerations are more fully defined. Therefore, the results of the analyses described below should not be used for design. Rather, these analyses are intended to assist you in roughly evaluating construction costs and development viability for the proposed project.

Our analyses have not considered the potential loss of steel due to corrosion during the design life of the structure. The final structural design should consider the anticipated steel loss as determined by a qualified Corrosion Engineer. Thicker pile sections or additional corrosion protection measures may be required if steel loss is predicted by corrosion analyses.

Applied Loads

Approximate structural loading conditions were analyzed based on the provided top-of-pile loading conditions as provided in the following tables. The actual top-of-pile structural loads will vary based on the selected racking system and the Manufacturer's load information as determined in accordance with requirements by the applicable building codes and local municipality.

For the Soltec racking system analyses, the following table outlines the top-of-pile loads used in our structural analysis and the resulting preliminarily recommended pile section and embedment depths.

Soltec Racking System						
Pile Type	Compression (kips)	Uplift (kips)	Shear (kips)	Moment (kip-ft)	Preliminary Design Pile Shape	Height of Lateral Load (ft)
2x42P_Simple - Exterior	2.470	0.712	2.250	0.000	W8x13	6¾

Soltec Racking System						
2x42P_Motor- Exterior	2.390	0.618	2.140	23.340	W8x18	6¾
2x42P_Simple - Interior	1.580	0.144	0.621	0.000	W6x8.5	6¾
2x42P_Motor- Interior	1.560	0.088	0.594	22.760	W8x15	6¾
2x45P_Simple - Exterior	2.130	0.617	1.880	0.000	W6x8.5	6¾
2x45P_Motor- Exterior	2.290	0.538	1.980	24.580	W8x18	6¾
2x45P_Simple - Interior	1.720	0.156	0.677	0.000	W6x8.5	6¾
2x45P_Motor- Interior	1.930	0.103	0.734	23.990	W8x18	6¾

For the NEXTracker racking system analyses, the following table outlines the top-of-pile loads used in our structural analysis and the resulting preliminarily recommended pile section and embedment depths.

NEXTracker Racking System						
Pile Type	Compression (kips)	Uplift (kips)	Shear (kips)	Moment (kip-ft)	Preliminary Design Pile Shape	Height of Lateral Load (ft)
Motor - Exterior	3.153	1.712	1.520	14.865	W6x20	6
P2 - Exterior	2.510	1.171	2.071	0.288	W6x12	6
P3 & P5 - Exterior	2.390	1.096	2.389	0.308	W6x15	6
P4 & P6 - Exterior	2.646	1.225	2.920	0.276	W6x15	6
P7 - Exterior	0.884	0.421	1.013	0.174	W6x8.5	6
Motor - Interior	2.215	0.799	0.844	9.549	W6x12	6
P2 & P7 - Interior	1.948	0.680	0.816	0.205	W6x8.5	6
P4 - Interior	1.304	0.484	0.903	0.203	W6x8.5	6
P3, P5 & P6 - Interior	1.641	0.442	0.922	0.194	W6x8.5	6

Axial Pile Capacities

The ultimate axial capacity for this analysis used the information previously presented in this report under **Axial Capacity Recommendations**.

For our preliminary analyses, a Factor of Safety (FS) of 1.5 was applied to the ultimate skin friction and 2.0 was applied for ultimate end bearing parameters, respectively. The ultimate unit skin friction was determined using the soil strength parameters based on the pile load test results.

The axial tensile (pull-out) capacity is developed from skin friction while the axial compressive capacity is developed from skin friction and end bearing. The above indicated ultimate skin friction values, used with appropriate FS, may be used for uplift and compressive loading. The skin friction perimeter should be calculated using the perimeter of the pile which equals twice the sum of the flange width and web depth. Conservatively, the upper 1 feet of soil should be neglected when calculating skin friction.

Piles should have a minimum center-to-center spacing of at least 3 times their largest cross-sectional dimension to prevent reduction in the axial capacities due to group effects. If the piles are designed using the above parameters, settlements are not anticipated to exceed 1 in.

Lateral Analyses

Each pile type was modeled in LPILE v 2018.10.06 with the loading conditions applied at 6¾ ft for SOLTEC and 6 feet for NEXTracker above the ground surface as indicated in the above tables. The soil parameters utilized in the analyses are those presented in the "Preliminary Lateral Capacity Recommendations" section previously presented in this report.

Based on the analyses, the following Soltec pile shape and embedment depths were determined:

Zone A Soltec		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
2x42P_Simple - Exterior	W8x13	5.5
2x42P_Motor- Exterior	W8x18	6
2x42P_Simple - Interior	W6x8.5	5
2x42P_Motor- Interior	W8x15	5.5
2x45P_Simple - Exterior	W6x8.5	5.5
2x45P_Motor- Exterior	W8x18	6
2x45P_Simple - Interior	W6x8.5	5
2x45P_Motor- Interior	W8x18	5.5

Zone B Soltec		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
2x42P_Simple - Exterior	W8x13	6
2x42P_Motor- Exterior	W8x18	6
2x42P_Simple - Interior	W6x8.5	5

Zone B Soltec		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
2x42P_Motor- Interior	W8x15	5.5
2x45P_Simple - Exterior	W6x8.5	6
2x45P_Motor- Exterior	W8x18	6
2x45P_Simple - Interior	W6x8.5	5
2x45P_Motor- Interior	W8x18	5.5

Based on the analyses, the following NEXTracker pile shape and embedment depths were determined:

Zone A NEXTracker		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
Motor - Exterior	W6x20	5½
P2 - Exterior	W6x12	5
P3 & P5 - Exterior	W6x15	5.5
P4 & P6 - Exterior	W6x15	6
P7 - Exterior	W6x8.5	5
Motor - Interior	W6x12	5
P2 & P7 - Interior	W6x8.5	5
P4 - Interior	W6x8.5	5
P3, P5 & P6 - Interior	W6x8.5	5

Zone B NEXTracker		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
Motor - Exterior	W6x20	7.5
P2 - Exterior	W6x12	6
P3 & P5 - Exterior	W6x15	6

Zone B NEXTracker		
Pile Type	**Required Pile Shape**	**Recommended Embedment Depth (ft)**
P4 & P6 - Exterior	W6x15	6
P7 - Exterior	W6x8.5	5
Motor - Interior	W6x12	6
P2 & P7 - Interior	W6x8.5	5.5
P4 - Interior	W6x8.5	5
P3, P5 & P6 - Interior	W6x8.5	5

The analyses were performed by starting out with the design pile shape and minimum embedment depth to support the compression and/or tension load for each pile type. The pile embedment was deepened as necessary until a lateral deflection less than or equal to 0.60-inches was achieved at the ground surface. If the deflection criteria could not be met by deepening the pile embedment due to the pile reaching a point of fixity, the next larger size of pile was modeled.

As stated earlier, our analyses have been performed using preliminary information and are intended to assist you in roughly evaluating construction costs and viability for the proposed project. Ultimately, the design of foundations for the solar panel racking system will depend on a number of factors including the actual structural loading conditions, the structural serviceability requirements, anticipated corrosion losses, a detailed understanding of the site soil conditions, and other factors where complete and final information is not available at this time.

MAT FOUNDATIONS FOR SUPPORT OF INVERTERS

General

We understand the main foundation component in the array area will include driven pile foundations for support of solar arrays; however, some lightly-loaded, inverter structures are typically required across the site. In general, small, lightly-loaded, inverter structures may be supported on driven piles or isolated mat/slab foundation systems.

If the site has been prepared in accordance with the requirements noted in the **Earthwork** section of this report, the mat/slab foundations should be designed based on the criteria outlined below:

Mat/Slab Foundation Design Recommendations

Design Item	Description/Recommendations
Foundation Type	Mat/Slab Foundations
Minimum Embedment Depth	24 inches
Bearing Material	6 inches of Compacted, dense, Select Fill over prepared native subgrades
Design Modulus of Subgrade Reaction, k	250 pci
Minimum Width	4 feet
Modulus Correction Factor [1]	$k_c = k((b+1)/2b)^2$
Maximum Design Contact Stress	2,000 psf
Total Estimated Settlement	1 inch or less
Estimated Differential Settlement	About 2/3 of total settlement

1. It is common to reduce the k-value to account for dimensional effects of large loaded areas. Where k_c is the corrected or design modulus value and b is the mat width (short dimension) or tributary loaded area.

Foundations should be reinforced as necessary to reduce the potential for distress caused by differential foundation movement. The use of joints at openings or other discontinuities in walls is recommended.

Foundation excavations should be observed by the geotechnical engineer. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

Mat/Slab Foundations Construction Considerations

The mat foundation excavations should be evaluated under the direction of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the foundation excavations should be removed/reconditioned before foundation concrete is placed.

EARTHWORK

General

The site work conditions will be largely dependent on the weather conditions and the contractor's means and methods in controlling surface drainage and protecting the subgrade. Site preparation for the inverter mat foundations locations should include clearing and grubbing, installation of a site drainage system (where necessary), and subgrade preparation. Site preparation is not necessary in the Photovoltaic (PV) array field or where inverters will be supported on driven piles except to improve site drainage where necessary. The following paragraphs present our considerations and recommendations for the PV array field and access roadway portion of the site and subgrade preparation.

The following presents recommendations for site preparation, excavation, subgrade preparation and placement of engineered fills on the project. The recommendations presented for design and construction of earth supported elements including foundations and roadways are contingent upon following the recommendations outlined in this section.

Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, foundation bearing soils, and other geotechnical conditions exposed during the construction of the project.

Site Preparation

Strip and remove existing vegetation, crops, debris, and other deleterious materials from proposed mat foundations supporting inverters. Trees, tree stumps, and large vegetation should be cleared from the site at the location of mat foundations supporting inverters. Exposed surfaces should be free of mounds and depressions which could prevent uniform compaction in proposed array panel and inverter areas.

Stripped materials consisting of vegetation and organic materials should be wasted from the site. If it is necessary to dispose of organic materials on-site, they should be placed in non-structural areas.

Where proposed inverters will be located, the area should be initially graded to create a relatively level surface to receive fill or be constructed upon, and to provide for a relatively uniform thickness of fill beneath structures (if applicable).

Subgrade Preparation

In mat/slab foundations areas, engineered fill should extend below proposed foundations to depths indicated in the following table.

Foundation Type	Depth of Fill Below Foundations	Lateral Extent of Fill Beyond the Edge of Foundations
Mat/Slab	A minimum of 6 inches below the mat/slab foundation bottom	A minimum of 12 inches horizontally beyond the edges of mat/slab foundations

Foundation Type	Depth of Fill Below Foundations	Lateral Extent of Fill Beyond the Edge of Foundations
Recommendations assumes mat/slab thickness is 6 inches.		

After cutting the Access Roadways to design subgrade elevation, where required by the grading plan, we recommend that the exposed subgrade be proofrolled prior to fill placement. Proofrolling should be performed with heavy rubber-tired construction equipment, such as a fully-loaded tandem-axle dump truck, to detect soft and/or yielding soils. Proof-rolling and scarification and compaction are not always practical within confined excavations or when plastic soils are present. Unsuitable areas identified by proof-rolling and/or hand-probing by the geotechnical representative should be repaired with on of the stabilization measures defined below. Vibrating compactors (smooth drum or plate) should not be used on the fine-grained soils. Sheepsfoot compactors should be used for fine-grained soils.

Based on the outcome of the proofrolling operations, some overexcavation or subgrade stabilization should be expected, especially during wet periods of the year. Methods of stabilization, which are outlined below, could include scarification and recompaction and/or removal of unstable materials and replacement with granular fill (with or without geotextiles). The most suitable method of stabilization, if required, will be dependent upon factors such as schedule, weather, size of area to be stabilized and the nature of the instability.

- **Scarification and Recompaction** - It may be feasible to scarify, dry, and recompact the exposed soils only during the extended dry season. Very limited use of this method should be considered feasible for the site. The success of this procedure will depend primarily upon favorable weather and sufficient time to dry the soils. Even with adequate time and weather, stable subgrades may not be achievable if the thickness of the soft soil is greater than about 1 to 1½ feet.

- **Granular Fill** - The use of crushed stone or gravel could be considered to improve subgrade stability. Typical undercut depths typically range from about 1 to 1½ feet. The use of high modulus geotextiles (i.e., engineering fabric such as Mirafi HP370) may be used to aid in stabilization of the subgrade.

Over-excavations should be backfilled with Structural Fill material placed and compacted in accordance with the **Fill Material Types** and **Fill Compaction Requirements** of this report. Subgrade preparation and selection, placement, and compaction of Structural Fill should be performed under engineering observation in accordance with the project specifications.

Fill Material Type

All fill materials should be inorganic soils free of vegetation, debris, and fragments larger than four inches in size. Pea gravel or other similar non-cementitious, poorly-graded materials should not be used as fill or backfill without the prior approval of the geotechnical engineer.

Clean on-site soils or approved imported materials may be used as fill material for the following:

Structural Fill Type [1,3]	Specifications	Acceptable Parameters (for Structural Fill)
Common Fill	OSSC Section 00300.13 Selected General Backfill	All locations across the site. Dry weather only acceptable
Select Fill	OSSC Section 00330.14 Selected Granular Backfill [2]	All locations across the site. Wet and Dry weather acceptable.
Crushed Aggregate Base	OSSC Section 02630.10 Dense Graded Aggregate (2"-0 to ¾"-0) [2]	All locations across the site. Wet and Dry weather acceptable.
Trench Backfill	OSSC Section 00405.14 for Trench Backfill with additional stipulations [4]	Acceptable materials include Common and Select Fill listed above.
Utility Subbase Stabilization	OSSC Section 00330.14 for Selected Granular Backfill above groundwater seepage and OSSC Section 00330.16 for Stone Embankment Material with additional stipulations [4]	12-inch compacted lift in wet or soft subgrades encountered in trench base and other utility excavations.
Bedding & Haunching	OSSC Section 00405.13, Pipe Zone Material	Thickness above and below pipe recommended by Electrical Engineer

1. Controlled, compacted fill should consist of approved materials that are free (free = less than 3% by weight) of organic matter and debris (i.e. wood sticks greater than ½ inch in diameter). A sample of each material type should be submitted to the geotechnical engineer for evaluation.
2. Material should have a maximum aggregate size of 2 inches, and a minimum laboratory CBR of 20% for granular soils, and no more than 12% passing the No. 200 sieve by weight determined by ASTM D6913. Fines should have a Plasticity Index (PI) of less than 20% per ASTM D4318. Reclaimed glass will not be accepted.
3. The contractor shall select the appropriate material for use based on the current and forecasted weather conditions at the time of construction.
4. Maximum aggregate size shall be limited to 2½ inches.

Fill should be placed and compacted in horizontal lifts, not exceeding 10 inches loose thickness, using equipment and procedures that will produce recommended densities throughout the lift.

Compaction Requirements

Recommended compaction and moisture content criteria for engineered fill materials are as follows:

Item	Structural Fill
Fill Lift Thickness	10-inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack, plate compactor, etc.) is used
Compaction Requirements [1]	Native Scarified and Recompacted Subgrades: 95% of ASTM D698; Structural Fill (Granular) Materials: 95% of ASTM D1557 AC Utility Trench Materials: 85% of ASTM D698 [3]
Moisture Content Material [2]	Workable moisture levels

1. We recommend that engineered fill be tested for moisture content and compaction during placement. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested as required until the specified moisture and compaction requirements are achieved. Compaction levels are relative to the soils modified proctor maximum dry density (ASTM D1557).
2. Specifically, moisture levels should be maintained low enough to allow for satisfactory compaction to be achieved without the cohesionless fill material pumping when proofrolled.
3. Compaction percentage to be confirmed and provided by Electrical Engineer of Record.

Road surface rock, if used, should be seated in-place by vibratory smooth drum roller. Terracon should be contacted to review exposed subgrade conditions prior to placing fill.

Grading and Drainage

Adequate drainage should be provided at the site to reduce the likelihood of an increase in moisture content of the foundation soils. The site should be graded to shed water and avoid ponding over the subgrade.

Earthwork Construction Considerations

It is anticipated that shallow excavations for the proposed construction can be accomplished with conventional earthmoving equipment.

Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction of the access roads. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and re-compacted prior to access road construction.

The individual contractors are responsible for designing and constructing stable, temporary excavations (including utility trenches) as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards.

Terracon should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation; proof-rolling; placement and compaction of controlled compacted fills; backfilling of excavations to the completed subgrade.

Construction Observation and Testing

The exposed subgrade and each lift of compacted fill should be tested, evaluated and reworked, as necessary, until approved by the geotechnical engineer's representative prior to placement of additional lifts of fill. We recommend that each lift of fill be tested for density and moisture content at a minimum frequency of one test for every 5,000 square feet of compacted fill in the structure areas. We recommend one density and moisture content test for every 300 linear feet of compacted utility trench backfill. If engineered fill is placed beneath individual structures, we recommend at least one density and moisture content test per each vertical lift per structure.

Terracon should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation; proofrolling; placement and compaction of controlled compacted fills; backfilling of excavations into the completed subgrade, and just prior to construction of building floor slabs.

ACCESS ROADWAYS

Aggregate Surface Roadway Design Recommendations

We understand that new roadways within the project site will consist of aggregate surfaced roadways. We understand the roadways may be subjected to fire truck loading. Design truck load frequencies during construction and post-construction have not been provided. Aggregate Roadway sections based upon a more detailed design could be provided if specific traffic loading, frequencies, and desired design life are provided.

Subgrade soils beneath aggregate surfaced roadways should be prepared and constructed as outlined in the **Subgrade Preparation** section of this report.

An analysis of the proposed 8-inch thick aggregate surfaced pavement section was performed as outlined in the 1993 AASHTO Design of Pavement Structures for aggregate-surfaced roads (Section 4.1.2). The design analysis evaluates both the allowable rutting depth and allowable serviceability loss as design considerations. For the analyses, an allowable rutting depth of 2 inches and a serviceability loss of 3.5 were used.

The subgrade soils classification used for the analyses is based on the American Association of State Highway and Transportation Officials (AASHTO) soil classification system. Based on the results of the laboratory tests, the soil classification class of A-7-5 was used for the analyses. A CBR of 4 was used for this analysis based on our laboratory testing and experience onsite during wet weather conditions.

Based on the subgrade conditions and an allowable rutting depth of 2 inches, an aggregate-surfaced roadway section consisting of a minimum 8-inch thickness of compacted aggregate base course placed over prepared and compacted subgrade could support approximately 5,000 Equivalent Single Axle Loads (ESALs) over the design life with proper maintenance and adequate surface drainage. Periodic maintenance of the aggregate surface should be anticipated, particularly in high traffic and turning areas or the aggregate surfaced pavement may need to be reconstructed after exceeding 5,000 ESALs of traffic.

To reduce rutting, increase traffic loads, reduce maintenance costs, reduce serviceability loss and reduce roadway dust a geotextile can be utilized at the base of the crushed aggregate surface. Additionally, the use of the geotextile could reduce the gravel thickness, depending on the anticipated traffic. We believe the use of Mirafi's RS280i geotextile could provide the described benefits, as well as reduce the gravel thickness by 2 inches with the defined traffic loading conditions. Additionally, we believe this could provide a cost savings based on the difference in cost between the gravel and geotextile. This is inherently dependent on amount of lineal roadway and delivery and product costs.

A concern regarding the use of permeable aggregate materials in large pavement areas is that surface water cannot be drained over the surface before it permeates through the aggregate surfacing, which would create a condition where the subgrade soils increase moisture content. If the subgrade soils do become elevated in moisture content, the overall performance of the aggregate surfaced pavement areas will be reduced and could result in excessive rutting and may require maintenance or reconstruction of the gravel surface pavement. To help direct surface water over the aggregate surface, we suggest surface slopes of 2% to 3% be constructed and maintained. 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.

An additional concern is the development of seasonal ponding onsite during wet-weather. At this time we do not know the anticipated depths of these ponded areas but anticipated additions to the gravel section could be required to have the driving surface exposed. Site specific topographic maps may be able to define these areas of concern.

Access Roadway Design and Construction Considerations

The roadway subgrade, if prepared early in the project, should be carefully evaluated as the time for construction approaches. We recommend the roadway area be stripped of existing topsoil/organic subsoil, or otherwise unsuitable material, rough graded, and compacted with a heavy roller compactor without vibration, before being proof-rolled with a loaded tandem-axle dump truck. Particular attention should be paid to high traffic areas that were rutted and disturbed, and areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by replacing the materials with properly compacted fill. When proof-rolling/subgrade stabilization has been completed to the satisfaction of Terracon, the geotextile fabric and/or gravel fill may be placed.

Aggregate surfaced drives, regardless of the section thickness or subgrade preparation measures, will require on-going maintenance and repairs to keep it in a serviceable condition. It is not practical to design a gravel section of sufficient thickness that on-going maintenance will not be required. This is due to the porous nature of the gravel that will allow precipitation and surface water to infiltrate and soften the subgrade soils, and the limited near surface strength of unconfined gravel that makes it susceptible to rutting. When potholes, ruts, depressions or yielding subgrades develop, they must be addressed as soon as possible in order to avoid major repairs.

Maintenance should consist of periodic grading with a road grader. Typical repairs could consist of placing additional gravel in ruts or depressed areas. Potholes and depressions should not be filled by blading adjacent ridges or high areas into the depression areas. New material should be added to the depressed areas as they develop.

RECOMMENDATIONS FOR FURTHER INVESTIGATION

Based on experience on similar sized projects and the site-specific soils conditions and preliminary testing we recommend the following additional explorations be conducted to define a full-scale/final Geotechnical Engineering investigation and design parameters.

Additional explorations and testing required for full-scale/final Geotechnical Engineering Report:

- One (1) exploration (boring, CPT, or test pit) per 25 acres of array area
- Two (2) to three (3) explorations (boring, CPT or test pit) per substation
- One (1) Pile Load Test location per 50 acres of array area
- One (1) laboratory corrosion suite per exploration location
- One (1) Field Electrical Resistivity test location per 100 acres of array area
- One (1) Field Electrical Resistivity test location per substation
- One (1) laboratory Thermal resistivity dry-out test per 100 acres of array area

- Two (2) to three (3) laboratory California Bearing Ratio (CBR) tests for access roadway design

The array area is not yet defined therefore we cannot define the total number of additional explorations required. Once the array area has been defined we can provide a "gap" analysis to define the number of explorations and test required for a full-scale/final Geotechnical Engineering design.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

The field exploration on the project consisted of the following exploration plan. The approximate boring and CPT locations are shown on the **Exploration Plan**.

Number of Explorations	Type of Exploration	Exploration No.	Exploration Depth (feet)	Planned Location
14	Boring	B-1 to B-14	1 to 21½	Proposed Array Area
4	CPT	CPT-1 to CPT-4	2 to 20½	Proposed Array Area
4	Test Pits	TP-1 to TP-4	10	Proposed Array Area
12	Pile Installation	PLT-1 to PLT-4	5 to 8	Proposed Array Area

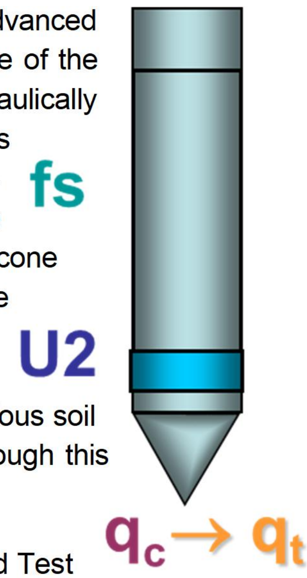
Exploration Layout and Elevations: Unless otherwise noted, Terracon personnel provided the exploration layout. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±10 feet) and approximate elevations were obtained by interpolation from Google Earth Pro. If elevations and a more precise exploration layout are desired, we recommend explorations be surveyed following completion of fieldwork.

Subsurface Exploration Procedures: We advanced soil borings with a trailer-mounted drill rig using continuous flight solid stem augers. Four samples are obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. Soil sampling is typically performed using thin-wall tube and/or split-barrel sampling procedures. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the soil to obtain a relatively undisturbed sample. In the split barrel sampling procedure, a standard 2-inch outer diameter split barrel sampling spoon is driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. The samples were placed in appropriate containers, taken to our soil laboratory for testing, and classified by a geotechnical engineer. In addition, we observed and recorded groundwater levels during drilling and sampling.

All explorations were supervised and logged by a field engineer to record field test data, classify soils, and to collect the samples from the explorations. Our exploration team prepared field boring logs as part of standard drilling operations including sampling depths, penetration distances, and other relevant sampling information. Field logs include visual classifications of materials encountered during drilling, and our interpretation of subsurface conditions between samples.

Final boring logs, prepared from field logs, represent the geotechnical engineer's interpretation, and include modifications based on observations and laboratory tests.

Cone Penetrometer Test Explorations: Four (4) CPT explorations were advanced with a track mounted rig under subcontract to Terracon. A continuous profile of the subsurface is obtained to the termination depth of the CPT. The CPT hydraulically pushes an instrumented cone through the soil while nearly continuous readings are recorded to a portable computer. The cone is equipped with electronic load cells to measure tip resistance and sleeve resistance and a pressure transducer to measure the generated ambient pore pressure. The face of the cone has an apex angle of 60° and an area of 10 cm^2. Digital data representing the tip resistance, friction resistance, pore water pressure, and probe inclination angle are recorded while advancing through the ground at a rate between 1½ and 2½ centimeters per second. These measurements are correlated to various soil properties used for geotechnical design. No soil samples are gathered through this subsurface investigation technique.



CPT testing is conducted in general accordance with ASTM D5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils." Seismic shear wave velocity testing was also performed in the CPT explorations.

Upon completion, the data collected were downloaded and processed by the project engineer.

Test Pits Explorations: A geotechnical engineer logged the excavations and collected bulk grab soil samples. The test pits were completed on January 22, 2020 up to the depths described above. The test pits were excavated using a tracked excavator under subcontract to our firm. The test pits areas were backfilled with the excavated materials and tamped with the bucket as it is placed.

Resistivity Testing: Four field soil electrical resistivity tests were performed by two Terracon personnel on February 12 and 13, 2020, in general accordance with ASTM G57 using the four-pin Wenner method with a MiniRES ULTRA earth resistivity meter.

Pile Installation: Twelve, three at each of the four locations, W6x9 piles were installed by Sunstall as directed by Terracon. Piles were advanced on January 21, 2020 with a track mounted GAYK Model HRE 4000 equipped with a hydraulic hammer.

Laboratory Testing

The project engineer reviewed field data and assigned various laboratory tests to better understand the engineering properties of various soil strata. Procedural standards noted below are for reference to methodology in general. In some cases, local practices and professional judgement require method variations. Standards noted below include reference to other related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D1140 Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing
- ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort

The laboratory testing program included examination of soil samples by an engineer. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System.

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Archway Solar Project ■ Christmas Valley, OR
February 24, 2020 ■ Terracon Project No. 82185058

Terracon GeoReport

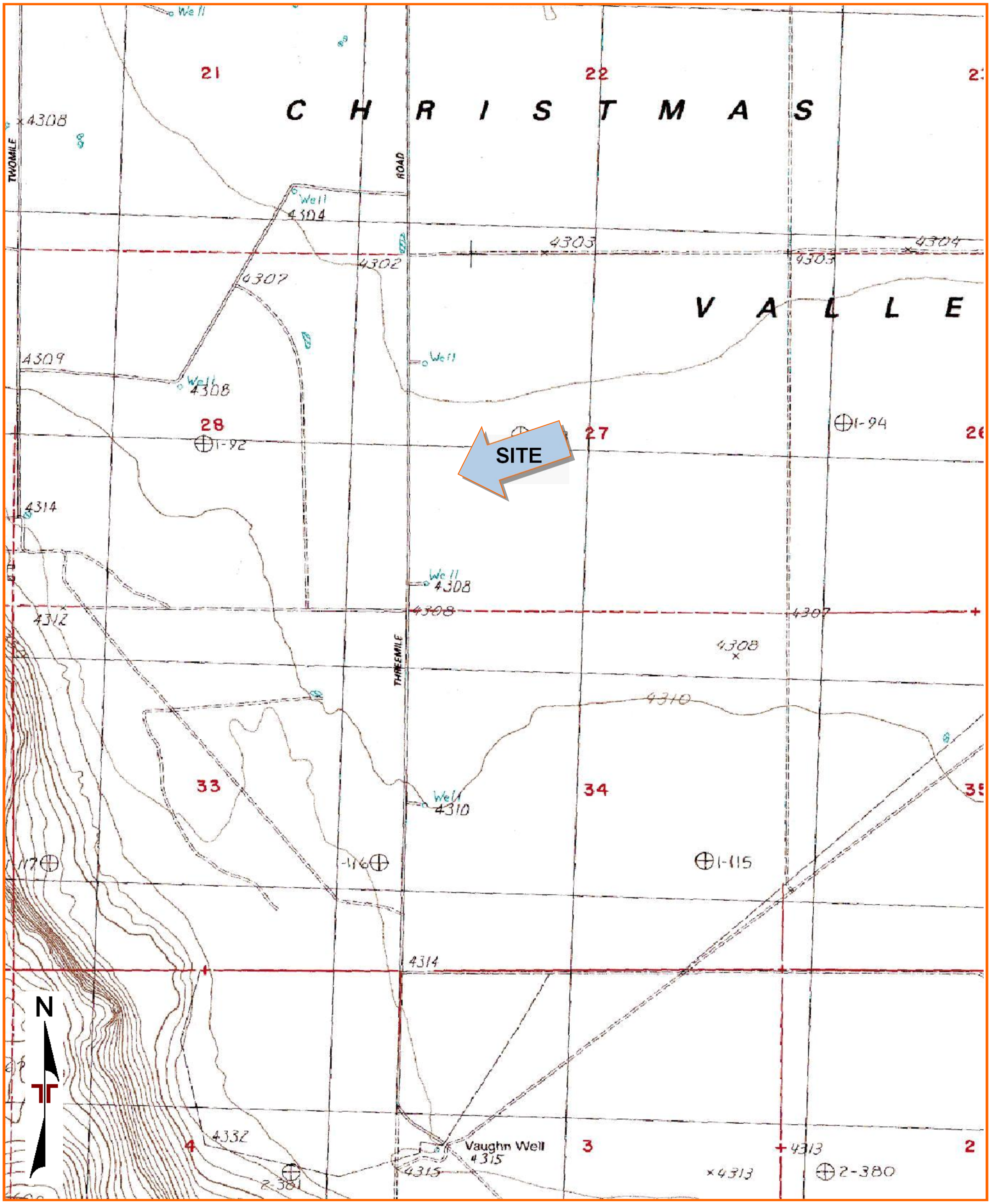
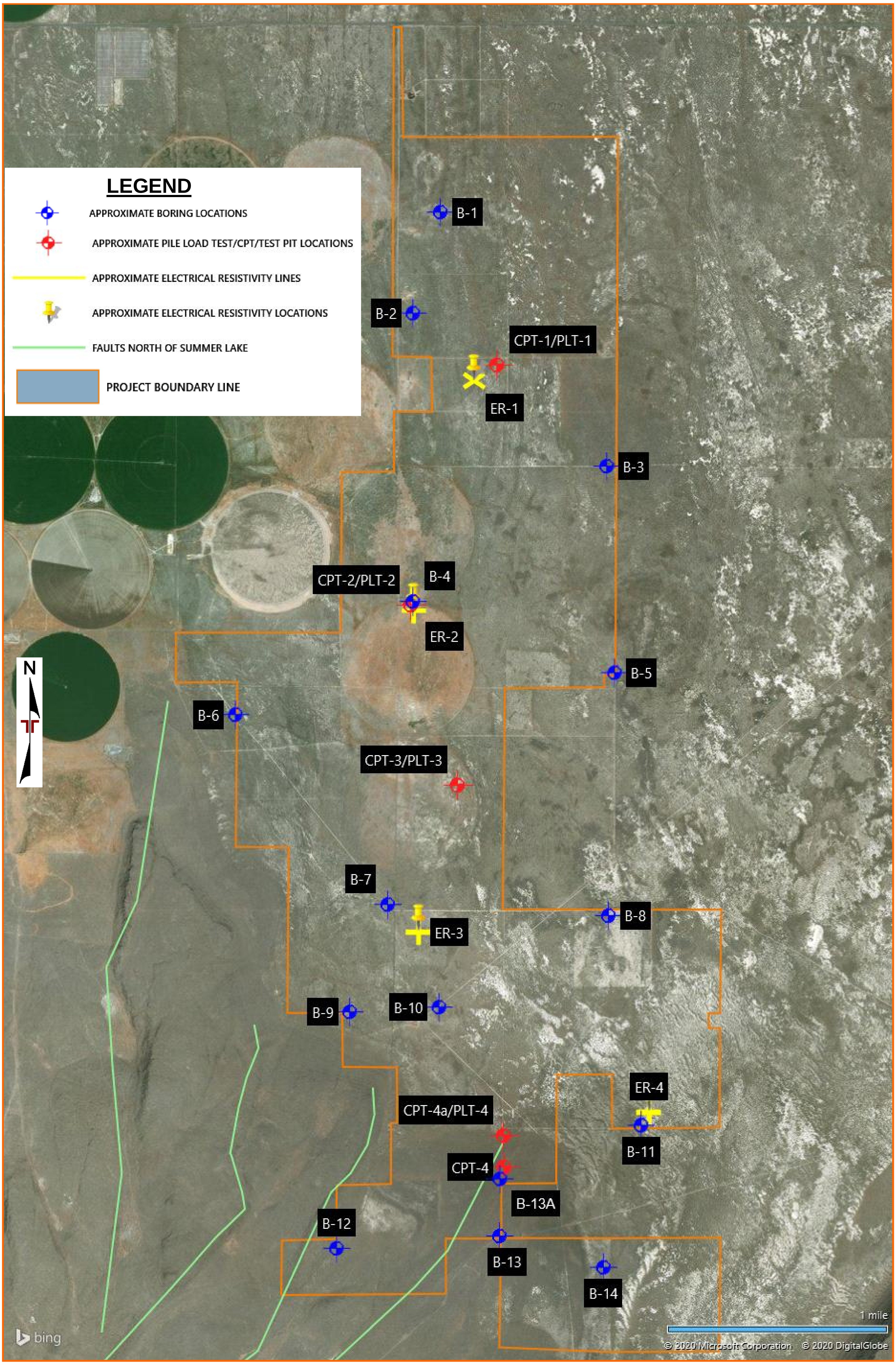


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY QUADRANGLES INCLUDE: VAUGHN WELL, OR (1/1/1986).


LEGEND
APPROXIMATE BORING LOCATIONS
APPROXIMATE PILE LOAD TEST/CPT/TEST PIT LOCATIONS
APPROXIMATE ELECTRICAL RESISTIVITY LINES
APPROXIMATE ELECTRICAL RESISTIVITY LOCATIONS
FAULTS NORTH OF SUMMER LAKE
PROJECT BOUNDARY LINE
N
B-1
B-2
CPT-1/PLT-1
ER-1
B-3
CPT-2/PLT-2
B-4
ER-2
B-5
B-6
CPT-3/PLT-3
B-7
ER-3
B-8
B-9
B-10
ER-4
CPT-4a/PLT-4
B-11
CPT-4
B-13A
B-12
B-13
B-14
1 mile
bing
© 2020 Microsoft Corporation © 2020 DigitalGlobe

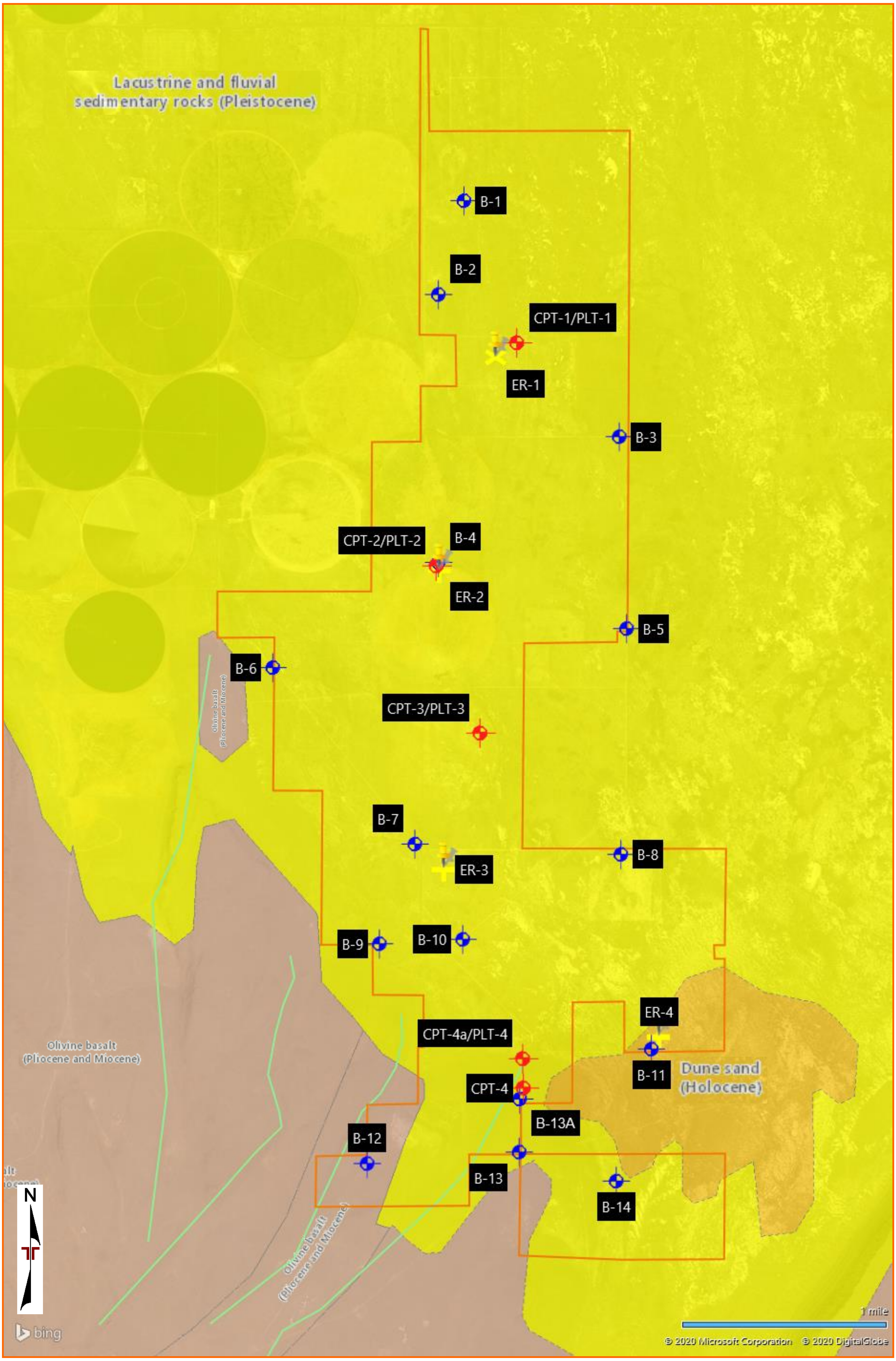


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

Photo 1: General site ground cover

Photo 2: Drilling activities at project site

Photo 3: Truck-mounted drill rig used for exploration

Photo 4: Drilling activities at project site

Photo 5: Installation of ZL6 data logger from METER Group

Photo 6: Electrical resistivity testing activities

Photo 7: Electrical resistivity testing around area near boring B-11

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Archway Solar Preliminary Geotechnical Study ■ Christmas Valley, OR

Terracon Project No. 82185058

SAMPLING	WATER LEVEL	FIELD TESTS
Grab Sample Shelby Tube Standard Penetration Test	Water Initially Encountered Water Level After a Specified Period of Time Water Level After a Specified Period of Time Cave In Encountered Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	**N** Standard Penetration Test Resistance (Blows/Ft.) **(HP)** Hand Penetrometer **(T)** Torvane **(DCP)** Dynamic Cone Penetrometer **UC** Unconfined Compressive Strength **(PID)** Photo-Ionization Detector **(OVA)** Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

LOCATION AND ELEVATION NOTES

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELEVANCE OF SOIL BORING LOG

The soil boring logs contained within this document are intended for application to the project as described in this document. Use of these soil boring logs for any other purpose may not be appropriate.

CPT GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Archway Solar Preliminary Geotechnical Study ■ Christmas Valley, OR

Terracon Project No. 82185058

Terracon GeoReport

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$
Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
u_1 - sensor on the face of the cone
u_2 - sensor on the shoulder (more common)

Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

DESCRIPTION OF GEOTECHNICAL CORRELATIC

Normalized Tip Resistance, Q_{tn}
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
OCR (1) $= 0.25(Q_{tn})^{1.25}$
OCR (2) $= 0.33(Q_{tn})$

Undrained Shear Strength, S_u
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
N_{kt} is a soil-specific factor (shown on S_u plot)

Sensitivity, S_t
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, ϕ'
ϕ' (1) $= \tan^{-1}(0.373[\log(q_t/\sigma'_{V0}) + 0.29])$
ϕ' (2) $= 17.6 + 11[\log(Q_{tn})]$

Unit Weight, γ
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
σ_{V0} is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, G_0
G_0 (1) $= \rho V_s^2$
G_0 (2) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, I_c
$I_c = [(3.47 - \log(Q_{tn})^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT N_{60}
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, E_s (assumes $q/q_{ultimate} \sim 0.3$, i.e. FS = 3)
E_s (1) $= 2.6\psi G_0$ where $\psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
E_s (2) $= G_0$
E_s (3) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
E_s (4) $= 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

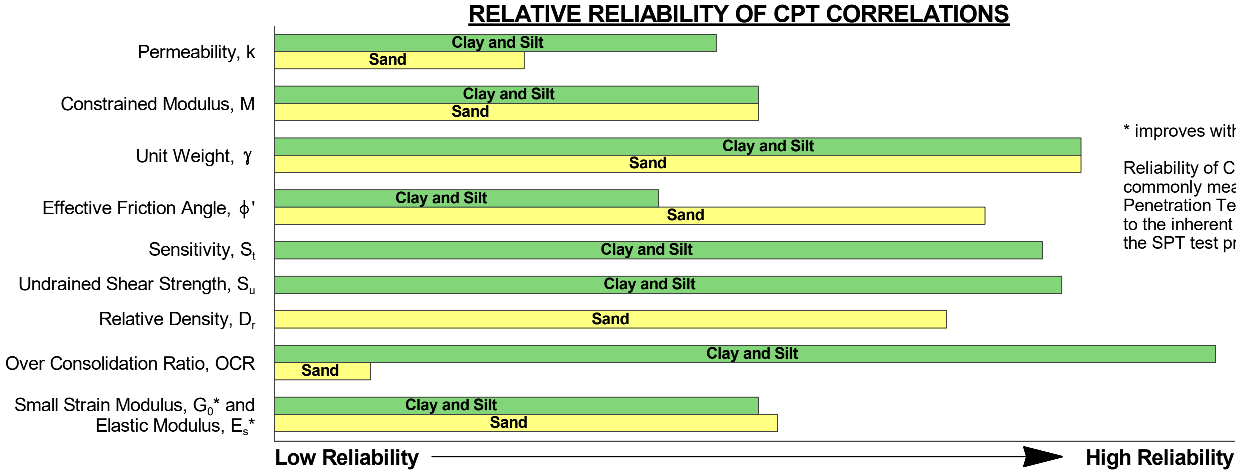
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, D_r
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t, f_s, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



* improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"

Measured - Depth to water directly measured in the field

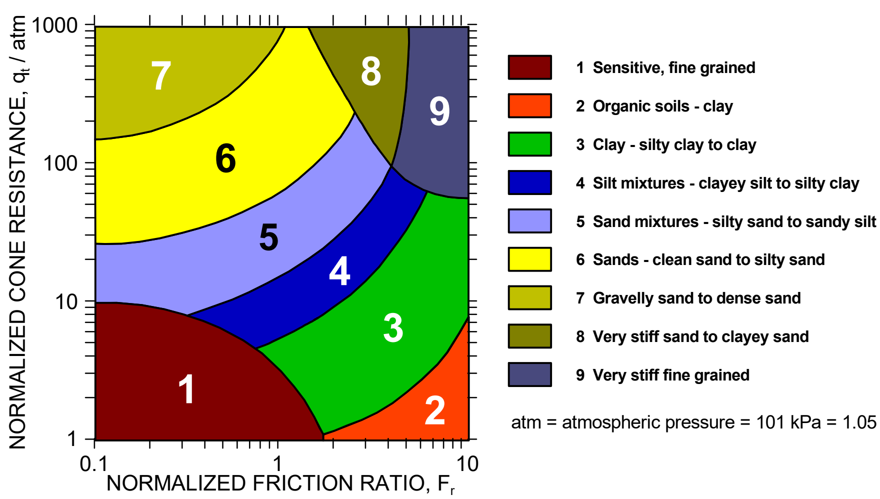
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



atm = atmospheric pressure = 101 kPa = 1.05 tsf

REFERENCES

Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.

Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.

Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.

Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu ³ 4 and 1 £ Cc £ 3 [E]	GW	Well-graded gravel [F]
			Cu < 4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ³ 6 and 1 £ Cc £ 3 [E]	SW	Well-graded sand [I]
			Cu < 6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above “A” line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below “A” line [J]	ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above “A” line	CH	Fat clay [K, L, M]
			PI plots below “A” line	MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ³ 15% sand, add “with sand” to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add “with organic fines” to group name.

[I] If soil contains ³ 15% gravel, add “with gravel” to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add “with sand” or “with gravel,” whichever is predominant.

[L] If soil contains ³ 30% plus No. 200 predominantly sand, add “sandy” to group name.

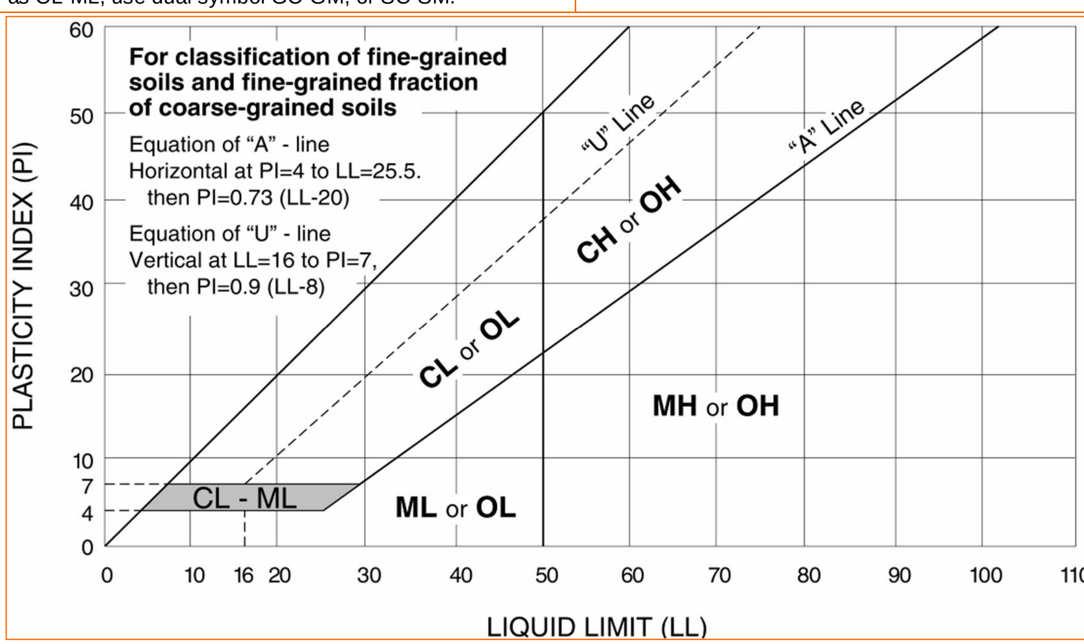
[M] If soil contains ³ 30% plus No. 200, predominantly gravel, add “gravelly” to group name.

[N] PI ³ 4 and plots on or above “A” line.

[O] PI < 4 or plots below “A” line.

[P] PI plots on or above “A” line.

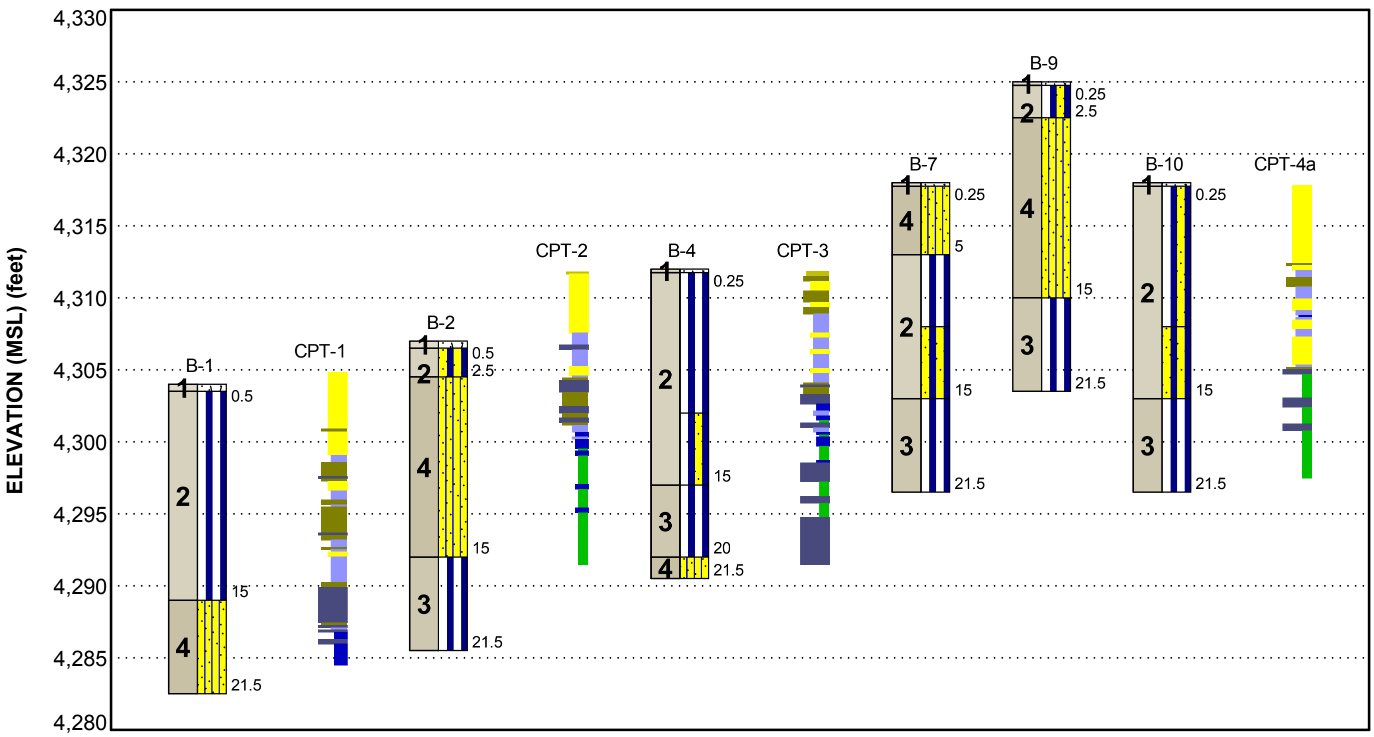
[Q] PI plots below “A” line.



GEOMODEL

Archway Solar Preliminary Geotechnical Study ■ Christmas Valley, OR
Terracon Project No. 82185058

Terracon GeoReport



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Topsoil	Rootlet zone
2	Probable Diatomaceous Earth	Elastic Silt; Elastic Silt with Sand; Sandy Elastic Silt; Silt with Sand, tan, light brown, brown, low to medium plasticity, medium stiff to hard
3	Diatomaceous Earth	Elastic Silt; Elastic Silt with Sand; tan, light brown, or white, low to medium plasticity, medium stiff to hard, moisture contents above 50%
4	Silty Sand	Silty Sand, brown, fine-grained, loose to very dense
5	Silty Gravel	Silty Gravel with Sand, brown and gray, angular, very dense

LEGEND

- Topsoil
- Sandy Elastic Silt
- Elastic Silt
- Elastic Silt with Sand
- Silty Sand

Soil Behavior Type (SBT)

- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravelly sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

CPT Assumed Water Depth

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project. Numbers adjacent to soil column indicate depth below ground surface.

BORING LOG NO. B-1

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.2289° Longitude: -120.4436°

Surface Elev.: 4304 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.5	**TOPSOIL**, Rootlet zone - 6 inches	4303.5								
2			**ELASTIC SILT (MH)**, trace sand, light brown, stiff, probably diatomaceous earth				X	3-6-6 N=12	26			
							X	5-5-8 N=13	27			86
			very stiff, increased sand content		5		X	10-10-11 N=21	21			
			intermittent layers of silty sand and sandy silt				X	9-13-17 N=30	18			
			tan, hard, moderate cementation		10		X	10-14-27 N=41	24			
		15.0		4289	15							
4			**SILTY SAND (SM)**, fine grained, brown and tan, medium dense, weak cementation				X	8-6-7 N=13	24			43
					20		X	9-10-15 N=25	40			
		21.5		4282.5								

Boring Terminated at 21.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.
Difficult drilling from 5.5 to 7.5 feet.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-2

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.2109° Longitude: -120.4671°

Surface Elev.: 4307 (Ft.)

MODEL LAYER	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.5	**TOPSOIL**, Rootlet zone - 6 inches	4306.5								
2	2.5	**SANDY ELASTIC SILT (MH)**, brown, soft to medium stiff, probable diatomaceous earth, trace mica	4304.5			X	4-2-2 N=4	36			
4		**SILTY SAND (SM)**, fine grained, brown, loose, trace mica, intermittent layers of silty sand and sandy silt				X	3-2-4 N=6	38			
		medium dense		5		X	7-7-12 N=19	28			17
		2 inch silt lense				X	6-6-6 N=12	23			
	15.0	brown	4292	10		X	5-6-7 N=13	31			
3		**ELASTIC SILT (MH)**, brown, very stiff, moderate cementation, diatomaceous earth, blocky		15		X	9-11-18 N=29	53		76-59-17	
	21.5		4285.5	20		X	5-8-13 N=21	48			

Boring Terminated at 21.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-3

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.2083° Longitude: -120.4439°

Surface Elev.: 4307 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.5	**TOPSOIL**, Rootlet zone - 6 inches	4306.5								
2		**ELASTIC SILT WITH SAND (MH)**, tan, hard, probable diatomaceous earth, slightly cemented				X	6-18-23 N=41	28			
						X	11-13-22 N=35	28		61-36-25	78
		very stiff, increased sand content		5		X	10-12-12 N=24	21			
		intermittent layers of silty sand and silt with sand				X	9-11-15 N=26	20			
				10		X	7-10-10 N=20	23			
		intermittent layers of silty sand and silt with sand		15		X	6-7-11 N=18	36			
	21.5		4285.5	20		X	11-13-16 N=29	27			
		Boring Terminated at 21.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-02-2020	Boring Completed: 01-02-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-4

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1994° Longitude: -120.4614°

Surface Elev.: 4312 (Ft.)

MODEL LAYER	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4312								
2		**ELASTIC SILT (MH)**, trace sand, light brown, stiff, probable diatomaceous earth					4-5-5 N=10	13			
		medium stiff to stiff					6-4-4 N=8	48		66-47-19	92
				5					88		
		stiff, weak cementation					2-4-6 N=10	27			
		very stiff, intermittent layers of silty sand and sandy silt					7-11-19 N=30	28			
	10.0		4302	10							
		ELASTIC SILT WITH SAND (MH), brown, medium stiff to stiff, intermittent layers of sandy silt and silty sand					3-3-5 N=8	32			71
	15.0		4297	15							
3		**ELASTIC SILT (MH)**, tan with white veins, stiff to very stiff, weak cementation, diatemaceous earth, blocky					5-6-9 N=15	63			
	20.0		4292	20							
4		**SILTY SAND (SM)**, fine grained, brown, medium dense					7-8-15 N=23	32			
	21.5		4290.5								
		Boring Terminated at 21.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-5

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1938° Longitude: -120.4435°

Surface Elev.: 4311 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.5	**TOPSOIL**, Rootlet zone - 6 inches	4310.5								
		ELASTIC SILT WITH SAND (MH), tan, very stiff, slightly cemented, blocky, probable diatamaceous earth				X	3-8-8 N=16	34			
						X	6-10-16 N=26	34			75
				5		X	16-10-9 N=19	15			
2	7.5	**SANDY ELASTIC SILT (MH)**, tan, hard	4303.5			X	11-15-24 N=39	14			61
		medium stiff		10		X	4-3-4 N=7	35			
3	15.0	**ELASTIC SILT (MH)**, tan, stiff to very stiff, diatomaceous earth	4296	15		X	9-7-8 N=15	79			
				20		X	7-6-10 N=16	51			
	21.5	***Boring Terminated at 21.5 Feet***	4289.5								

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.
Difficult drilling at 7.5 feet.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-02-2020	Boring Completed: 01-02-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-6

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.192° Longitude: -120.4774°

Surface Elev.: 4318 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4318								
		ELASTIC SILT WITH SAND (MH), brown with white veins, stiff, weak cementation, probable diatomaceous earth				X	5-5-4 N=9	27			
						X	3-5-5 N=10	40			
2	5.0	**ELASTIC SILT (MH)**, brown and tan, medium stiff to stiff	4313	5		X	4-4-4 N=8	40		58-40-18	
		very stiff				X	7-8-11 N=19	41			
3	10.0	**ELASTIC SILT (MH)**, light brown, medium stiff, diatomaceous earth, blocky	4308	10		X	3-3-3 N=6	157			
		light brown, soft, less sand content		15		X	1-1-2 N=3	122			
	21.5		4296.5	20		X	3-1-2 N=3	74			

Boring Terminated at 21.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-7

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1795° Longitude: -120.4636°

Surface Elev.: 4318 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4318								
4		**SILTY SAND (SM)**, fine grained, brown, medium dense				SPT	5-6-9 N=15	24			
						Grab					
		dense, weak cementation				SPT	16-17-17 N=34	31			
	5.0		4313	5		Ring			81		
2		**ELASTIC SILT (MH)**, trace sand and gravel, brown, medium stiff, probable diatomaceous earth				SPT	5-4-3 N=7	38			94
		hard				SPT	13-20-16 N=36	28			
	10.0		4308	10							
		SANDY ELASTIC SILT (MH), brown, hard				SPT	15-42-50/4"	36			52
	15.0		4303	15							
3		**ELASTIC SILT (MH)**, trace sand, light brown, stiff, diatomaceous earth, blocky				SPT	4-5-8 N=13	125			
		medium stiff		20		SPT	4-3-3 N=6	134		137-81-56	
	21.5		4296.5								

Boring Terminated at 21.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-8

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1791° Longitude: -120.4342°

Surface Elev.: 4313 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.5	**TOPSOIL**, Rootlet zone - 6 inches	4312.5								
2		**ELASTIC SILT WITH SAND (MH)**, tan, hard, probable diatomaceous earth, slightly cemented, small grains hard to break up with finger				X	13-14-22 N=36	25			
						X	15-25-33 N=58	28		62-39-23	
				5		X	15-23-35 N=58	30			
	7.5		4305.5								
4		**SILTY SAND (SM)**, fine grained, brown, very dense				X	27-48-50/5"	11			
		dense, intermittent layers of silty sand and silt with sand		10		X	15-19-20 N=39	22			
	15.0		4298								
3		**ELASTIC SILT WITH SAND (MH)**, tan, hard, blocky, moderate cementation, diatomaceous earth		15		X	15-21-37 N=58	47			
				20		X	11-18-24 N=42	57			
	21.5		4291.5								
		Boring Terminated at 21.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.
Frozen surface. Tough drilling throughout boring

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-03-2020	Boring Completed: 01-03-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL BACKUP OF 82185058 ARCHWAY SOLAR FIELD LOGS 1-3-2020.GPJ TERRACON_DATATEMPLATE.GDT 2/25/20

BORING LOG NO. B-9

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1721° Longitude: -120.473°

Surface Elev.: 4325 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4325								
2	2.5	**ELASTIC SILT WITH SAND (MH)**, low to medium plasticity, brown, very stiff, probable diatomaceous earth	4322.5			X	3-8-10 N=18	32			
4	15.0	**SILTY SAND (SM)**, fine grained, brown, medium dense	4310			X	9-8-11 N=19	27			
				5		X	8-9-19 N=28	34			
						X	4-7-11 N=18	29			44
				10		X	10-8-15 N=23	34			
3	21.5	**ELASTIC SILT (MH)**, white and tan, medium stiff, diatomaceous earth	4303.5	15		X	8-9-15 N=24	78			
				20		X	4-2-3 N=5	105			

Boring Terminated at 21.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-15-2019	Boring Completed: 12-15-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-10

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1702° Longitude: -120.4552°

Surface Elev.: 4318 (Ft.)

MODEL LAYER	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4318
2		**ELASTIC SILT WITH SAND (MH)**, low to medium plasticity, brown, stiff, intermittent layers of sandy silt and silty sand, probable diatomaceous earth very stiff	
2	10.0	**SANDY ELASTIC SILT (MH)**, low to medium plasticity, brown, very stiff	4308
3	15.0	**ELASTIC SILT (MH)**, low to medium plasticity, light brown, very stiff, diatomaceous earth stiff	4303
	21.5	***Boring Terminated at 21.5 Feet***	4296.5

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
			6-7-7 N=14	18			
			10-8-12 N=20	33			
5			9-7-9 N=16	34			74
			7-5-13 N=18	24			
10			7-10-18 N=28	27			58
15			5-6-11 N=17	90			
20			4-5-8 N=13	116			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-15-2019	Boring Completed: 12-15-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-11

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC, Chicago, IL

SITE: 3 Mile Road, Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1649° Longitude: -120.4413°

Surface Elev.: 4314 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	0.3	**TOPSOIL**, Rootlet zone - 3 inches	4314								
2		**SILT WITH SAND (ML)**, low to medium plasticity, tan, stiff, probable diatomaceous earth				X	3-4-6 N=10	26			
						Hand					
						X	5-5-6 N=11	18		45-28-17	87
						Tube			78		
		very stiff, intermittent layers of sandy silt and silty sand		5		X	7-10-11 N=21	24			
						X	3-9-12 N=21	34			
	10.0		4304	10							
3		**ELASTIC SILT (MH)**, low to medium plasticity, white and tan, very stiff, blocky, weak cementation, diatomaceous earth				X	7-10-14 N=24	89			
		hard, moderate cementation		15		X	15-24-50/5"	64			
		dark tan with black and orange spotting		20		X	6-9-12 N=21	103			
	21.5		4292.5								
		Boring Terminated at 21.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.
Tough drilling from 12 to 17 feet.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-03-2020	Boring Completed: 01-03-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-12

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.157° Longitude: -120.4683°

Surface Elev.: 4383 (Ft.)

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)
1	0.5	**TOPSOIL**, Rootlet zone - 6 inches	4382.5
4		**SILTY SAND (SM)**, trace gravel, fine grained, brown, dense	
		medium dense	
		intermittent layers of silty sand and sandy silt, diatomaceous layer	
	15.0	light brown, very dense	4368
2	20.0	**SANDY ELASTIC SILT (MH)**, brown, hard, moderate cementation, probable diatomaceous earth	4363
4	21.5	**SILTY SAND (SM)**, trace gravel, fine grained, light brown, very dense	4361.5
		Boring Terminated at 21.5 Feet	

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0-1.5		X	6-16-19 N=35	10			
2.5-4		X	7-9-15 N=24	21			38
5-6.5		X	7-6-9 N=15	13			
7.5-9		X	5-5-9 N=14	60			
10-11.5		X	9-25-42 N=67	8			18
15-16.5		X	15-19-29 N=48	24			60
20-21.5		X	10-19-34 N=53	6			13

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 12-14-2019	Boring Completed: 12-14-2019
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-13

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC, Chicago, IL

SITE: 3 Mile Road, Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1579° Longitude: -120.4535°

Surface Elev.: 4394 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.3	**TOPSOIL**, Rootlet zone - 3 inches. Surficial gravels and cobbles noted.	4394				5-18-30 N=48	7			15
5		2.7	**SILTY GRAVEL WITH SAND (GM)**, angular, low plasticity, brown and gray, very dense	4391.5				50-50/2"				
			Auger Refusal at 2.66 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-03-2020	Boring Completed: 01-03-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-13A

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1617° Longitude: -120.4535°

Surface Elev.: 4357 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
4		0.8	**POORLY GRADED SAND WITH GRAVEL (SP)**, fine grained, angular, brown	4356						96		
5		1.0	**POORLY GRADED GRAVEL WITH SAND (GP)**, angular, brown and gray, very dense	4356				50/3"	3			
			Auger Refusal at 1 Foot									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-03-2020	Boring Completed: 01-03-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

BORING LOG NO. B-14

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.1557° Longitude: -120.4442°

Surface Elev.: 4319 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1		0.5	**TOPSOIL**, Rootlet zone - 6 inches	4318.5								
2			**ELASTIC SILT WITH SAND (MH)**, brown, medium stiff to stiff, probable diatomaceous earth				X	3-2-3 N=5	27			
			intermittent layers of silt with sand and silty sand				X	4-5-8 N=13	19			59
		5.0		4314	5							
4			**SILTY SAND (ML)**, trace gravel, fine grained, brown, medium dense				X	7-9-13 N=22	19			37
		7.5		4311.5								
2			**ELASTIC SILT WITH SAND (MH)**, brown, hard, probable diatomaceous earth				X	9-14-23 N=37	32			
		10.0		4309	10							
3			**ELASTIC SILT (MH)**, white and tan, very stiff, diatomaceous earth				X	8-9-12 N=21	95			
			blocky, weak cementation		15		X	5-9-8 N=17	90			
			brown		20		X	6-10-12 N=22	70			
		21.5		4297.5								
			Boring Terminated at 21.5 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Rope and Cathead

Advancement Method:
Solid stem auger - 4" OD

Abandonment Method:
Boring backfilled with bentonite chips upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Boring Started: 01-03-2020	Boring Completed: 01-03-2020
Drill Rig: Big beaver	Driller: DFE
Project No.: 82185058	

CPT LOG NO. CPT-1

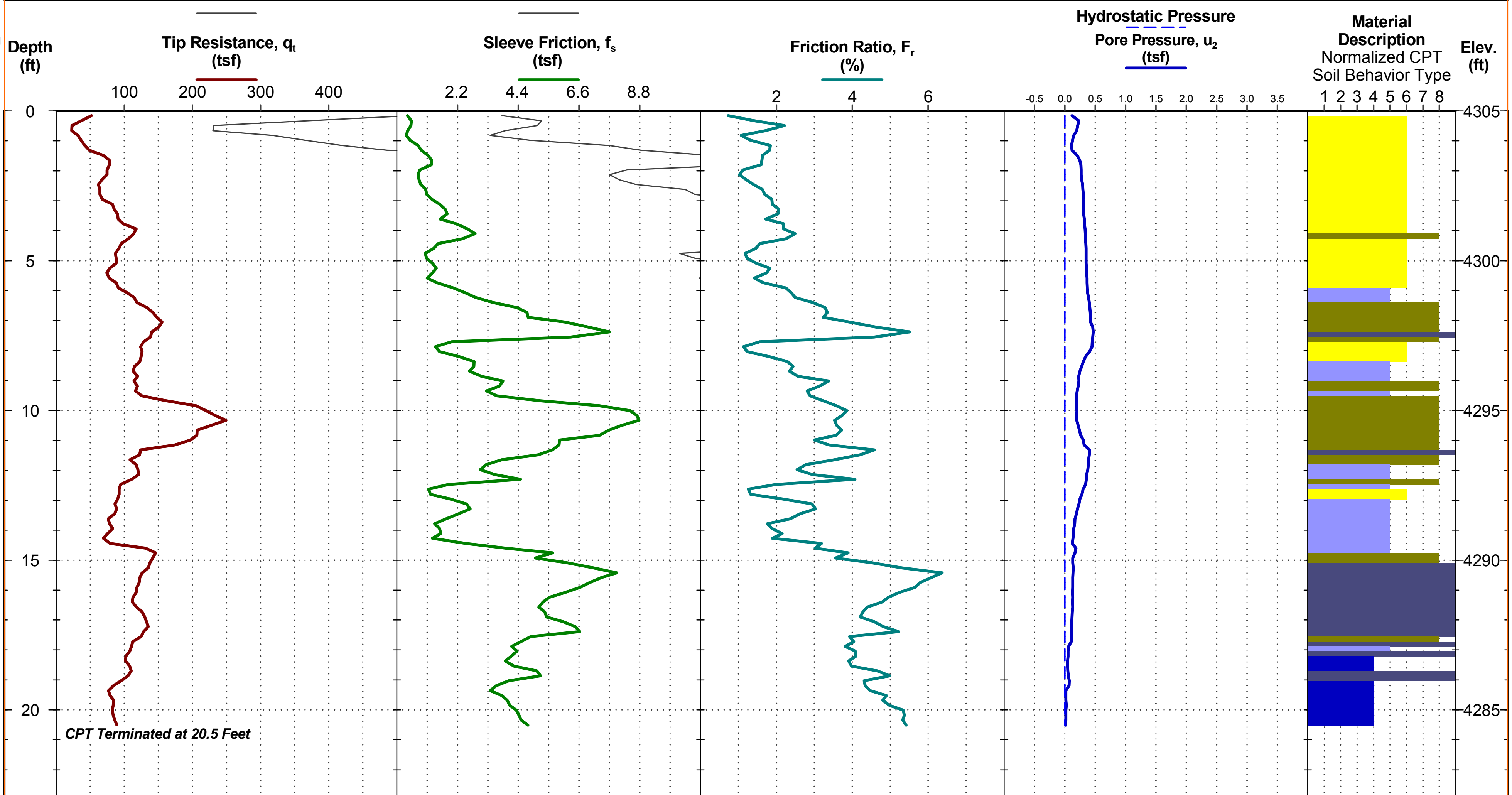
PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

TEST LOCATION: See Exploration Plan

Surface Elev.: 4305 ft
Latitude: 43.21524722°
Longitude: -120.4530139°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

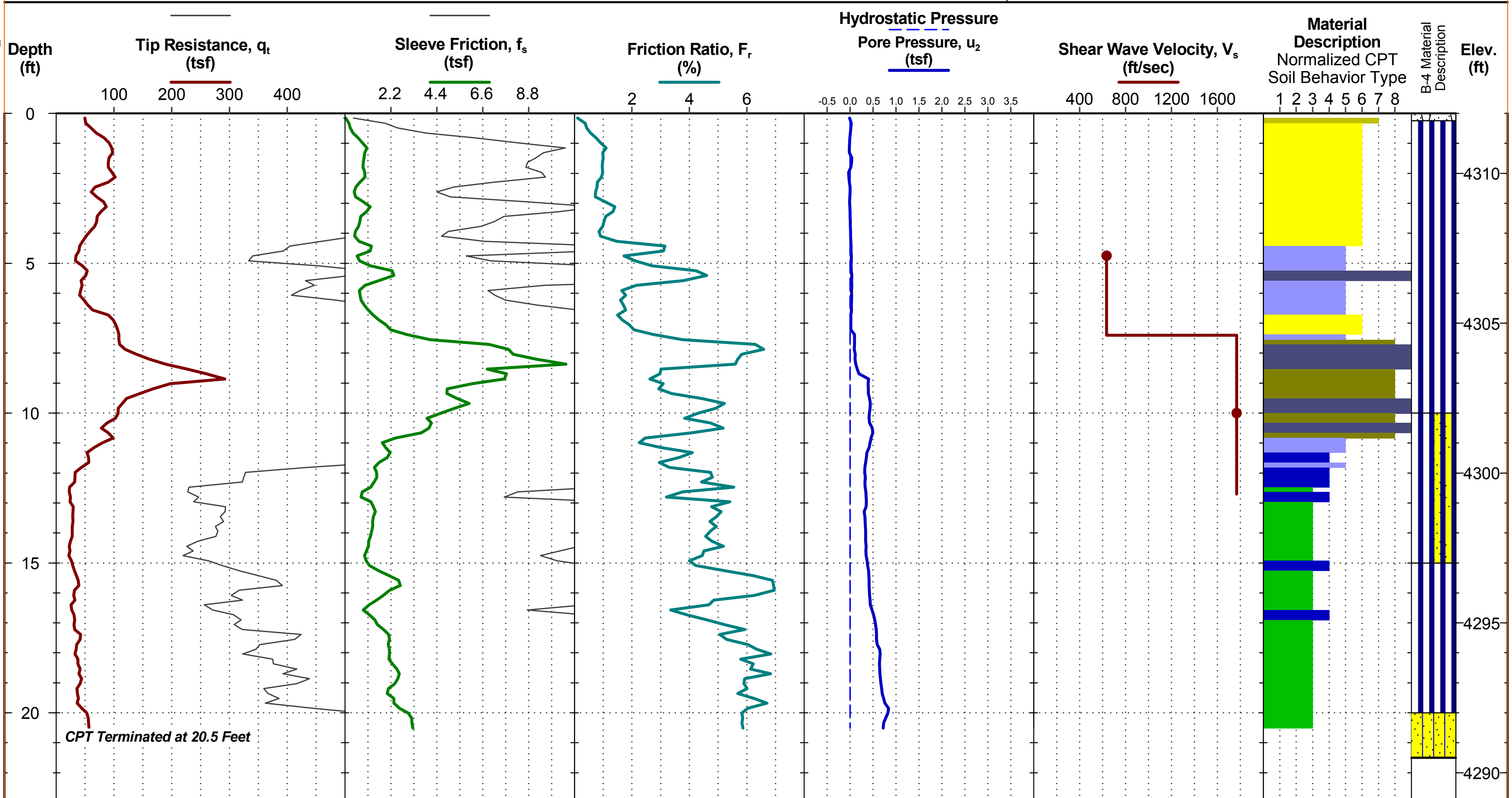
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DSG0707	Terracon 700 NE 55th Ave Portland, OR	CPT Started: 1/21/2020	CPT Completed: 1/21/2020
100 ft estimated water depth			Rig:	Operator: OGE DMM
			Project No.: 82185058	

CPT LOG NO. CPT-2

PROJECT: Archway Solar Preliminary Geotechnical Study	**CLIENT:** Invenergy LLC Chicago, IL	**TEST LOCATION:** See Exploration Plan
SITE: 3 Mile Road Christmas Valley, OR		Surface Elev.: 4312 ft Adjacent Test: B-4 Latitude: 43.19940861° Longitude: -120.4613707°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See B-4 for the adjacent test's full details.

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION 100 ft estimated water depth	Probe no. DSG0707	Terracon 700 NE 55th Ave Portland, OR	CPT Started: 1/21/2020 Rig: Project No.: 82185058	CPT Completed: 1/21/2020 Operator: OGE DMM

CPT LOG NO. CPT-3

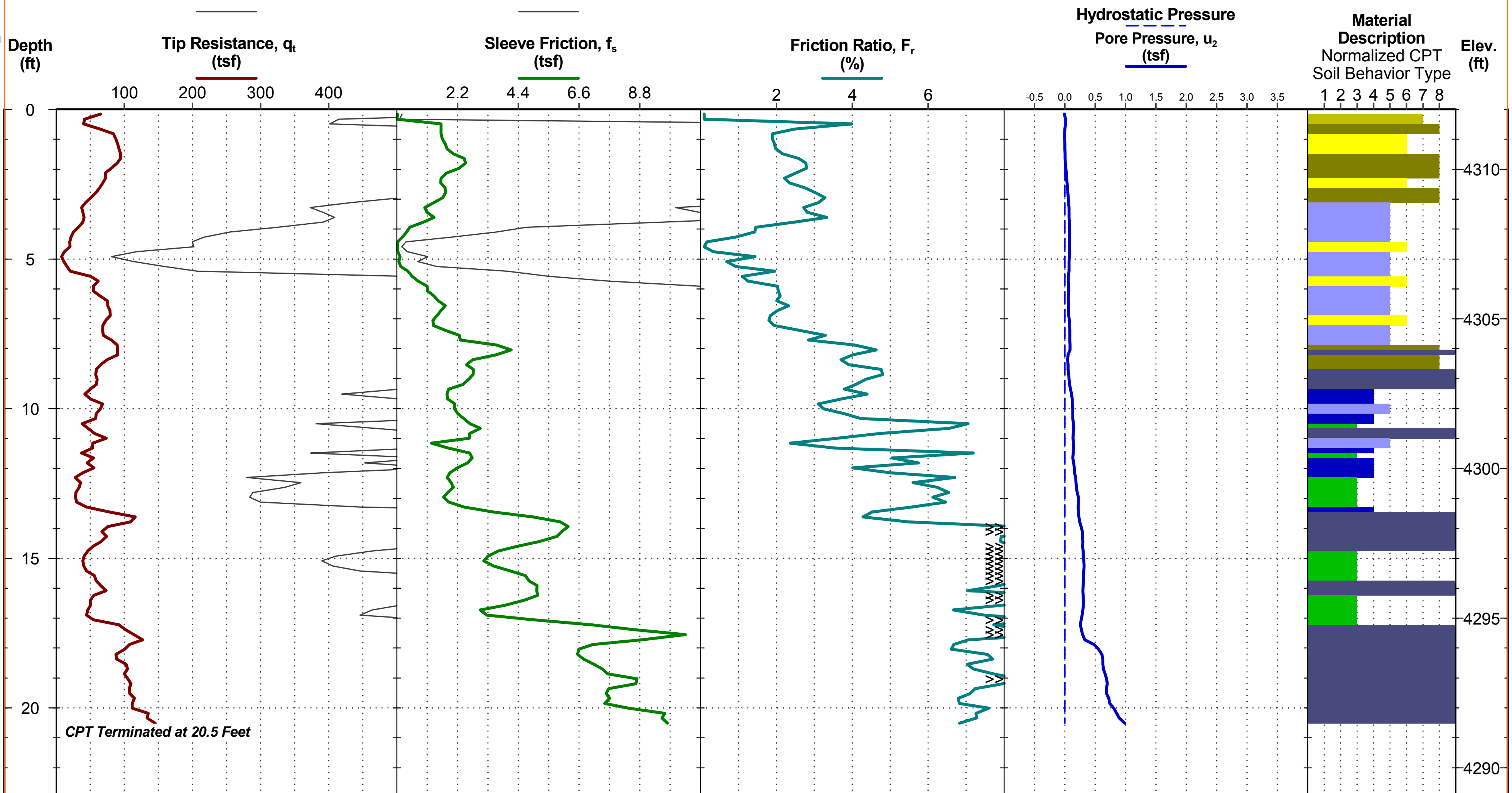
PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

TEST LOCATION: See Exploration Plan

Surface Elev.: 4312 ft
Latitude: 43.18648333°
Longitude: -120.4568806°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DSG0707
100 ft estimated water depth	

Terracon
700 NE 55th Ave
Portland, OR

CPT Started: 1/21/2020	CPT Completed: 1/21/2020
Rig:	Operator: OGE DMM
Project No.: 82185058	

CPT LOG NO. CPT-4

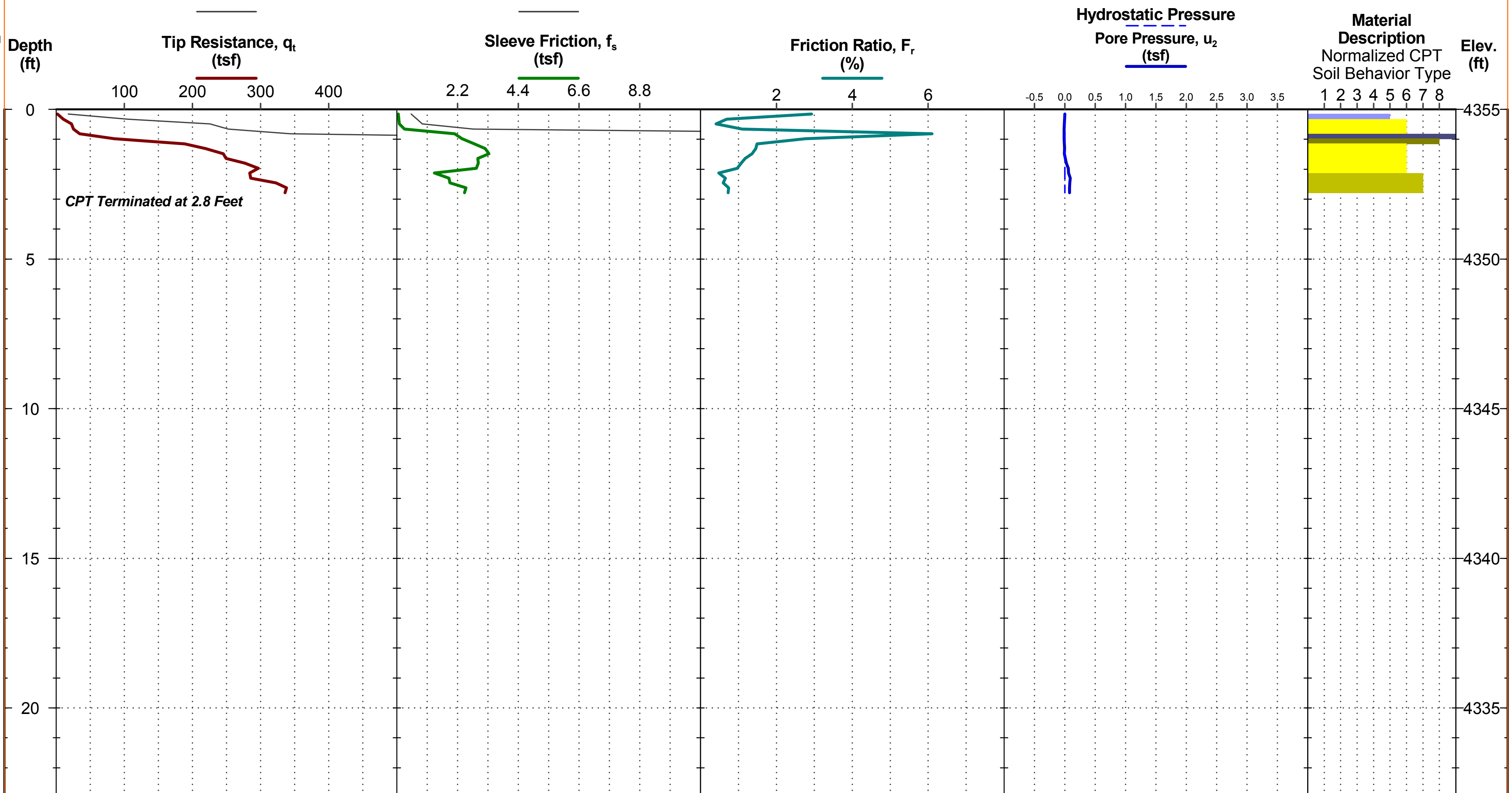
PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

TEST LOCATION: See Exploration Plan

Surface Elev.: 4355 ft
Latitude: 43.16220833°
Longitude: -120.4529667°

SITE: 3 Mile Road
Christmas Valley, OR



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DSG0707	Terracon 700 NE 55th Ave Portland, OR	CPT Started: 1/21/2020	CPT Completed: 1/21/2020
100 ft estimated water depth			Rig:	Operator: OGE DMM
			Project No.: 82185058	

CPT LOG NO. CPT-4a

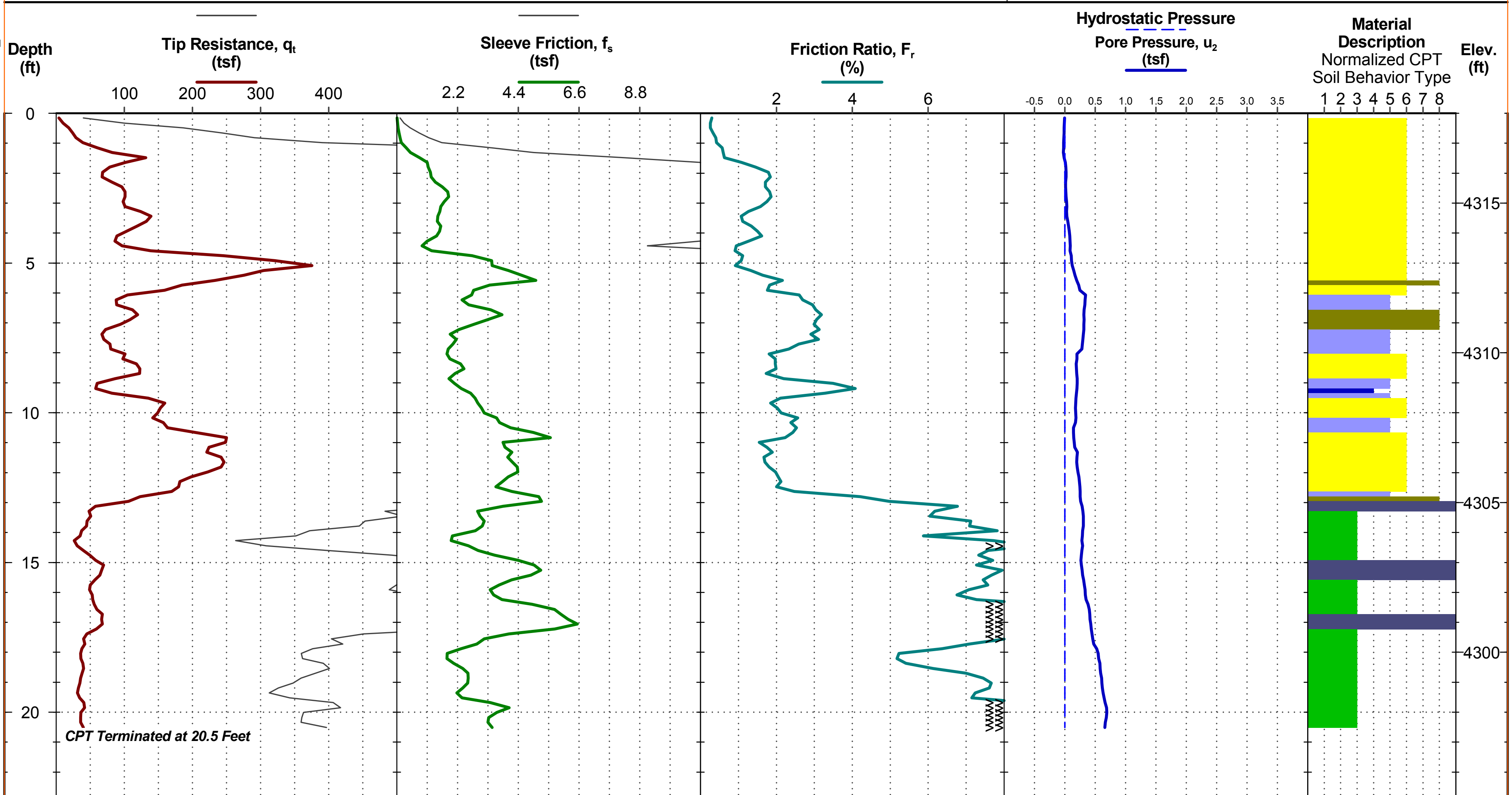
PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

TEST LOCATION: See Exploration Plan

Surface Elev.: 4318 ft
Latitude: 43.16220833°
Longitude: -120.4529667°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DSG0707	Terracon 700 NE 55th Ave Portland, OR	CPT Started: 1/21/2020	CPT Completed: 1/21/2020
100 ft estimated water depth			Rig:	Operator: OGE DMM
			Project No.: 82185058	

TEST PIT LOG NO. TP-1

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC
Chicago, IL

SITE: 3 Mile Road
Christmas Valley, OR

LOCATION See Exploration Plan

Latitude: 43.2148° Longitude: -120.4537°

Surface Elev.: 4305 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH		ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE
1		0.3	**TOPSOIL**, Rootlet zone - 3 inches	4305			
2		10.0	**ELASTIC SILT WITH SAND (MH)**, medium plasticity, light brown, probable diatomaceous earth	4295	5, 10		
			Test Pit Terminated at 10 Feet				

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
2-foot wide tooth bucket

Abandonment Method:
Test pit backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Test Pit Started: 01-30-2020	Test Pit Completed: 01-30-2020
Excavator: Track hoe	Operator: DFE
Project No.: 82185058	

TEST PIT LOG NO. TP-2

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Latitude: 43.1992° Longitude: -120.4615° DEPTH	Surface Elev.: 4312 (Ft.) ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE
1		0.3 **TOPSOIL**, Rootlet zone - 3 inches	4312			
2		**ELASTIC SILT (MH)**, medium plasticity, light brown, probable diatomaceous earth blocky sandy		5		
		10.0	4302	10		
		Test Pit Terminated at 10 Feet				

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
2-foot wide tooth bucket

Abandonment Method:
Test pit backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Test Pit Started: 01-30-2020	Test Pit Completed: 01-30-2020
Excavator: Track hoe	Operator: DFE
Project No.: 82185058	

TEST PIT LOG NO. TP-3

PROJECT: Archway Solar Preliminary Geotechnical Study

CLIENT: Invenergy LLC Chicago, IL

SITE: 3 Mile Road Christmas Valley, OR

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Latitude: 43.1873° Longitude: -120.4573° Surface Elev.: 4312 (Ft.) DEPTH / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE
1		0.3 **TOPSOIL**, Rootlet zone - 3 inches 4312			
2		**ELASTIC SILT (MH)**, medium plasticity, light brown, probable diatomaceous earth blocky sandy 10.0 4302	5 10		
		Test Pit Terminated at 10 Feet			

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
2-foot wide tooth bucket

Abandonment Method:
Test pit backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Test Pit Started: 01-30-2020	Test Pit Completed: 01-30-2020
Excavator: Track hoe	Operator: DFE
Project No.: 82185058	

TEST PIT LOG NO. TP-4

PROJECT: **Archway Solar Preliminary Geotechnical Study**

SITE: **3 Mile Road Christmas Valley, OR**

CLIENT: **Invenergy LLC Chicago, IL**

LOCATION See Exploration Plan

Latitude: 43.1644° Longitude: -120.4531°

Surface Elev.: 4327 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE
1		0.3		4327			
4			**SILTY SAND (SM)**, fine grained, brown				
		3.0		4324			
			SILTY SAND WITH GRAVEL (SM), angular, brown and gray		5		
		10.0		4317	10		
			Test Pit Terminated at 10 Feet				

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
2-foot wide tooth bucket

Abandonment Method:
Test pit backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

Notes:

Elevations were interpolated from aerial photographs using Google Earth Pro.

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
700 NE 55th Ave
Portland, OR

Test Pit Started: 01-30-2020	Test Pit Completed: 01-30-2020
Excavator: Track hoe	Operator: DFE
Project No.: 82185058	

FIELD ELECTRICAL RESISTIVITY TEST DATA

Archway Solar ■ Christmas Valley, Lake County, Oregon

February 12, 2020 ■ Terracon Project No. 82185058

Terracon GeoReport

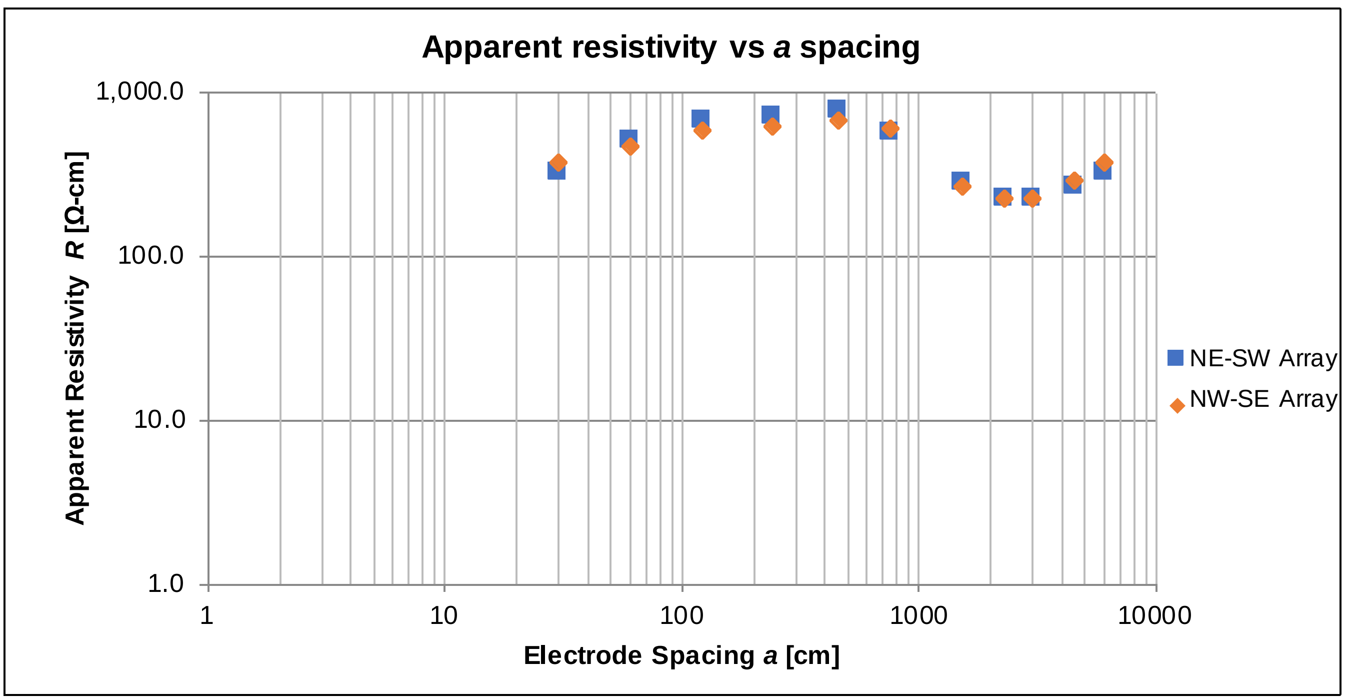
Array Loc.	Latitude/Longitude: 43.2137978, -120.4558671. Area near PLT-1, ER-1

Instrument	MiniRes	**Weather**	41° F, partly cloudy
Serial #	SN-303	**Ground Cond.**	Elastic Silt
Cal. Check	February 12, 2020 w/ Calibration Belt	**Tested By**	JAE
Test Date	February 12, 2020	**Method**	Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)

Notes & Conflicts Dense surficial sage brush present while testing. Test area was relatively flat.

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi a R}{1 + \dfrac{2a}{\sqrt{a^2 + 4b^2}} - \dfrac{a}{\sqrt{a^2 + b^2}}}$

Electrode Spacing *a*		Electrode Depth *b*		NE-SW Test		NW-SE Test	
[feet]	[centimeters]	[inches]	[centimeters]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]
1	30	6	15	1.31	330	1.54	380
2	61	6	15	1.24	520	1.14	480
4	122	6	15	0.86	680	0.77	600
8	244	6	15	0.47	730	0.41	640
15	457	6	15	0.27	790	0.24	680
25	762	6	15	0.12	590	0.13	620
50	1524	6	15	0.03	290	0.03	270
75	2286	6	15	0.02	230	0.02	230
100	3048	6	15	0.01	230	0.01	230
150	4572	6	15	0.01	270	0.01	300
200	6096	6	15	0.01	330	0.01	380



FIELD ELECTRICAL RESISTIVITY TEST DATA

Archway Solar ■ Christmas Valley, Lake County, Oregon

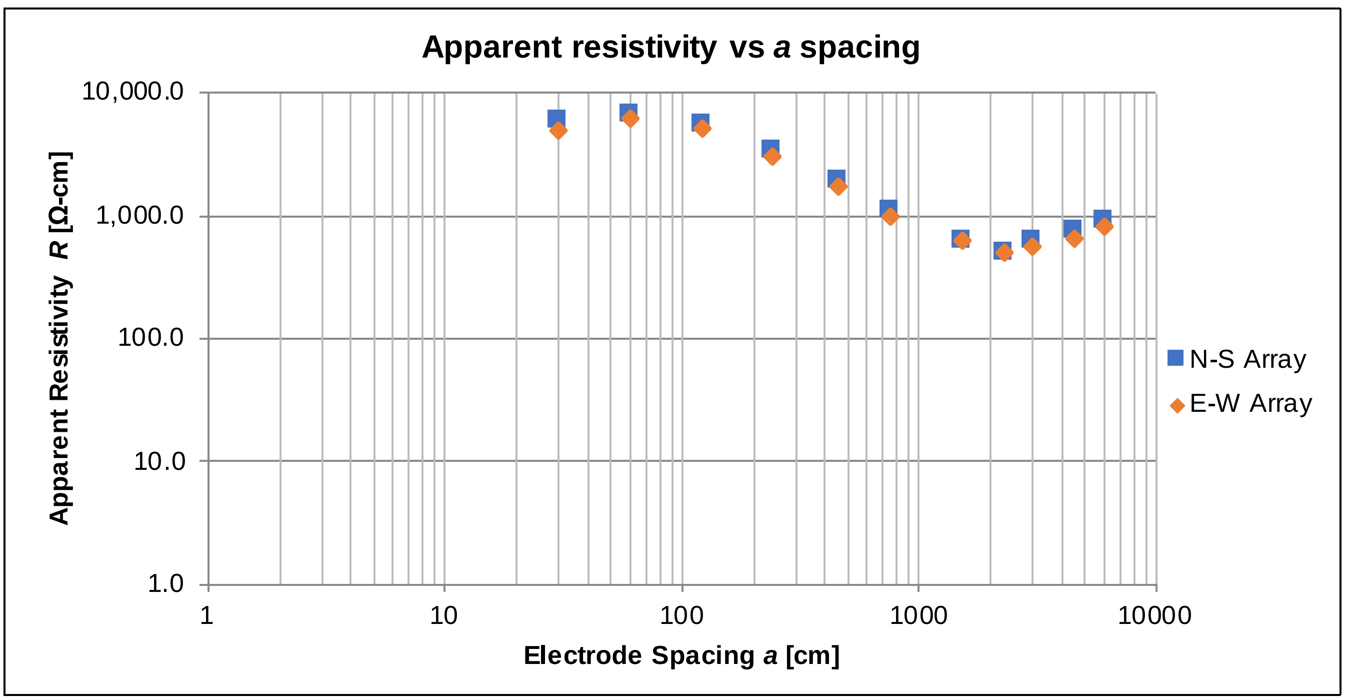
February 12, 2020 ■ Terracon Project No. 82185058

Terracon GeoReport

Array Loc.	Latitude/Longitude: 43.1987304, -120.4613803. Area near B-4, ER-2		
Instrument	MiniRes	**Weather**	50° F, partly cloudy
Serial #	SN-303	**Ground Cond.**	Elastic Silt with Sand
Cal. Check	February 12, 2020 w/ Calibration Belt	**Tested By**	JAE
Test Date	February 12, 2020	**Method**	Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)
Notes & Conflicts	Cow grazing pasture. Test area was relatively flat.		

Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi aR}{1 + \dfrac{2a}{\sqrt{a^2 + 4b^2}} - \dfrac{a}{\sqrt{a^2 + b^2}}}$

Electrode Spacing *a*		**Electrode Depth *b***		**N-S Test**		**E-W Test**	
[feet]	[centimeters]	[inches]	[centimeters]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]
1	30	6	15	24.10	5980	20.01	4960
2	61	6	15	16.37	6880	15.01	6310
4	122	6	15	7.07	5560	6.67	5240
8	244	6	15	2.27	3500	1.98	3060
15	457	6	15	0.69	1990	0.61	1770
25	762	6	15	0.23	1120	0.21	1020
50	1524	6	15	0.07	640	0.07	630
75	2286	6	15	0.04	520	0.04	510
100	3048	6	15	0.03	630	0.03	580
150	4572	6	15	0.03	780	0.02	670
200	6096	6	15	0.02	920	0.02	820



FIELD ELECTRICAL RESISTIVITY TEST DATA

Archway Solar ■ Christmas Valley, Lake County, Oregon

February 13, 2020 ■ Terracon Project No. 82185058

Array Loc. Latitude/Longitude: 43.1776489, -120.4603168. Area near B-7, ER-3

Instrument MiniRes **Weather** ~~37° F, partly cloudy~~

Serial # SN-303 **Ground Cond.** Elastic Silt with Sand

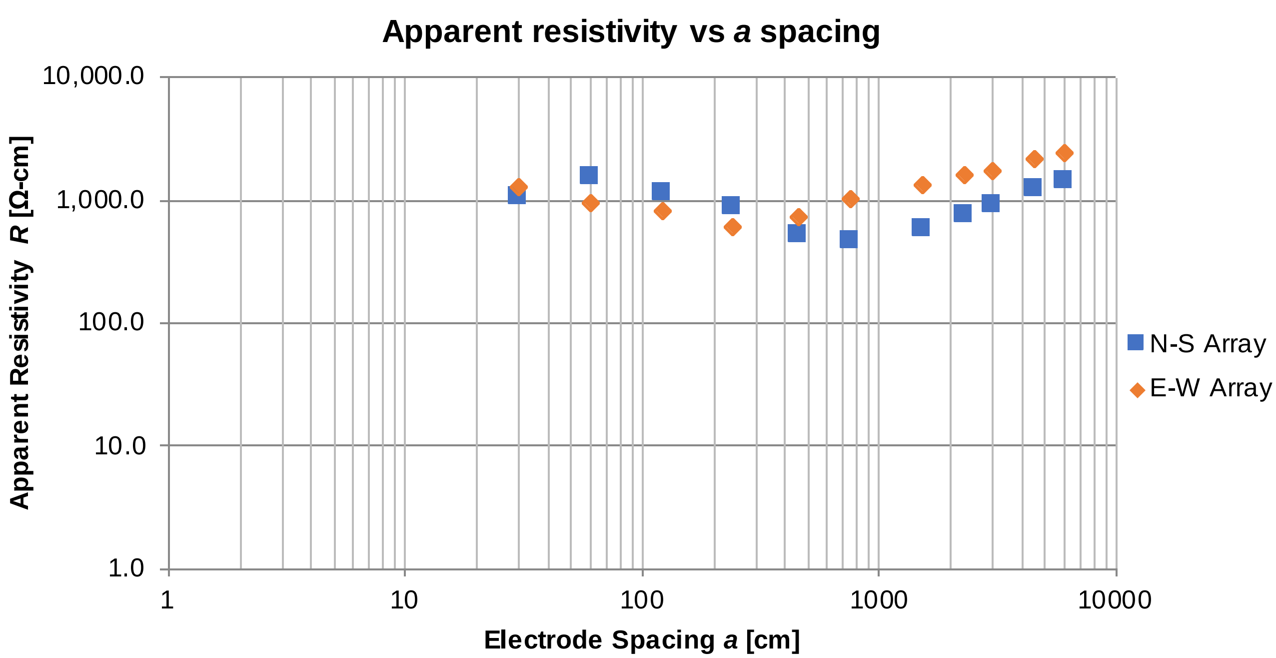
Cal. Check February 13, 2020 w/ Calibration Belt **Tested By** JAE/BWP

Test Date February 13, 2020 **Method** Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)

Notes & Conflicts Dense surficial sage brush present while testing. Test area was relatively flat.

Apparent resistivity ρ is calculated as : $= \dfrac{\quad}{1 + \dfrac{2a}{\sqrt{\ ^{2}\quad ^{2}}} - \dfrac{a}{\sqrt{\ ^{2}}}}$

Electrode Spacing *a*		**Electrode Depth *b***		**N-S Test**		**E-W Test**	
[feet]		[inches]	[centimeters]	Measured Resistance *R*	Apparent Resistivity *ρ*	Measured Resistance *R*	Apparent Resistivity *ρ*
				Ω	[Ω-cm]	Ω	[Ω-cm]
1	30	6	15	4.42	1100	5.20	1290
2	61	6	15	3.75	1570	2.32	970
4	122	6	15	1.51	1190	1.04	820
8	244	6	15	0.57	890	0.39	610
15	457	6	15	0.19	540	0.26	740
25	762	6	15	0.10	470	0.22	1040
50	1524	6	15	0.06	600	0.14	1340
75	2286	6	15	0.05	760	0.11	1610
100	3048	6	15	0.05	950	0.09	1790
150	4572	6	15	0.04	1260	0.08	2170
200	6096	6	15	0.04	1470	0.06	2460

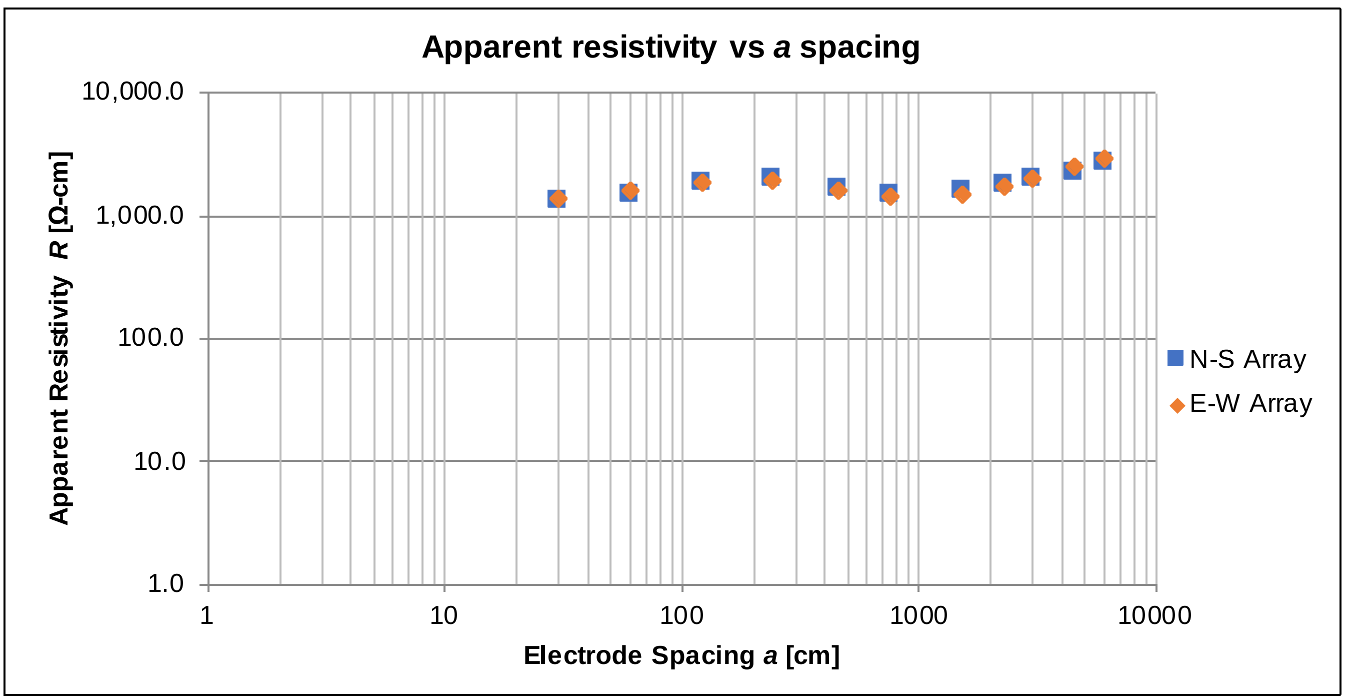


FIELD ELECTRICAL RESISTIVITY TEST DATA
Archway Solar ■ Christmas Valley, Lake County, Oregon
February 13, 2020 ■ Terracon Project No. 82185058

Array Loc.	Latitude/Longitude: 43.1663745, -120.4361774. Area near B-11, ER-4
Instrument	MiniRes
Weather	50° F, partly cloudy
Serial #	SN-303
Ground Cond.	Elastic Silt with Sand
Cal. Check	February 13, 2020 w/ Calibration Belt
Tested By	JAE/BWP
Test Date	February 13, 2020
Method	Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)
Notes & Conflicts	Dense surficial sage brush present while testing. Test area was relatively flat.

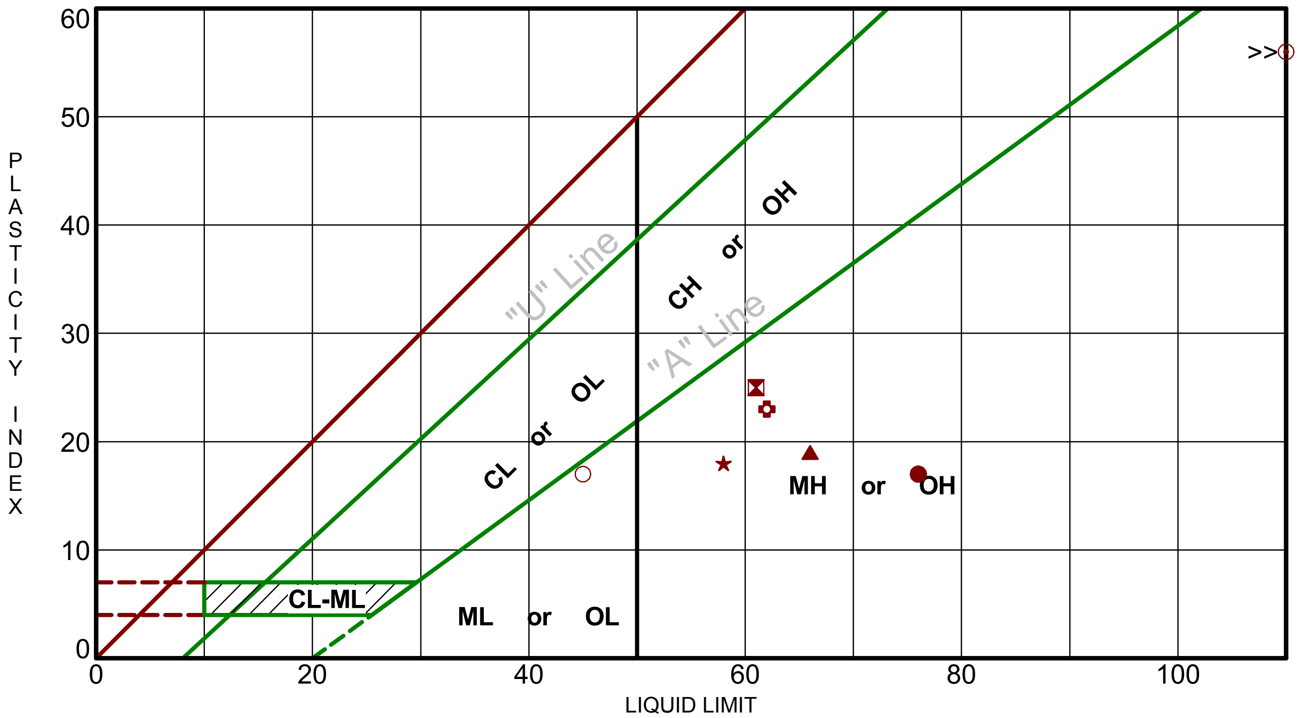
Apparent resistivity ρ is calculated as : $\rho = \dfrac{4\pi a R}{1 + \dfrac{2a}{\sqrt{a^2 + 4b^2}} - \dfrac{a}{\sqrt{a^2 + b^2}}}$

Electrode Spacing *a*		Electrode Depth *b*		N-S Test		E-W Test	
[feet]	[centimeters]	[inches]	[centimeters]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]	Measured Resistance *R* Ω	Apparent Resistivity ρ [Ω-cm]
1	30	6	15	5.57	1380	5.76	1430
2	61	6	15	3.60	1510	3.93	1650
4	122	6	15	2.40	1890	2.43	1910
8	244	6	15	1.32	2040	1.30	2010
15	457	6	15	0.59	1700	0.57	1650
25	762	6	15	0.32	1510	0.31	1460
50	1524	6	15	0.17	1610	0.16	1520
75	2286	6	15	0.13	1820	0.12	1760
100	3048	6	15	0.11	2020	0.11	2080
150	4572	6	15	0.08	2310	0.09	2580
200	6096	6	15	0.07	2790	0.08	3030



ATTERBERG LIMITS RESULTS

ASTM D4318



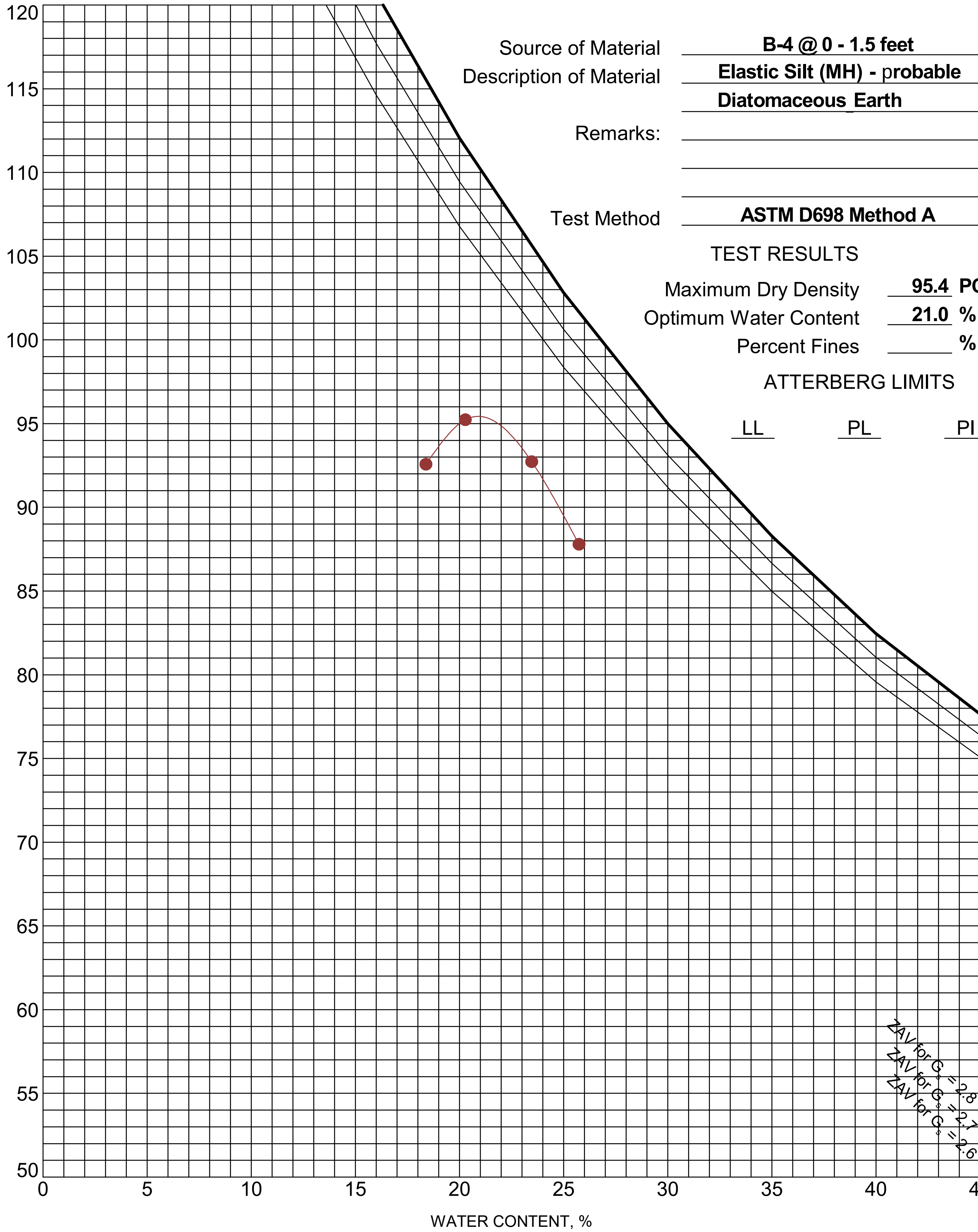
	Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
●	B-2	15 - 16.5	76	59	17		MH	ELASTIC SILT
⊠	B-3	2.5 - 4	61	36	25	77.9	MH	ELASTIC SILT with SAND
▲	B-4	2.5 - 4	66	47	19	91.5	MH	ELASTIC SILT
★	B-6	5 - 6.5	58	40	18		MH	ELASTIC SILT
⊙	B-7	20 - 21.5	137	81	56		MH	ELASTIC SILT
✿	B-8	2.5 - 4	62	39	23		MH	ELASTIC SILT
○	B-11	2.5 - 4	45	28	17	86.7	ML	SILT

PROJECT: Archway Solar Preliminary Geotechnical Study	PROJECT NUMBER: 82185058
SITE: 3 Mile Road Christmas Valley, OR	CLIENT: Invenergy LLC Chicago, IL

Terracon
700 NE 55th Ave
Portland, OR

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-4 @ 0 - 1.5 feet**

Description of Material: **Elastic Silt (MH) - probable Diatomaceous_Earth**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **95.4** **PCF**

Optimum Water Content **21.0** **%**

Percent Fines ____ **%**

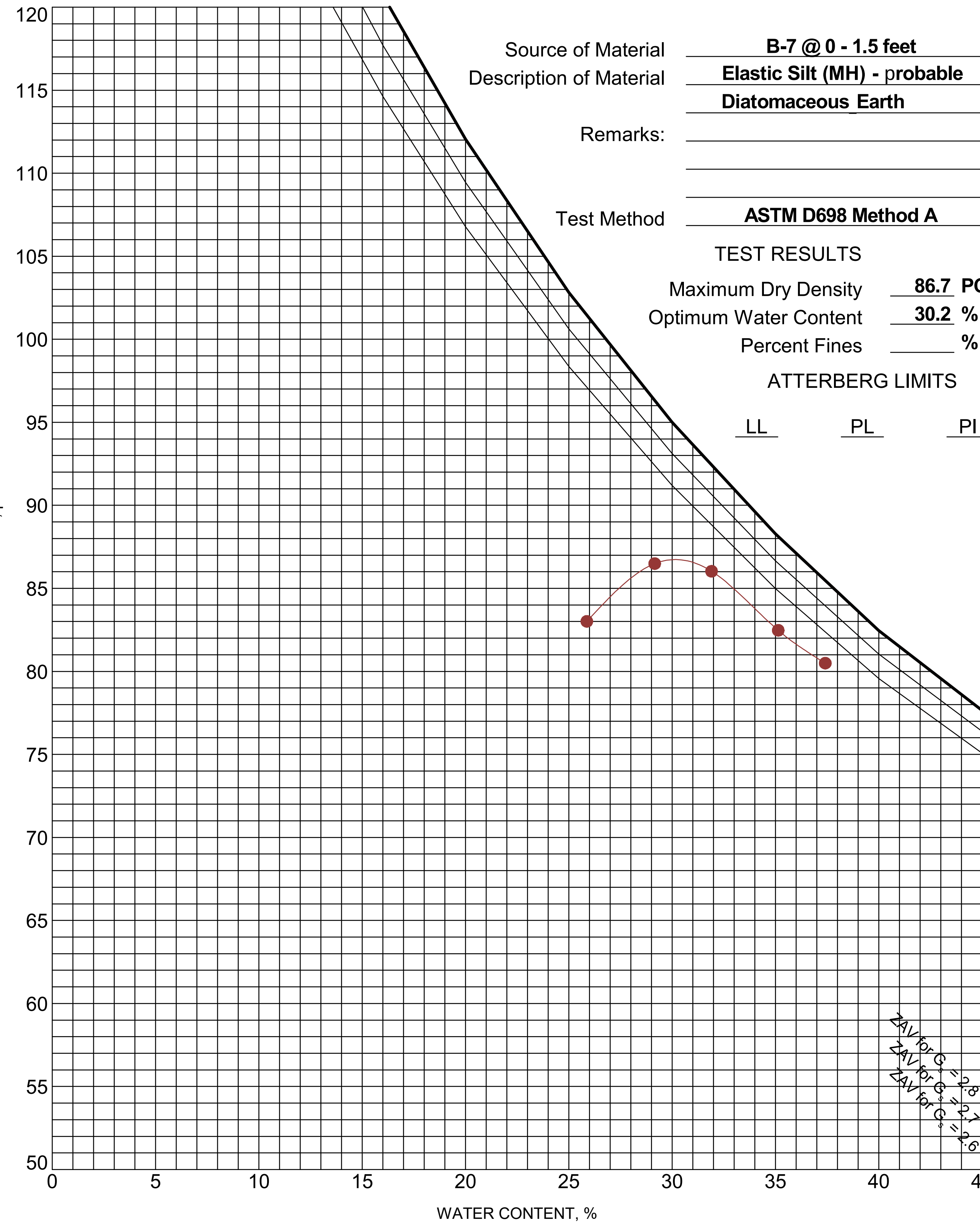
ATTERBERG LIMITS

LL	PL	PI

PROJECT: Archway Solar Preliminary Geotechnical Study	Terracon 700 NE 55th Ave Portland, OR	PROJECT NUMBER: 82185058
SITE: 3 Mile Road Christmas Valley, OR		CLIENT: Invenergy LLC Chicago, IL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-7 @ 0 - 1.5 feet**

Description of Material: **Elastic Silt (MH) - probable Diatomaceous_Earth**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **86.7 PCF**

Optimum Water Content **30.2 %**

Percent Fines **%**

ATTERBERG LIMITS

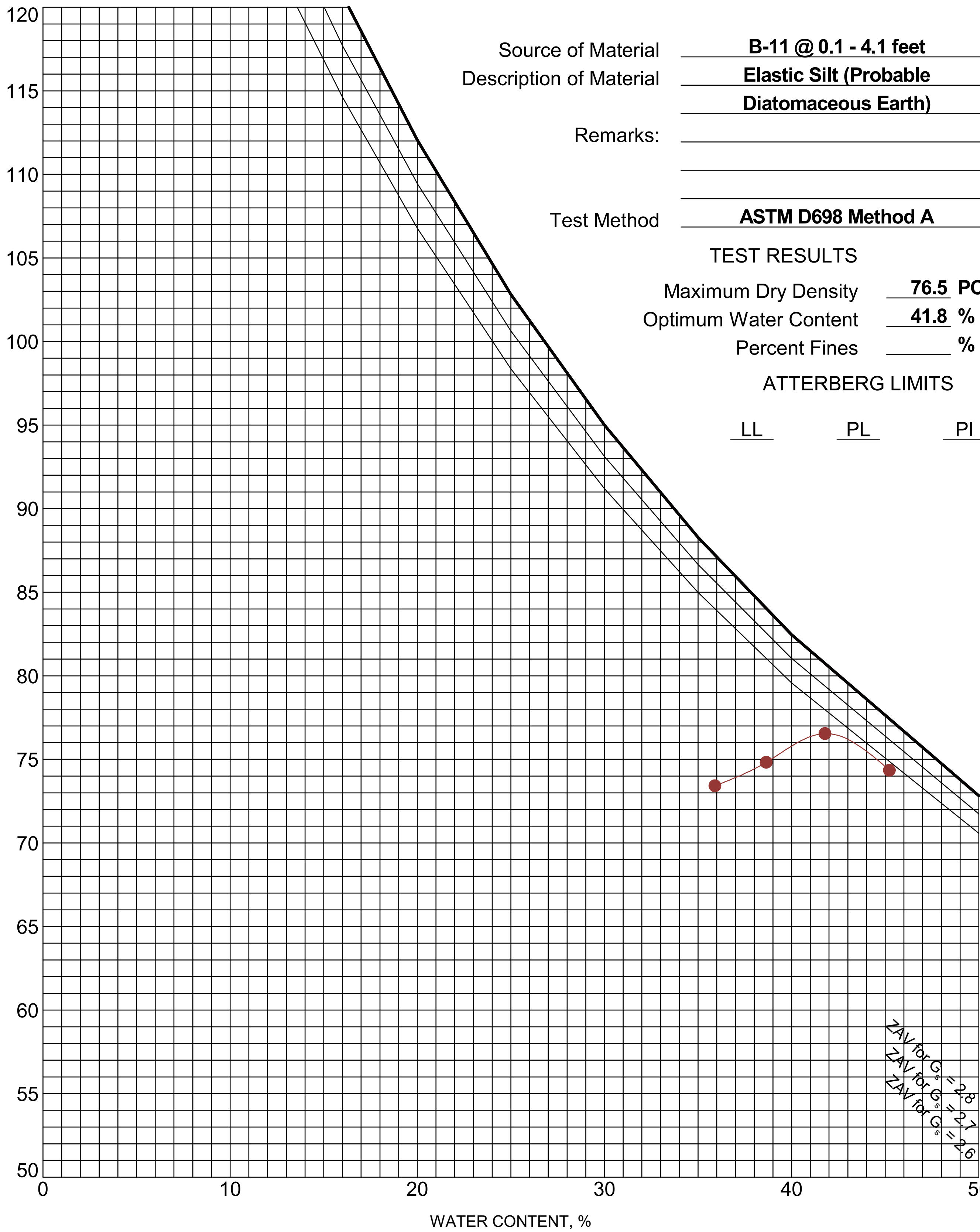
LL	PL	PI

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V3 BACKUP OF 82185058 ARCHWAY SOLAR FIELD LOGS 1-3-2020 - COPY.GPJ TERRACON_DATATEMPLATE.GDT 2/28/20

PROJECT: Archway Solar Preliminary Geotechnical Study	Terracon 700 NE 55th Ave Portland, OR	PROJECT NUMBER: 82185058
SITE: 3 Mile Road Christmas Valley, OR		CLIENT: Invenergy LLC Chicago, IL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**B-11 @ 0.1 - 4.1 feet**
Description of Material	**Elastic Silt (Probable Diatomaceous Earth)**
Remarks:	
Test Method	**ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**76.5**	**PCF**
Optimum Water Content	**41.8**	**%**
Percent Fines		**%**

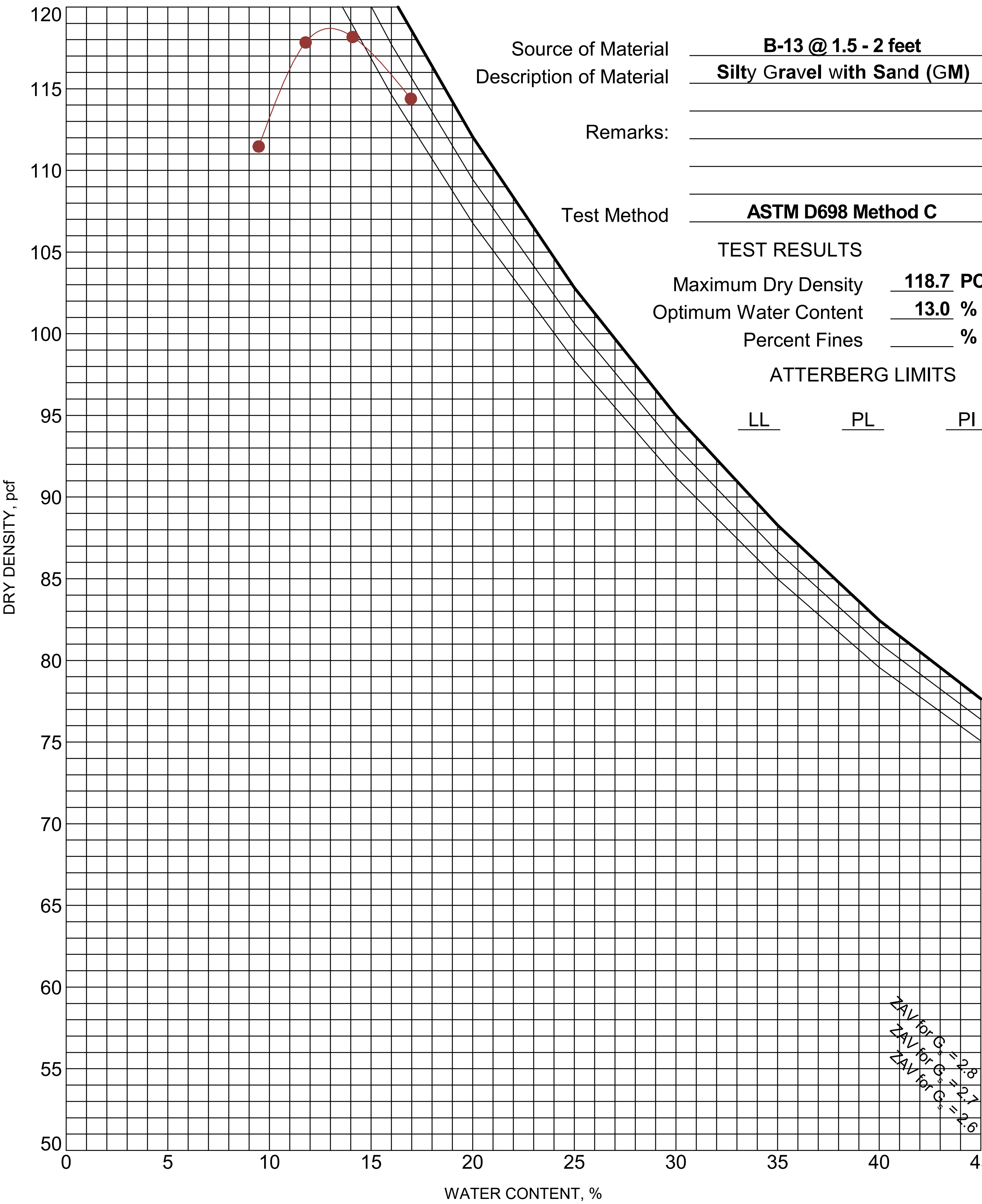
ATTERBERG LIMITS

LL	PL	PI

PROJECT: Archway Solar Preliminary Geotechnical Study	Terracon 700 NE 55th Ave Portland, OR	PROJECT NUMBER: 82185058
SITE: 3 Mile Road Christmas Valley, OR		CLIENT: Invenergy LLC Chicago, IL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-13 @ 1.5 - 2 feet**

Description of Material: **Silty Gravel with Sand (GM)**

Remarks:

Test Method: **ASTM D698 Method C**

TEST RESULTS

Maximum Dry Density **118.7 PCF**

Optimum Water Content **13.0 %**

Percent Fines ____ %

ATTERBERG LIMITS

LL	PL	PI

PROJECT: Archway Solar Preliminary Geotechnical Study

SITE: 3 Mile Road
Christmas Valley, OR

Terracon
700 NE 55th Ave
Portland, OR

PROJECT NUMBER: 82185058

CLIENT: Invenergy LLC
Chicago, IL

Geotherm USA

21239 FM529 Rd., Bldg. F
Cypress, TX 77433
Tel: 281-985-9344
Fax: 832-427-1752
info@geothermusa.com
http://www.geothermusa.com

January 15, 2020

Terracon Consultants
700 NE 55th Ave
Portland, OR 97213
Attn: Brice W. Plouse, P.E.

Re: Thermal Analysis of Native Soil Samples
Archway Solar Farm – Christmas Valley, OR (PO No. 82185058)

The following is the report of thermal dryout characterization tests conducted on the two (2) bulk soil samples and two (2) Shelby tube samples from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The Shelby tube samples were tested 'as received' and the bulk samples were tested at the 'as received' moisture content and at 85% and 95% of the Proctor density ***provided by Terracon.*** The tests were conducted in accordance with the IEEE standard 442-2017. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 & 2.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Compaction Effort (%)	Soil Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
			Wet	Dry		
B-4	Tube @ 5' – 5.5'	Diatomaceous Earth (Elastic Silt)	147	509	27	88
	85 @ 0' – 4'		240	585	17	81
	95 @ 0' – 4'		205	480		91
B-7	Tube @ 4.25' – 4.75'	Diatomaceous Earth (Elastic Silt)	176	590	33	81
	85 @ 0' – 4'		239	650	18	74
	95 @ 0' – 4'		194	579		82

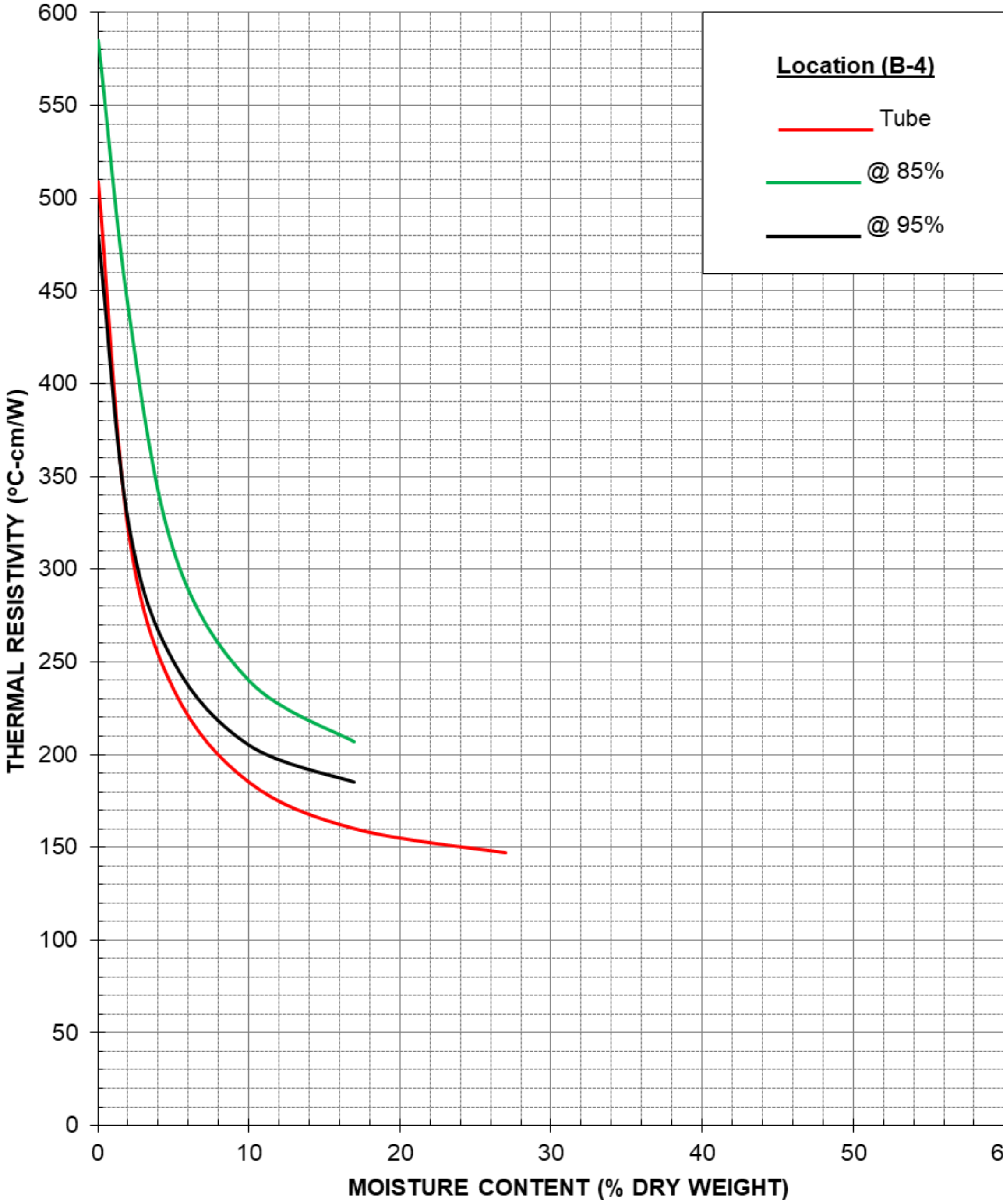
Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Nimesh Patel

THERMAL DRYOUT CURVES



Terracon Consultants (PO No. 82185058)

Thermal Analysis of Native Soil

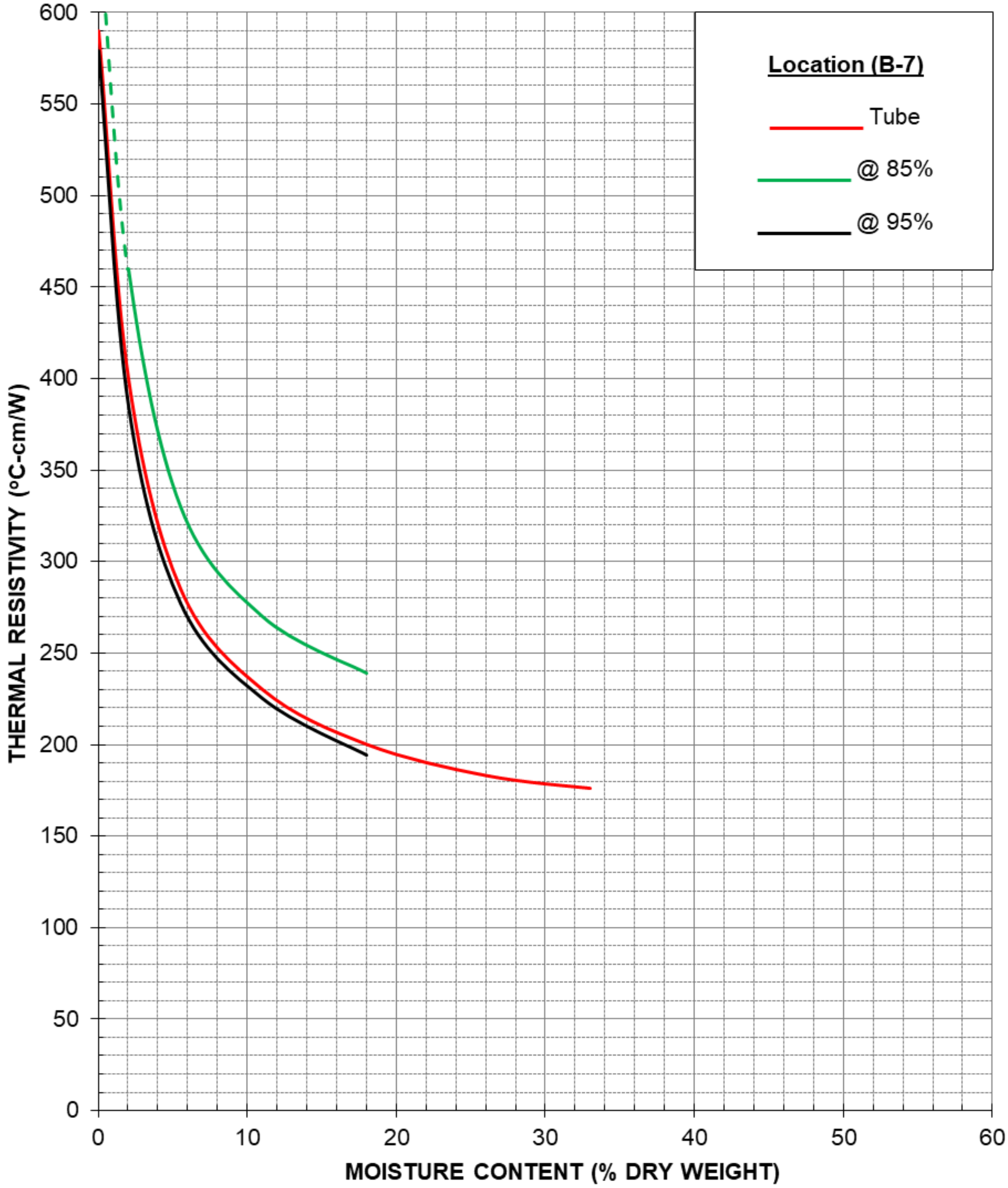
Archway Solar Farm – Christmas Valley, OR

January 2020

Figure 1

Geotherm USA

THERMAL DRYOUT CURVES



Terracon Consultants (PO No. 82185058)

Thermal Analysis of Native Soil

Archway Solar Farm – Christmas Valley, OR

January 2020 **Figure 2**

21239 FM529 Rd., Bldg. F
Cypress, TX 77433
Tel: 281-985-9344
Fax: 832-427-1752
info@geothermusa.com
http://www.geothermusa.com

February 3, 2020

Terracon Consultants
700 NE 55th Ave
Portland, OR 97213
Attn: Brice W. Plouse, P.E.

Re: Thermal Analysis of Native Soil Samples
Archway Solar Farm – Christmas Valley, OR (PO No. 82185058)

The following is the report of thermal dryout characterization tests conducted on the two (2) bulk soil samples and two (2) Shelby tube samples from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The Shelby tube samples were tested 'as received' and the bulk samples were tested at the 'as received' moisture content and at 85% and 95% of the Proctor density ***provided by Terracon.*** The tests were conducted in accordance with the IEEE standard 442-2017. The results are tabulated below and the thermal dryout curves are presented in **Figures 1 & 2.**

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

<table>
<tr><th rowspan="2">Sample ID</th><th rowspan="2">Compaction Effort (%)</th><th rowspan="2">Soil Description (Terracon)</th><th colspan="2">Thermal Resistivity (°C-cm/W)</th><th rowspan="2">Moisture Content (%)</th><th rowspan="2">Dry Density (lb/ft^3)</th></tr>
<tr><th>Wet</th><th>Dry</th></tr>
<tr><td rowspan="3">B-11</td><td>Tube @ 4' – 4.5'</td><td rowspan="3">Diatomaceous Earth (Elastic Silt)</td><td>135</td><td>613</td><td>25</td><td>78</td></tr>
<tr><td>85% @ 0.1' – 4.1'</td><td>144</td><td>694</td><td rowspan="2">42</td><td>65</td></tr>
<tr><td>95% @ 0.1' – 4.1'</td><td>123</td><td>640</td><td>73</td></tr>
<tr><td rowspan="3">B-13</td><td>Tube @ 0' – 0.8'</td><td>Poorly Graded Gravel w/ Silt & Sand</td><td>127</td><td>353</td><td>15</td><td>96</td></tr>
<tr><td>85% @ 0.1' – 4.1'</td><td rowspan="2">Silty Gravel w/ Sand</td><td>117</td><td>318</td><td rowspan="2">13</td><td>101</td></tr>
<tr><td>95% @ 0.1' – 4.1'</td><td>106</td><td>279</td><td>113</td></tr>
</table>

<u>Comments:</u> The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

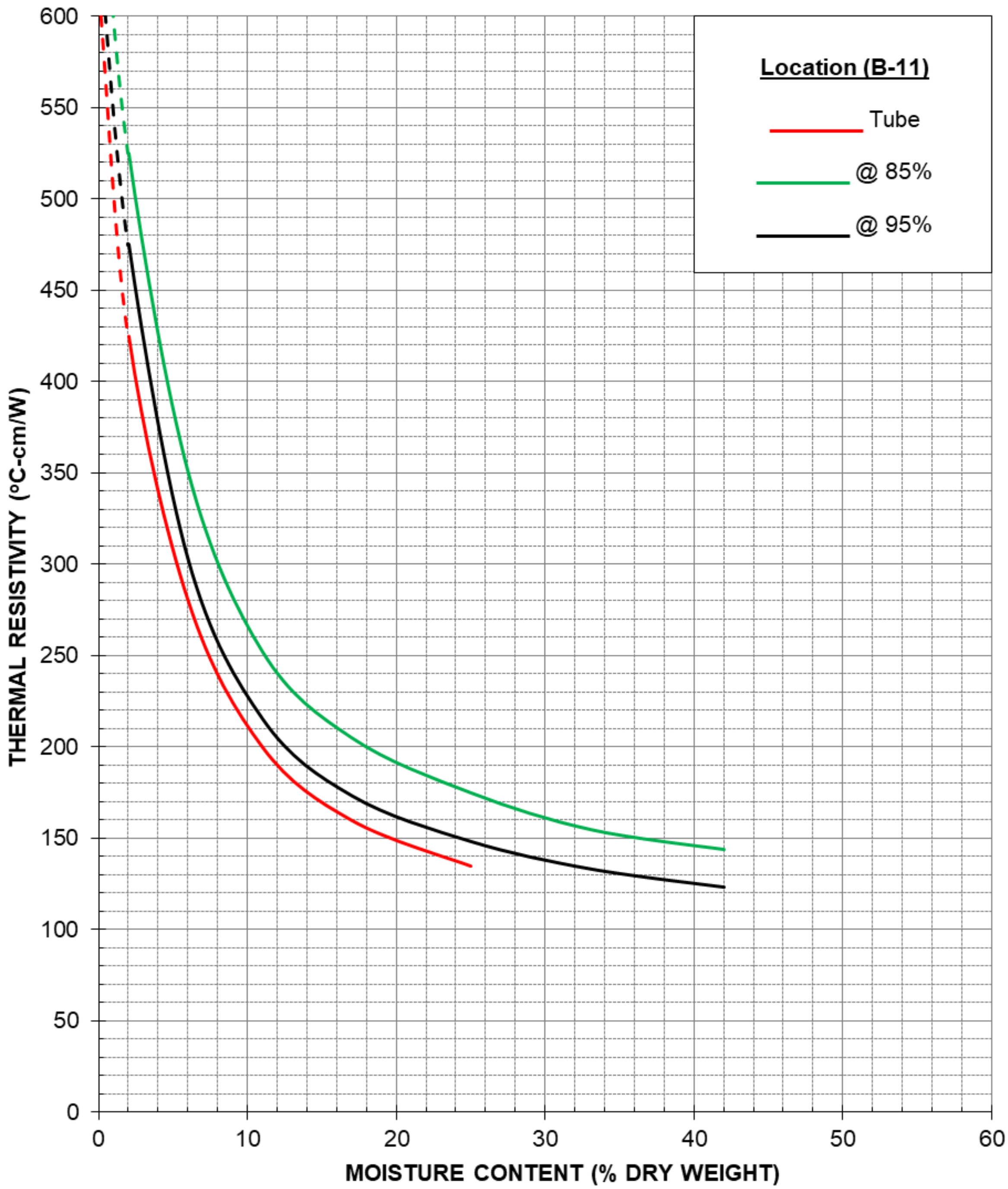
Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA

Nimesh Patel

Geotherm USA

THERMAL DRYOUT CURVES



Terracon Consultants (PO No. 82185058)

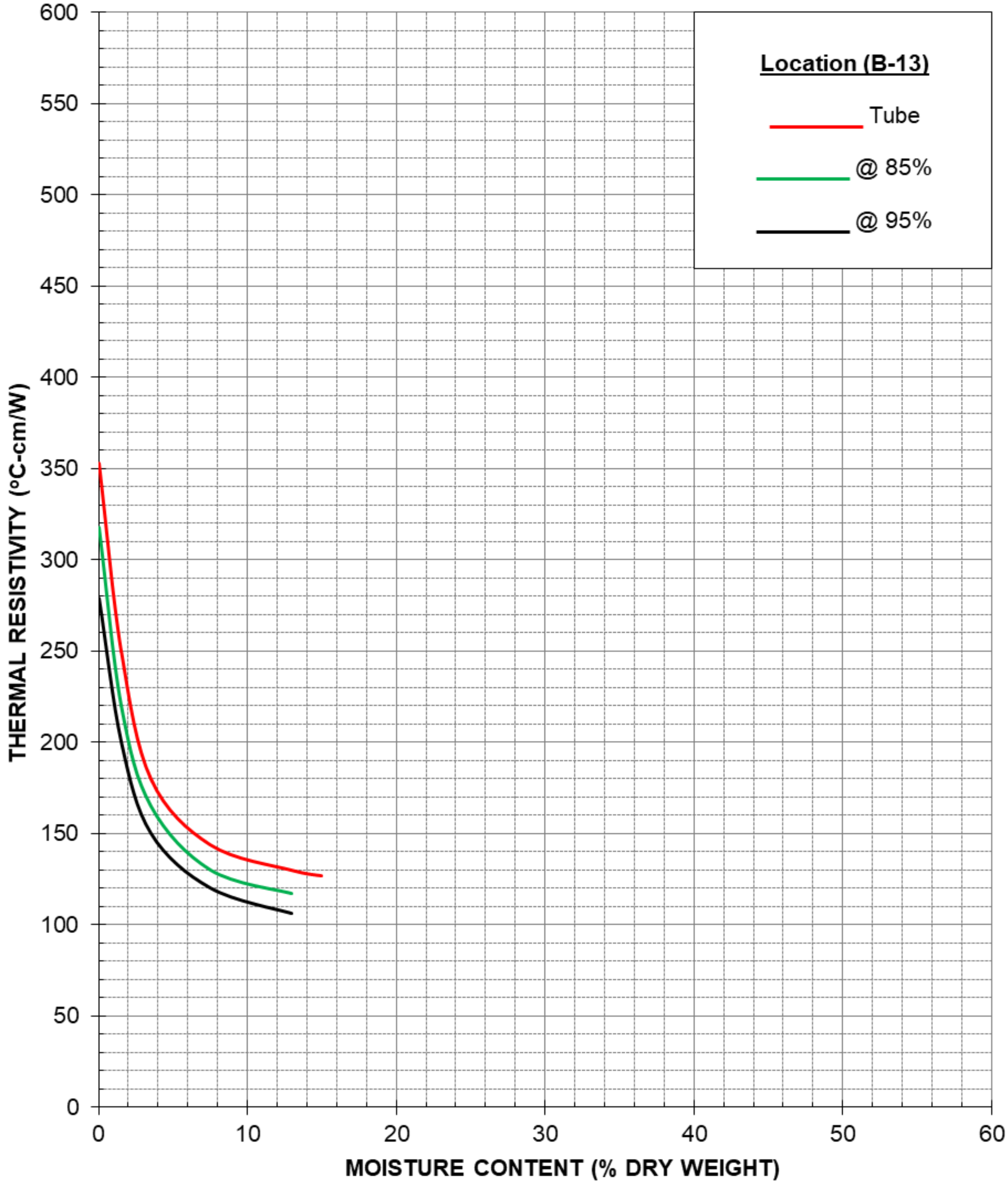
Thermal Analysis of Native Soil

Archway Solar Farm – Christmas Valley, OR

February 2020

Figure 1

THERMAL DRYOUT CURVES



Terracon Consultants (PO No. 82185058)

Thermal Analysis of Native Soil

Archway Solar Farm – Christmas Valley, OR

February 2020

Figure 2

CHEMICAL LABORATORY TEST REPORT

Project Number: 82185058
Service Date: 01/07/20
Report Date: 01/22/20
Task:

Terracon
750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Invenergy LLC

Project

Archway Solar Project

Sample Submitted By: Terracon (82) **Date Received:** 1/6/2020 **Lab No.: 20-0032**

Results of Corrosion Analysis

Sample Number	S-1	S-1	S-1	S-1
Sample Location	B-1	B-2	B-3	B-4
Sample Depth (ft.)	0.0	0.0	0.0	0.0
pH Analysis, AWWA 4500 H	8.07	9.11	8.57	9.33
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	132	3210	1205	88
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	145	2650	4250	108
Red-Ox, AWWA 2580, (mV)	+685	+693	+693	+687
Total Salts, AWWA 2540, (mg/kg)	2587	17696	17528	3035
Resistivity, ASTM G 57, (ohm-cm)	1038	116	126	922

Analyzed By: Trisha Campo

Trisha Campo
Chemist

CHEMICAL LABORATORY TEST REPORT

Project Number: 82185058
Service Date: 01/07/20
Report Date: 01/22/20
Task:

Terracon
750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Invenergy LLC

Project

Archway Solar Project

Sample Submitted By: Terracon (82) **Date Received:** 1/6/2020 **Lab No.: 20-0032**

Results of Corrosion Analysis

Sample Number	S-1	S-1	S-1	S-1
Sample Location	B-5	B-6	B-7	B-8
Sample Depth (ft.)	0.0	0.0	0.0	0.0
pH Analysis, AWWA 4500 H	8.32	9.15	8.15	8.25
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	2200	5291	1210	550
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	2875	495	1088	1823
Red-Ox, AWWA 2580, (mV)	+687	+685	+684	+688
Total Salts, AWWA 2540, (mg/kg)	14896	15904	10595	12656
Resistivity, ASTM G 57, (ohm-cm)	126	155	175	165

Analyzed By: Trisha Campo

Trisha Campo
Chemist

CHEMICAL LABORATORY TEST REPORT

Project Number: 82185058
Service Date: 01/07/20
Report Date: 01/22/20
Task:

750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Invenergy LLC

Project

Archway Solar Project

Sample Submitted By: Terracon (82) **Date Received:** 1/6/2020 **Lab No.: 20-0032**

Results of Corrosion Analysis

Sample Number	S-1	S-1	S-1	S-1
Sample Location	B-9	B-10	B-11	B-12
Sample Depth (ft.)	0.0	0.0	0.0	0.0
pH Analysis, AWWA 4500 H	8.77	8.77	8.16	9.03
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	4967	1983	102	105
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	1408	1755	753	485
Red-Ox, AWWA 2580, (mV)	+688	+685	+683	+683
Total Salts, AWWA 2540, (mg/kg)	18312	13552	8702	3528
Resistivity, ASTM G 57, (ohm-cm)	112	146	262	427

Analyzed By: Trisha Campo

Trisha Campo
Chemist

CHEMICAL LABORATORY TEST REPORT

Project Number: 82185058
Service Date: 01/07/20
Report Date: 01/22/20
Task:

Terracon
750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Invenergy LLC

Project

Archway Solar Project

Sample Submitted By: Terracon (82) **Date Received:** 1/6/2020 **Lab No.: 20-0032**

Results of Corrosion Analysis

Sample Number	S-1	S-1
Sample Location	B-13	B-14
Sample Depth (ft.)	0.0	0.0
pH Analysis, AWWA 4500 H	8.16	8.22
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	124	1515
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	68	1825
Red-Ox, AWWA 2580, (mV)	+680	+684
Total Salts, AWWA 2540, (mg/kg)	974	10214
Resistivity, ASTM G 57, (ohm-cm)	2037	223

Analyzed By: Trisha Campo

Trisha Campo
Chemist

CHEMICAL LABORATORY TEST REPORT

Project Number: 82185058
Service Date: 01/30/20
Report Date: 02/04/20
Task:

Terracon
750 Pilot Road, Suite F
Las Vegas, Nevada 89119
(702) 597-9393

Client

Invenergy LLC

Project

Archway Solar Project

Sample Submitted By: Terracon (82) **Date Received:** 1/28/2020 **Lab No.: 20-0103**

Results of Corrosion Analysis

Sample Number				
Sample Location	PLT-1	PLT-2	PLT-3	PLT-4
Sample Depth (ft.)	0.0-5.0	0.0-5.0	0.0-5.0	0.0-5.0
pH Analysis, AWWA 4500 H	8.51	8.27	8.11	8.55
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	58	85	63	50
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	72	52	123	50
Red-Ox, AWWA 2580, (mV)	+688	+691	+689	+676
Total Salts, AWWA 2540, (mg/kg)	1159	1893	1753	103
Resistivity, ASTM G 57, (ohm-cm)	1746	2037	1649	13580

Analyzed By: Trisha Campo

Trisha Campo
Chemist

PILE LOAD TESTING RESULTS

Contents:

Terracon

Tension Load Test Result for PLT-1, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

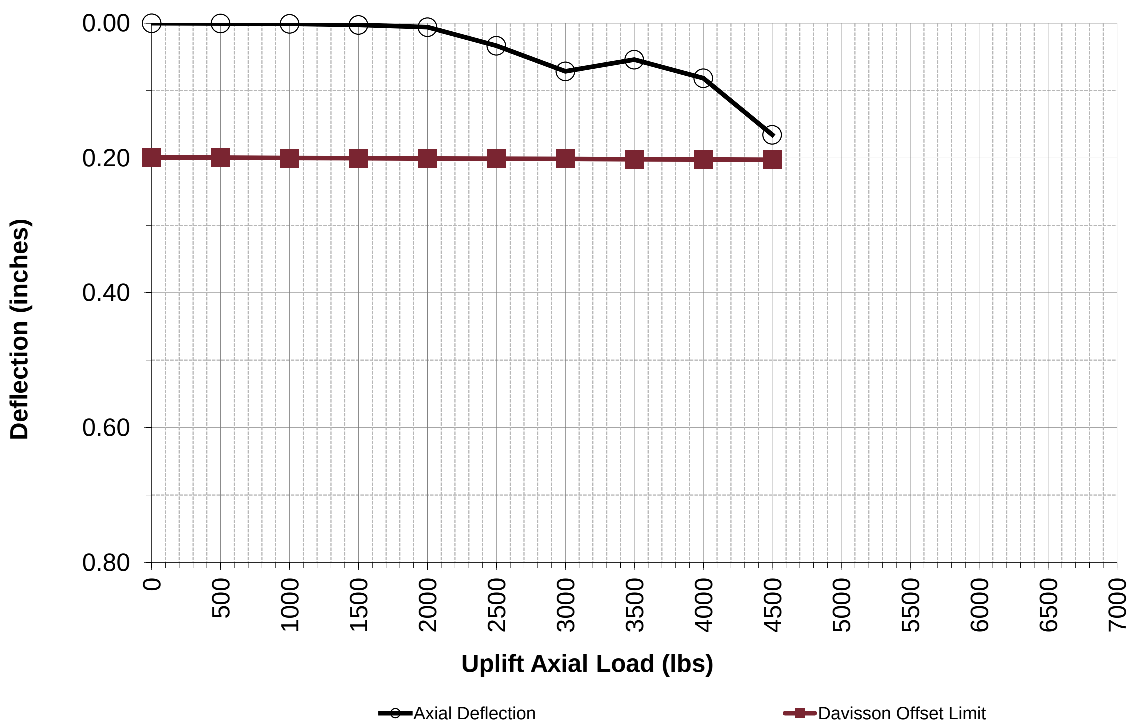
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-1, 9ft
Latitude:	43.21525
Longitude:	-120.45301
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	29

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.000	0.000	0.200	
14%	1000	0.001	0.001	0.200	
21%	1500	0.002	0.001	0.200	
29%	2000	0.006	0.002	0.201	
36%	2500	0.034	0.002	0.201	
43%	3000	0.072	0.002	0.201	
50%	3500	0.054	0.003	0.202	
57%	4000	0.081	0.003	0.202	
64%	4500	0.166	0.003	0.203	
71%	5000		0.004		
79%	5500		0.004		
86%	6000		0.005		
93%	6500		0.005		
100%	7000		0.005		



Tension Load Test Result for PLT-1, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

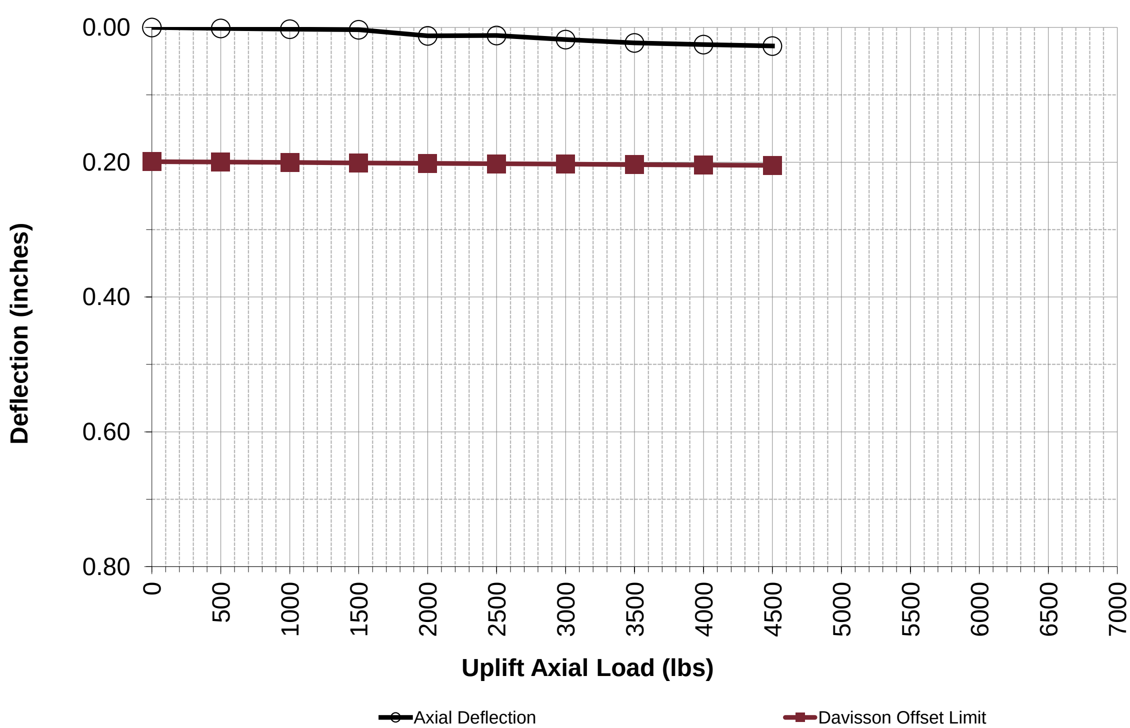
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-1, 12ft
Latitude:	43.21525
Longitude:	-120.45301
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	77

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.002	0.001	0.200	
14%	1000	0.003	0.001	0.200	
21%	1500	0.004	0.002	0.201	
29%	2000	0.013	0.002	0.202	
36%	2500	0.012	0.003	0.202	
43%	3000	0.018	0.004	0.203	
50%	3500	0.023	0.004	0.203	
57%	4000	0.026	0.005	0.204	
64%	4500	0.028	0.006	0.205	
71%	5000		0.006		
79%	5500		0.007		
86%	6000		0.007		
93%	6500		0.008		
100%	7000		0.009		



Tension Load Test Result for PLT-2, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

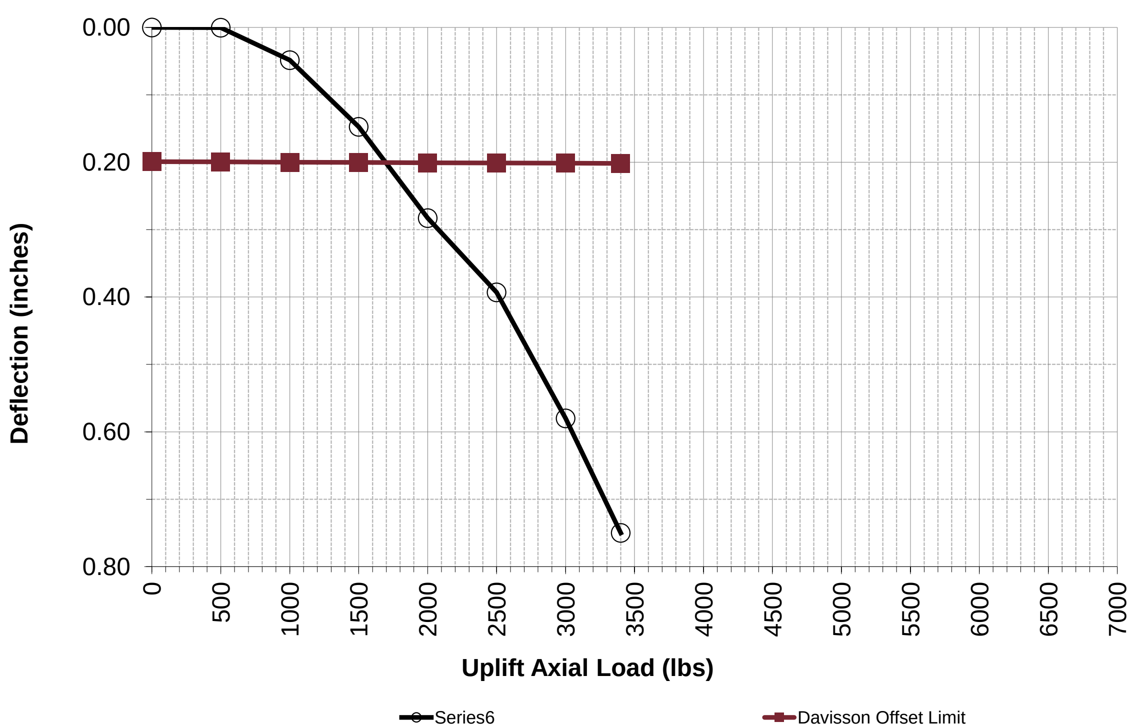
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-2, 9ft
Latitude:	43.19860
Longitude:	-120.46182
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	9

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.001	0.000	0.200	
14%	1000	0.049	0.001	0.200	
21%	1500	0.148	0.001	0.200	
29%	2000	0.283	0.002	0.201	
36%	2500	0.393	0.002	0.201	
43%	3000	0.580	0.002	0.201	
49%	3400	0.750	0.003	0.202	
57%	4000		0.003		
63%	4400		0.003		
71%	5000		0.004		
79%	5500		0.004		
86%	6000		0.005		
93%	6500		0.005		
100%	7000		0.005		



Tension Load Test Result for PLT-2, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

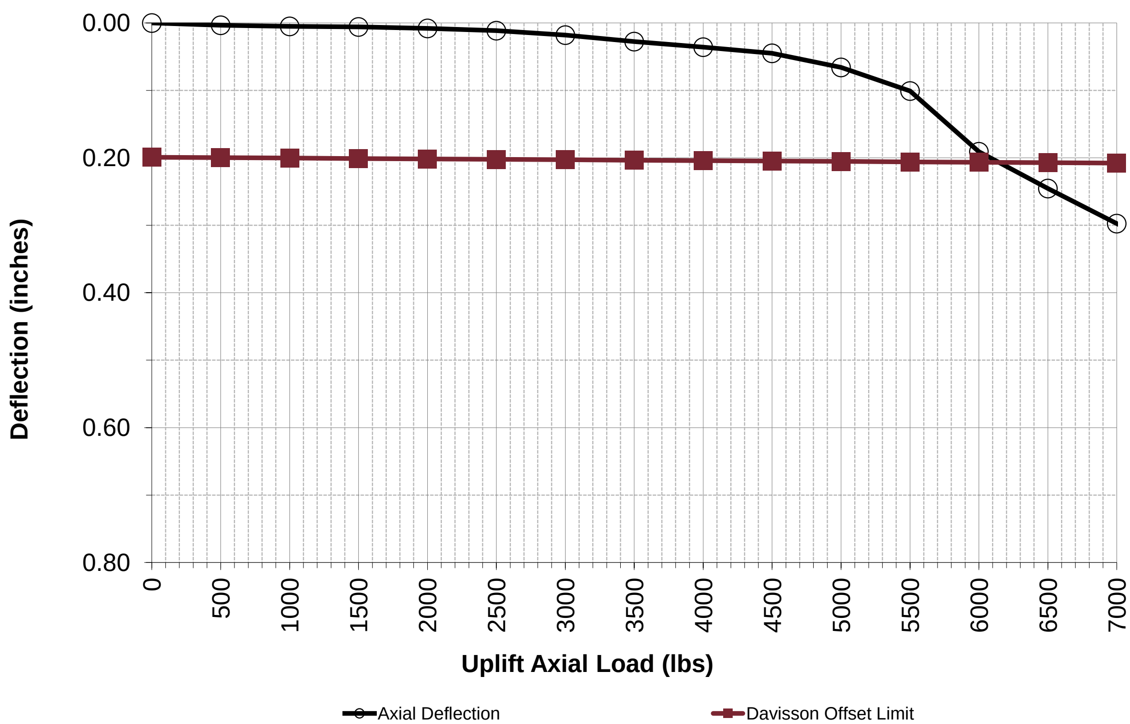
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-2, 12ft
Latitude:	43.19860
Longitude:	-120.46182
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	21

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.003	0.001	0.200	
14%	1000	0.005	0.001	0.200	
21%	1500	0.006	0.002	0.201	
29%	2000	0.008	0.002	0.202	
36%	2500	0.012	0.003	0.202	
43%	3000	0.018	0.004	0.203	
50%	3500	0.028	0.004	0.203	
57%	4000	0.036	0.005	0.204	
64%	4500	0.045	0.006	0.205	
71%	5000	0.066	0.006	0.205	
79%	5500	0.101	0.007	0.206	
86%	6000	0.191	0.007	0.207	
93%	6500	0.246	0.008	0.207	
100%	7000	0.298	0.009	0.208	



Tension Load Test Result for PLT-3, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

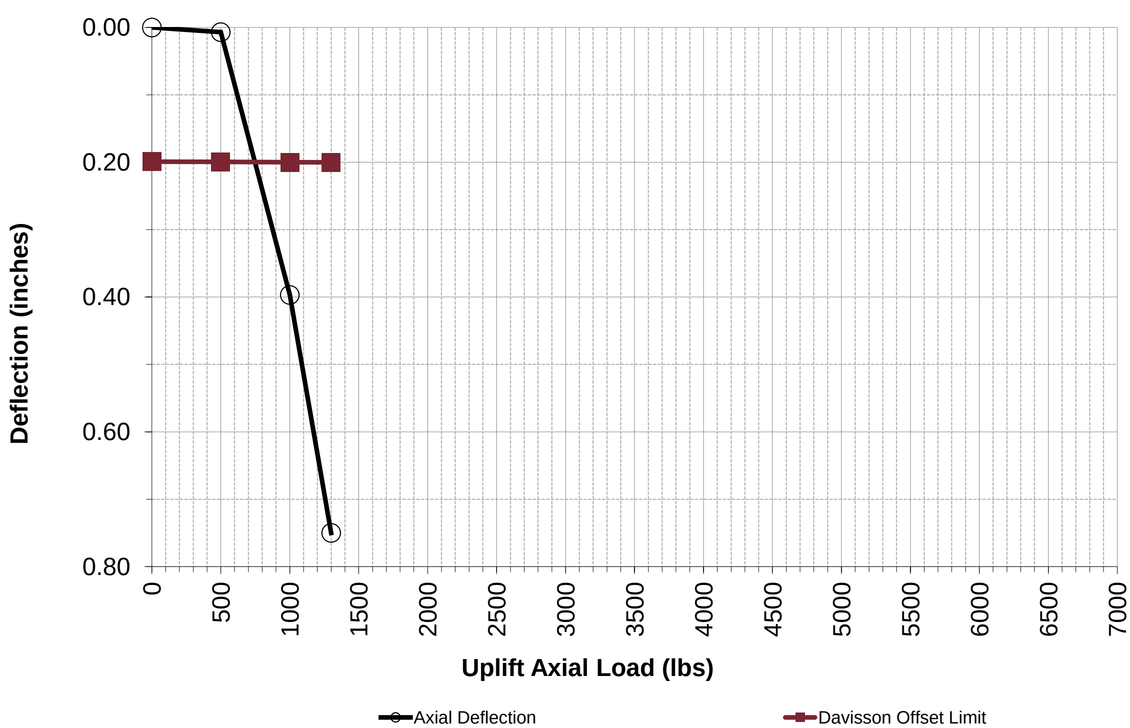
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-3, 9ft
Latitude:	43.18648
Longitude:	-120.45688
Pile Type:	W6X9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	8

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.007	0.000	0.200	
14%	1000	0.397	0.001	0.200	
19%	1300	0.750	0.001	0.200	
29%	2000		0.002		
36%	2500		0.002		
43%	3000		0.002		
50%	3500		0.003		
57%	4000		0.003		
64%	4500		0.003		
71%	5000		0.004		
79%	5500		0.004		
86%	6000		0.005		
93%	6500		0.005		
100%	7000		0.005		



Tension Load Test Result for PLT-3, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

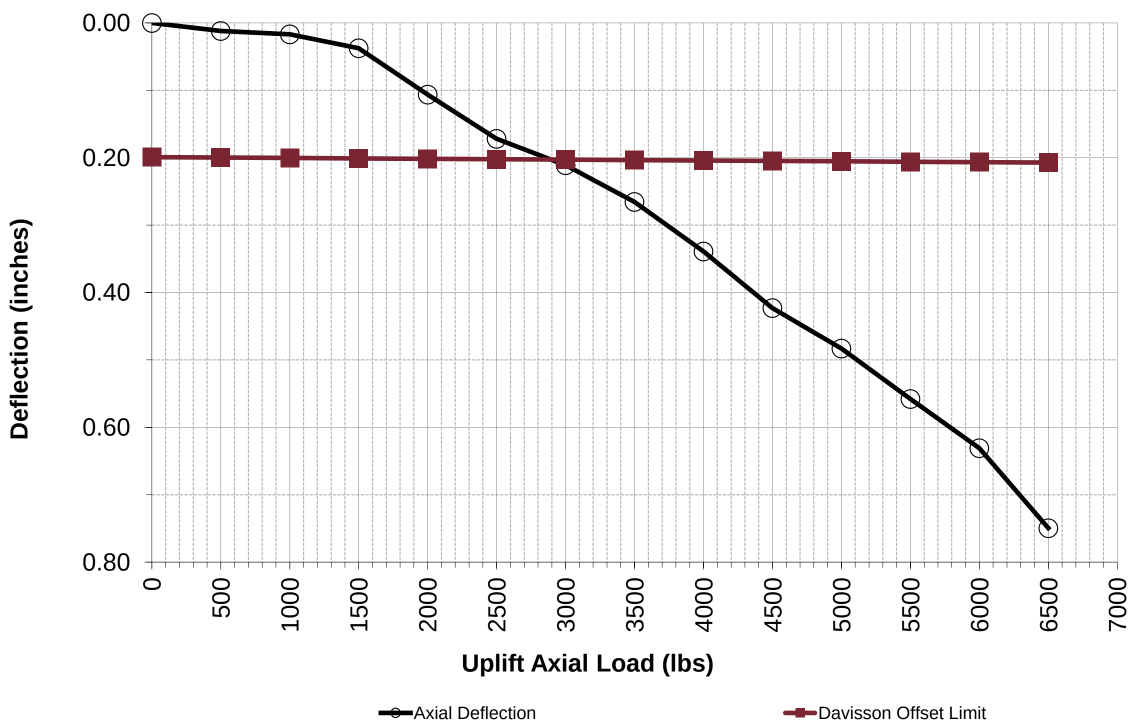
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-3, 12ft
Latitude:	43.18648
Longitude:	-120.45688
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	17

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.012	0.001	0.200	
14%	1000	0.017	0.001	0.200	
21%	1500	0.037	0.002	0.201	
29%	2000	0.107	0.002	0.202	
36%	2500	0.172	0.003	0.202	
43%	3000	0.211	0.004	0.203	
50%	3500	0.266	0.004	0.203	
57%	4000	0.339	0.005	0.204	
64%	4500	0.423	0.006	0.205	
71%	5000	0.483	0.006	0.205	
79%	5500	0.558	0.007	0.206	
86%	6000	0.631	0.007	0.207	
93%	6500	0.750	0.008	0.207	
100%	7000		0.009		



Tension Load Test Result for PLT-4, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

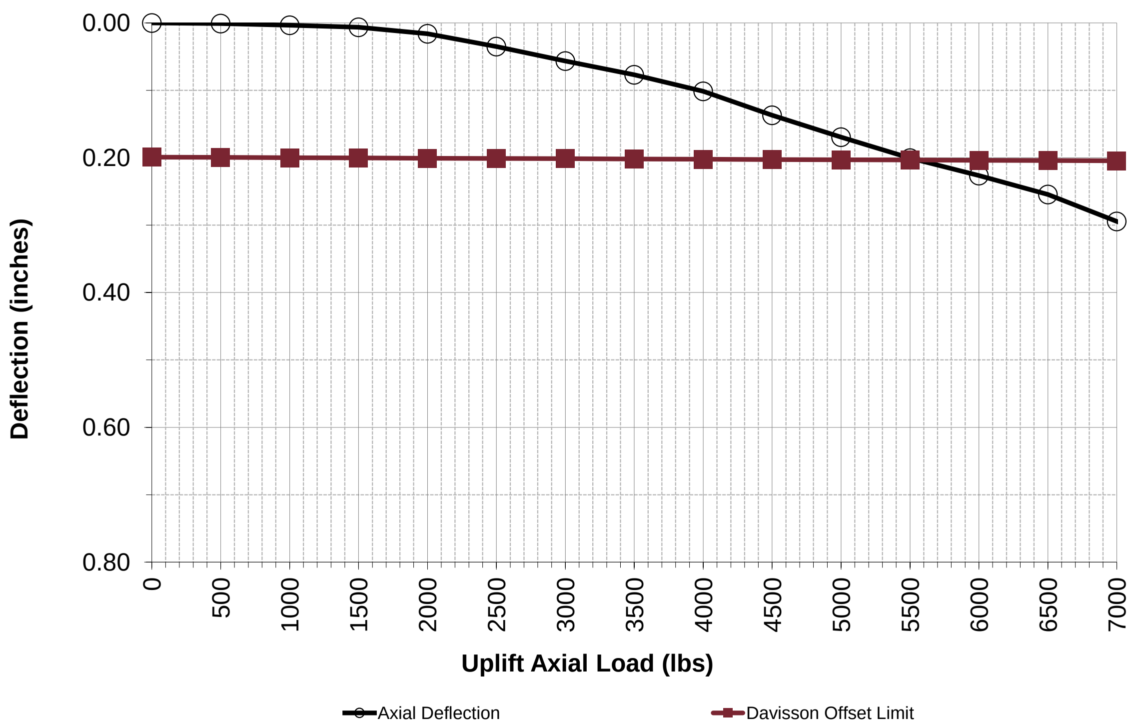
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-4, 9ft
Latitude:	43.16221
Longitude:	-120.45297
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	14

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offest Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.001	0.000	0.200	
14%	1000	0.003	0.001	0.200	
21%	1500	0.006	0.001	0.200	
29%	2000	0.016	0.002	0.201	
36%	2500	0.035	0.002	0.201	
43%	3000	0.056	0.002	0.201	
50%	3500	0.077	0.003	0.202	
57%	4000	0.102	0.003	0.202	
64%	4500	0.137	0.003	0.203	
71%	5000	0.170	0.004	0.203	
79%	5500	0.201	0.004	0.203	
86%	6000	0.227	0.005	0.204	
93%	6500	0.255	0.005	0.204	
100%	7000	0.295	0.005	0.205	



Tension Load Test Result for PLT-4, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

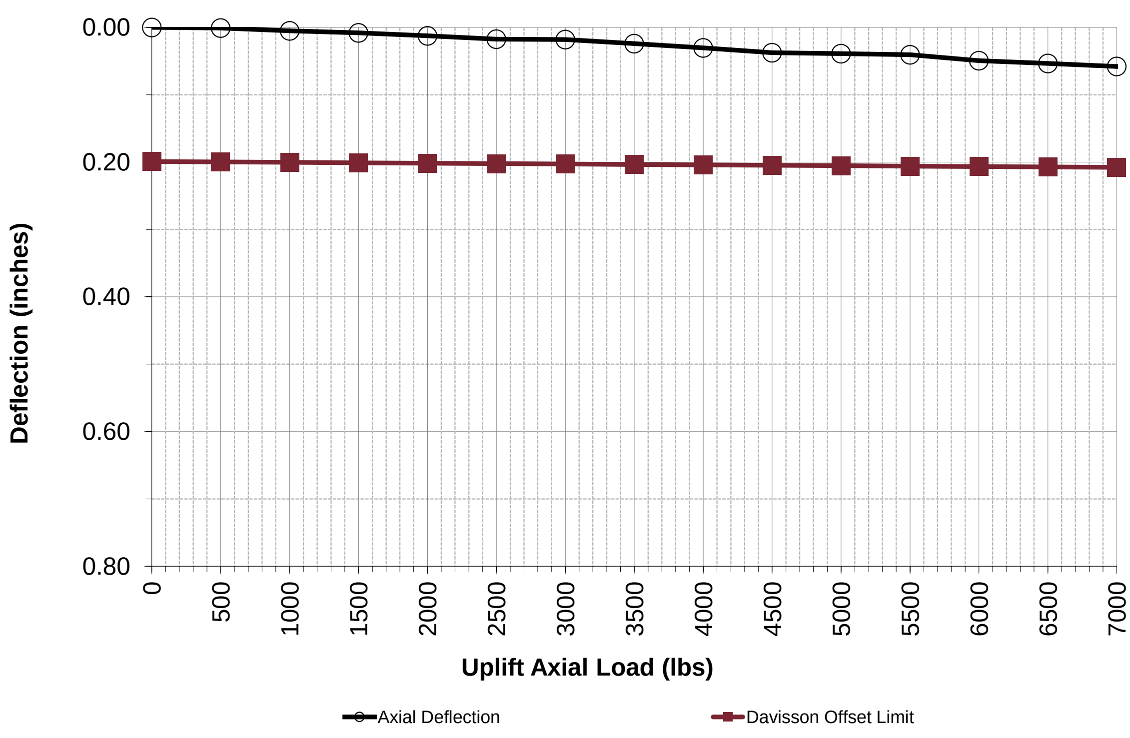
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-4, 12ft
Latitude:	43.16221
Longitude:	-120.45297
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	48
Axial Design Load [lbs]:	7000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	30

Tension Test Results			Davisson Offset Limit Lines		
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Elastic Data (in) (PL/AE)	Davisson Offset Limit (in) (0.15+D/120+(PL/AE))	Comments
0%	0	0.000	0.000	0.199	
7%	500	0.001	0.001	0.200	
14%	1000	0.005	0.001	0.200	
21%	1500	0.008	0.002	0.201	
29%	2000	0.013	0.002	0.202	
36%	2500	0.018	0.003	0.202	
43%	3000	0.018	0.004	0.203	
50%	3500	0.024	0.004	0.203	
57%	4000	0.031	0.005	0.204	
64%	4500	0.038	0.006	0.205	
71%	5000	0.039	0.006	0.205	
79%	5500	0.040	0.007	0.206	
86%	6000	0.050	0.007	0.207	
93%	6500	0.053	0.008	0.207	
100%	7000	0.058	0.009	0.208	



Compression Load Test Result for PLT-1, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

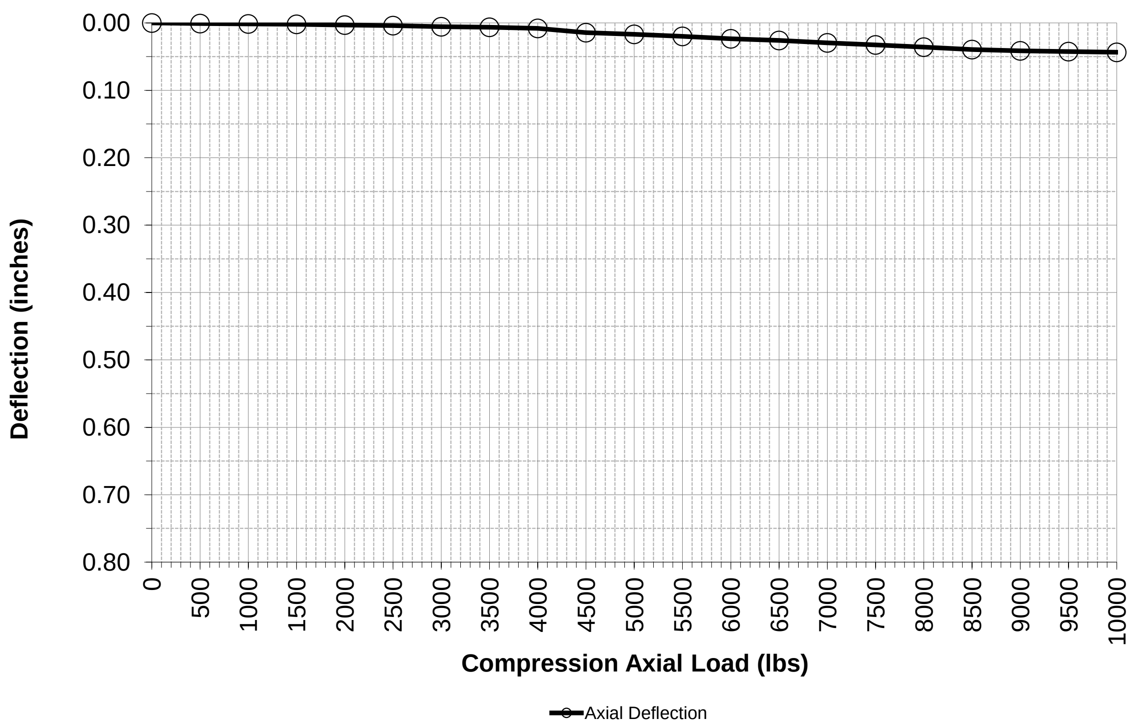
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-1, 6ft
Latitude:	43.21525
Longitude:	-120.45301
Pile Type:	6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	12
Axial Design Load [lbs]:	10000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	21

Compression Test Results			
% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Comments
0%	0	0.000	
5%	500	0.001	
10%	1000	0.002	
15%	1500	0.002	
20%	2000	0.003	
25%	2500	0.004	
30%	3000	0.006	
35%	3500	0.007	
40%	4000	0.008	
45%	4500	0.015	
50%	5000	0.017	
55%	5500	0.020	
60%	6000	0.024	
65%	6500	0.026	
70%	7000	0.030	
75%	7500	0.033	
80%	8000	0.036	
85%	8500	0.040	
90%	9000	0.042	
95%	9500	0.043	
100%	10000	0.044	



Compression Load Test Result for PLT-2, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

Test Date and Representative

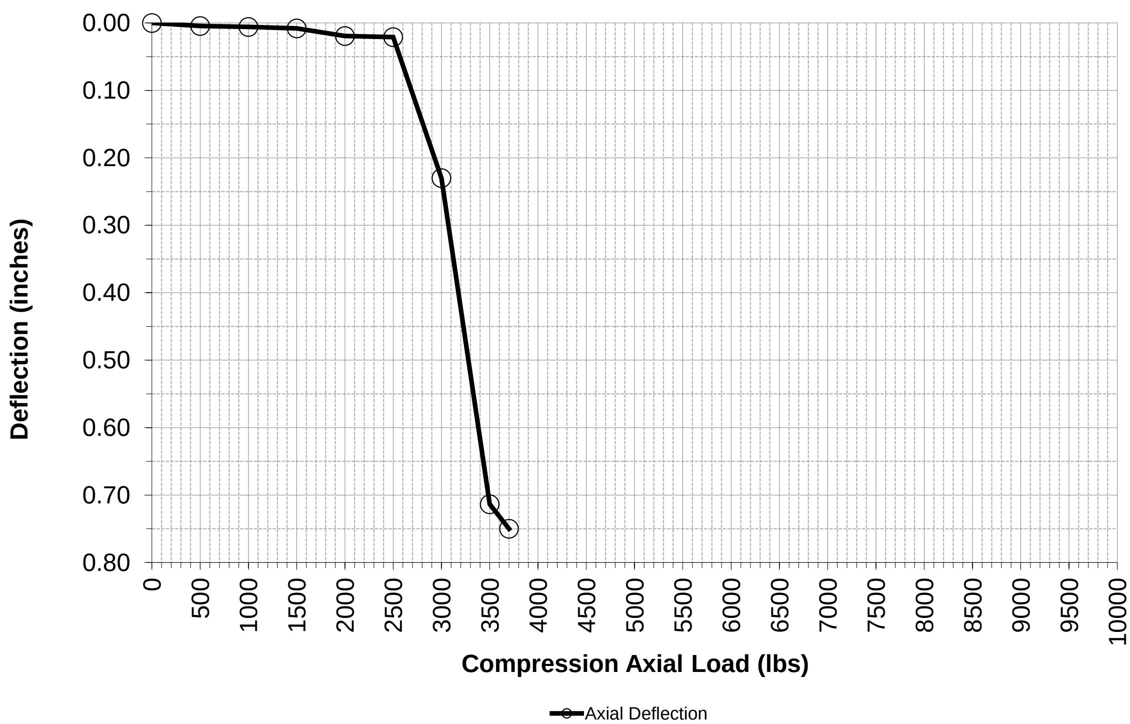
Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-2, 6ft
Latitude:	43.19860
Longitude:	-120.46182
Pile Type:	6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	12
Axial Design Load [lbs]:	10000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	10

Compression Test Results

% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Comments
0%	0	0.000	
5%	500	0.005	
10%	1000	0.006	
15%	1500	0.008	
20%	2000	0.020	
25%	2500	0.021	
30%	3000	0.230	
35%	3500	0.714	
37%	3700	0.750	
45%	4500		
50%	5000		
55%	5500		
60%	6000		
65%	6500		
70%	7000		
75%	7500		
80%	8000		
85%	8500		
90%	9000		
95%	9500		
100%	10000		



Compression Load Test Result for PLT-3, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

Test Date and Representative

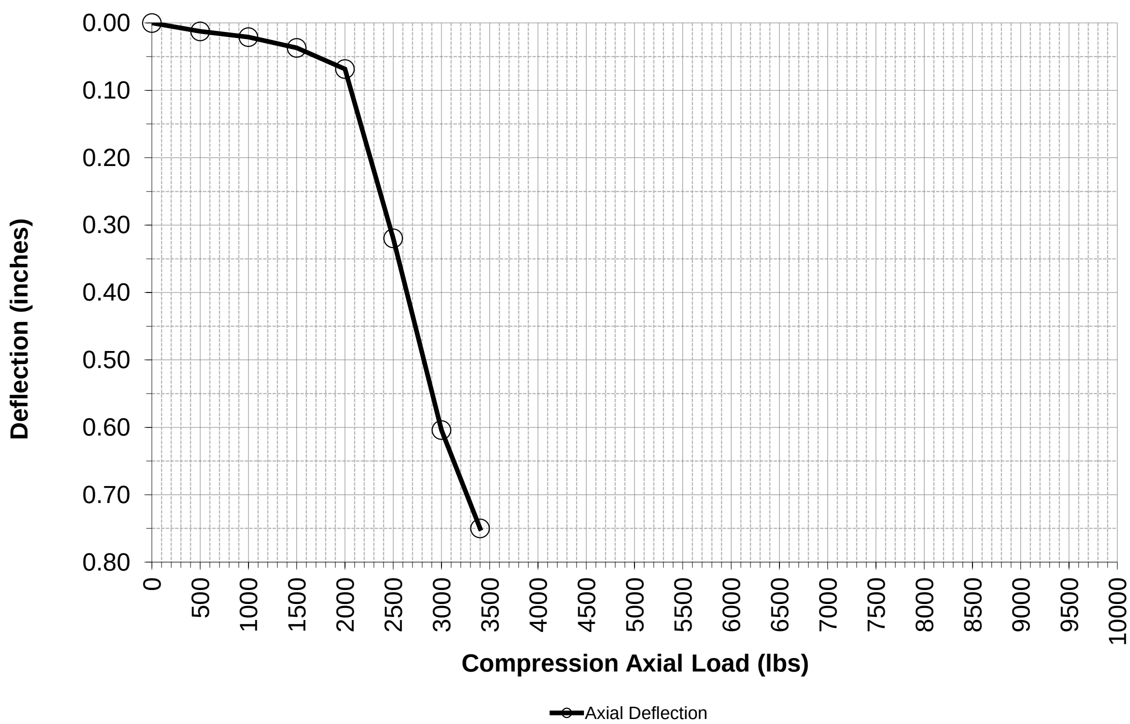
Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-3, 6ft
Latitude:	43.18648
Longitude:	-120.45688
Pile Type:	6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	12
Axial Design Load [lbs]:	10000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	9

Compression Test Results

% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Comments
0%	0	0.000	
5%	500	0.013	
10%	1000	0.021	
15%	1500	0.037	
20%	2000	0.069	
25%	2500	0.320	
30%	3000	0.604	
34%	3400	0.750	
40%	4000		
45%	4500		
50%	5000		
55%	5500		
60%	6000		
65%	6500		
70%	7000		
75%	7500		
80%	8000		
85%	8500		
90%	9000		
95%	9500		
100%	10000		



Compression Load Test Result for PLT-4, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Axial Load Test Set Up

Number of Gauges:	2
Height of Gauges [in]:	6
Load Cell:	25000

Test Date and Representative

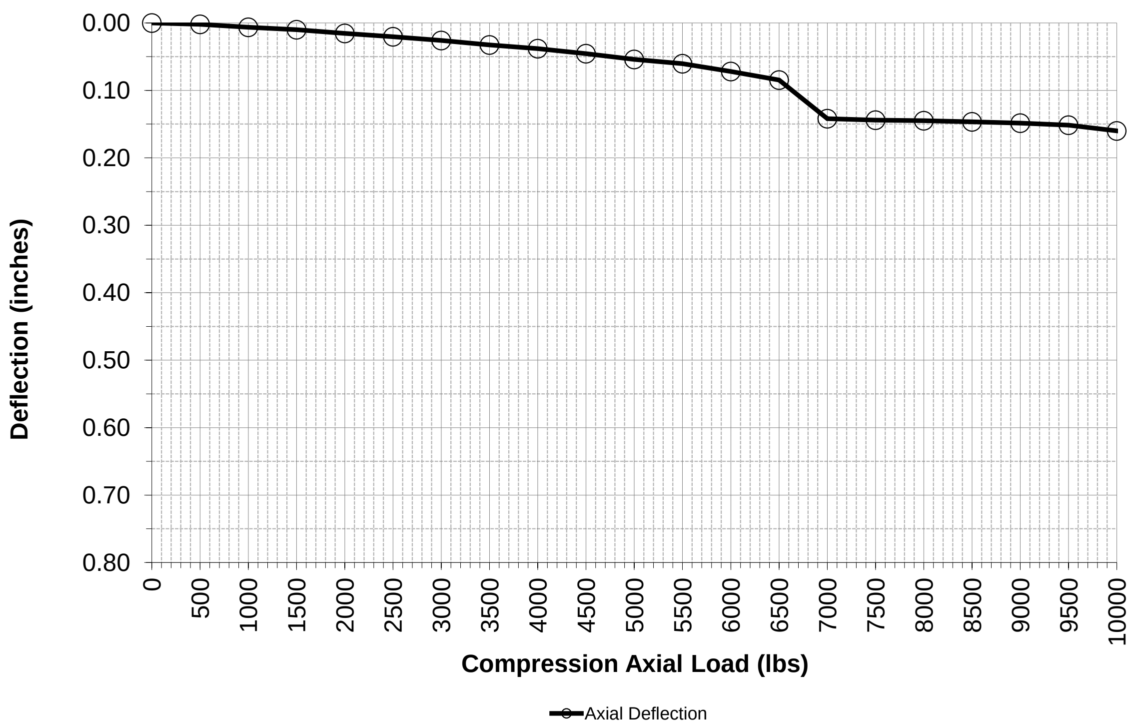
Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-4, 6ft
Latitude:	43.16221
Longitude:	-120.45297
Pile Type:	6x9
Pile Embedment Depth [in]:	60
Pile Diameter [in]:	5.9
Pile Stick-Up [in]:	12
Axial Design Load [lbs]:	10000
Pile Area [sq. in]:	2.68
Elastic Modulus [ksi]:	29,000
Drive Time [sec]:	16

Compression Test Results

% of Design Load	Axial Load [lbs]	Deflection Δ (in.) Gauges #1 & #2	Comments
0%	0	0.000	
5%	500	0.002	
10%	1000	0.007	
15%	1500	0.010	
20%	2000	0.016	
25%	2500	0.021	
30%	3000	0.026	
35%	3500	0.033	
40%	4000	0.038	
45%	4500	0.046	
50%	5000	0.054	
55%	5500	0.061	
60%	6000	0.072	
65%	6500	0.085	
70%	7000	0.142	
75%	7500	0.144	
80%	8000	0.145	
85%	8500	0.147	
90%	9000	0.149	
95%	9500	0.152	
100%	10000	0.160	



Lateral Load Test Result for PLT-1, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

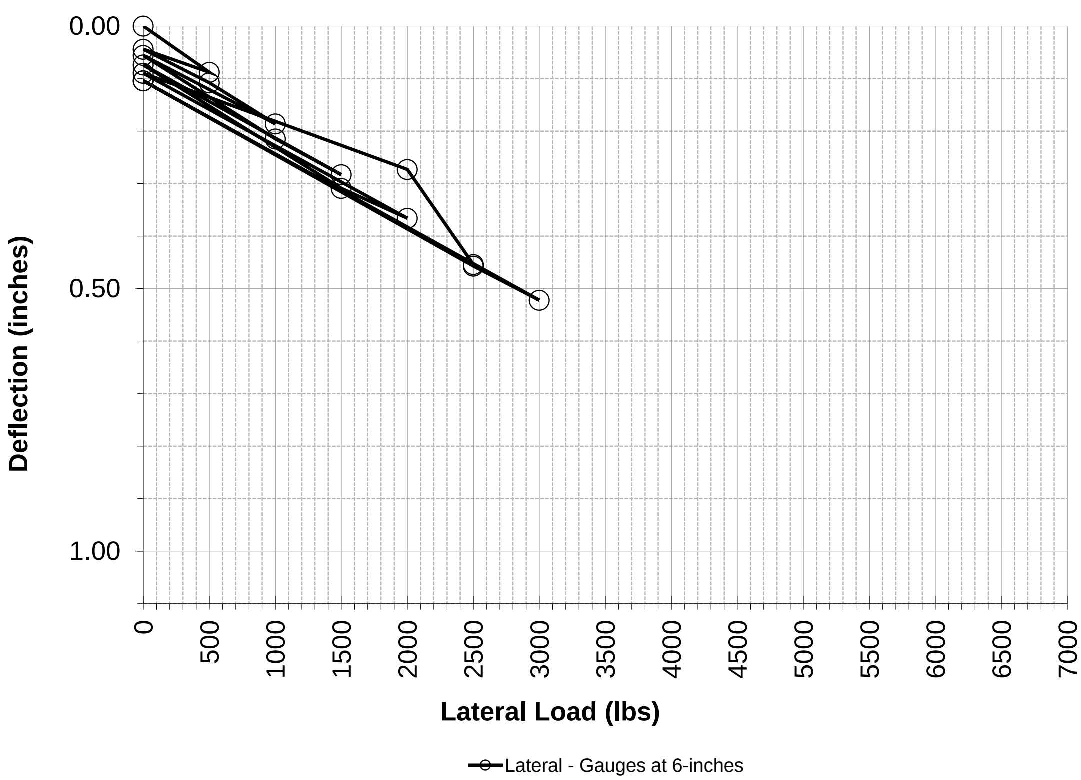
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-1, 9ft
Latitude:	43.21525
Longitude:	-120.45301
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	29

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
7%	500	0.088	
0%	0	0.044	
7%	500	0.108	
14%	1000	0.186	
0%	0	0.056	
14%	1000	0.215	
21%	1500	0.283	
0%	0	0.074	
21%	1500	0.309	
29%	2000	0.366	
0%	0	0.090	
29%	2000	0.373	
36%	2500	0.454	
0%	0	0.104	
36%	2500	0.457	
43%	3000	0.522	
0%	0	0.104	



Lateral Load Test Result for PLT-1, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

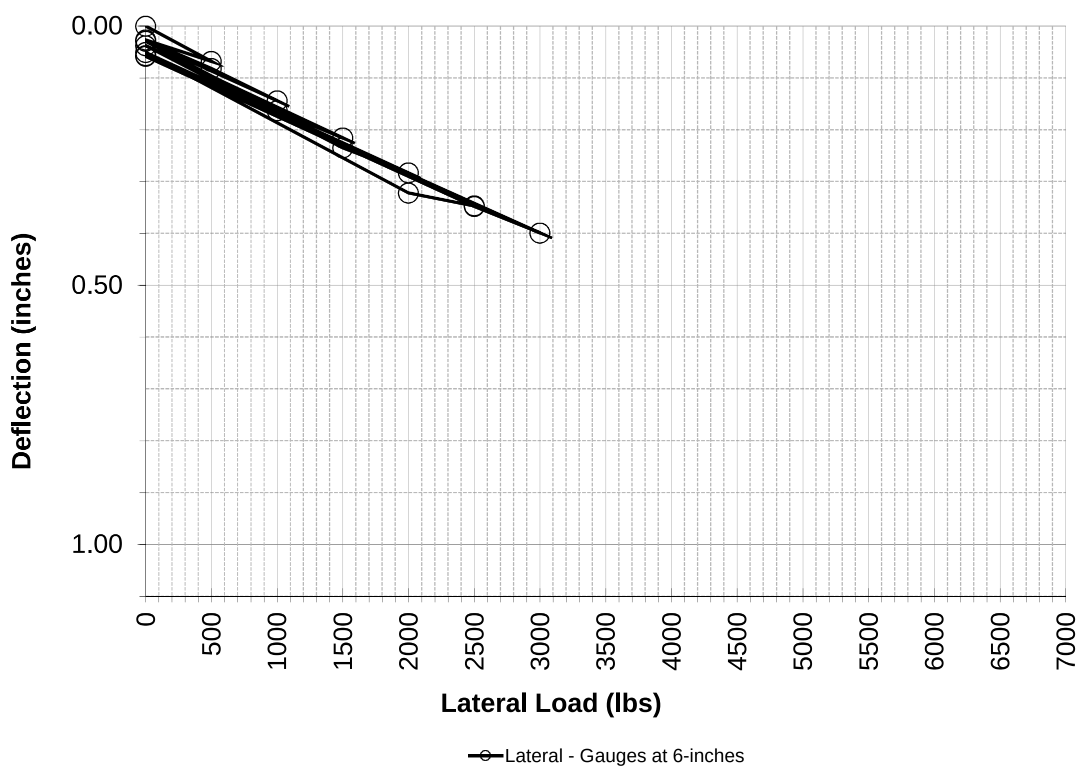
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-1, 12ft
Latitude:	43.21525
Longitude:	-120.45301
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	77

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
7%	500	0.068	
0%	0	0.027	
7%	500	0.082	
14%	1000	0.144	
0%	0	0.029	
14%	1000	0.163	
21%	1500	0.216	
0%	0	0.038	
21%	1500	0.235	
29%	2000	0.283	
0%	0	0.051	
29%	2000	0.322	
36%	2500	0.347	
0%	0	0.058	
36%	2500	0.348	
43%	3000	0.399	
0%	0	0.057	



Lateral Load Test Result for PLT-2, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

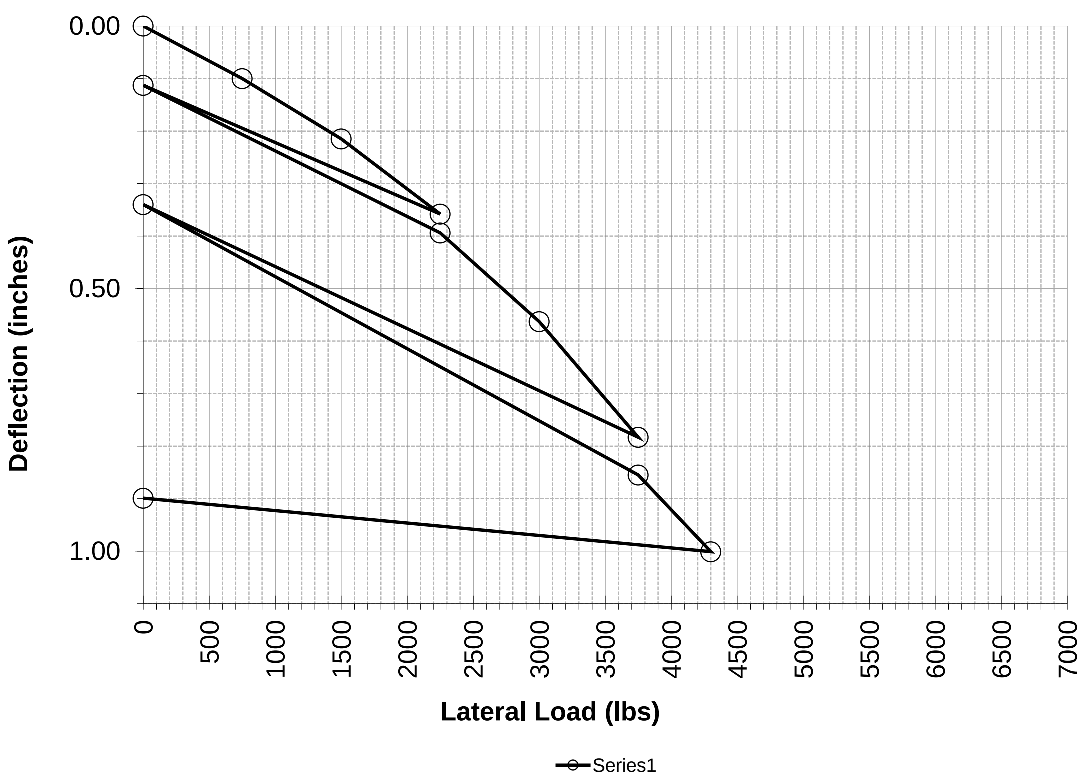
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-2, 9ft
Latitude:	43.19860
Longitude:	-120.46182
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	9

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.100	
21%	1500	0.215	
32%	2250	0.358	
0%	0	0.113	
32%	2250	0.394	
43%	3000	0.563	
54%	3750	0.783	
0%	0	0.340	
54%	3750	0.855	
61%	4300	1.001	
75%	5250		
0%	0	0.899	
75%	5250		
86%	6000		
100%	7000		
0%	0		



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Lateral Load Test Result for PLT-2, 8ft Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

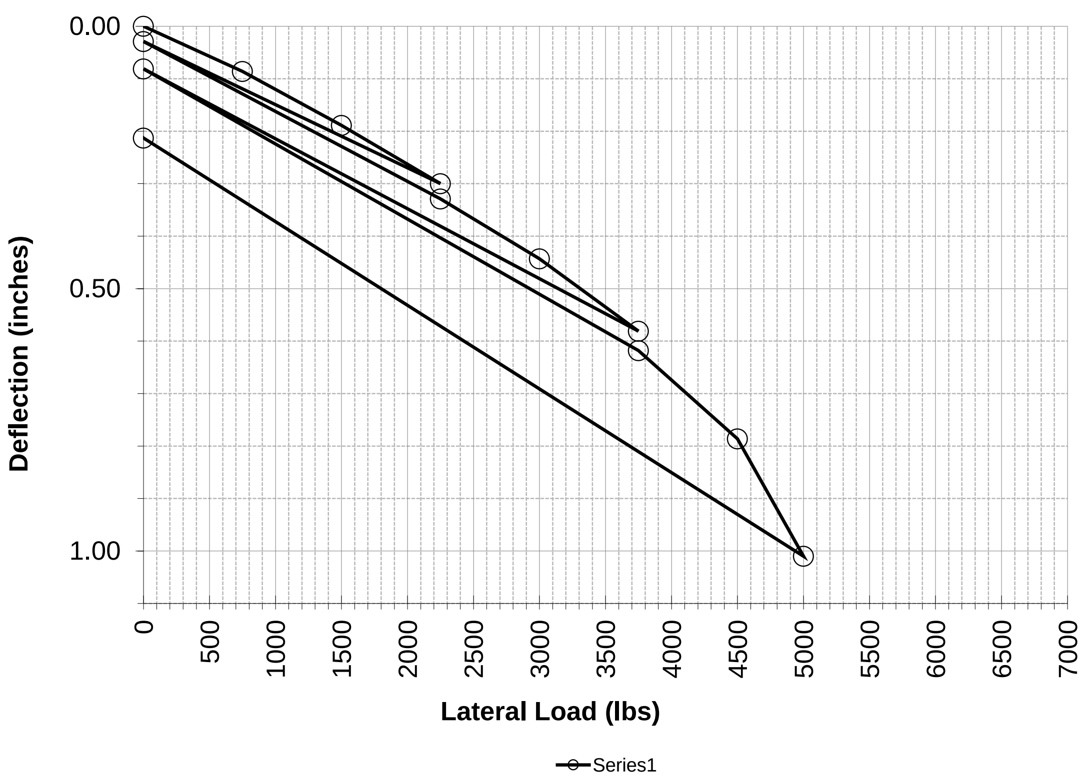
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-2, 12ft
Latitude:	43.19860
Longitude:	-120.46182
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	21

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.086	
21%	1500	0.189	
32%	2250	0.300	
0%	0	0.029	
32%	2250	0.329	
43%	3000	0.443	
54%	3750	0.581	
0%	0	0.081	
54%	3750	0.618	
64%	4500	0.786	
71%	5000	1.010	
0%	0	0.213	
75%	5250		
86%	6000		
100%	7000		
0%	0		



Lateral Load Test Result for PLT-3, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

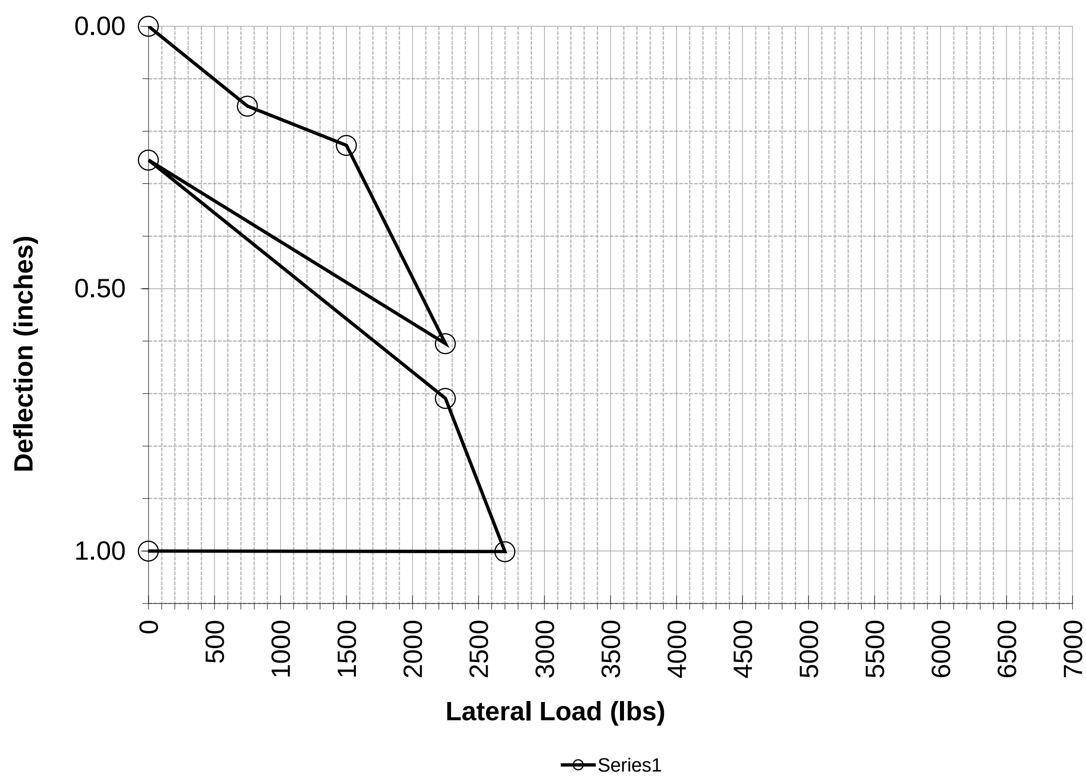
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-3, 9ft
Latitude:	43.18648
Longitude:	-120.45688
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	8

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.152	
21%	1500	0.227	
32%	2250	0.605	
0%	0	0.255	
32%	2250	0.709	
39%	2700	1.001	
54%	3750		
0%	0	1.000	
54%	3750		
64%	4500		
75%	5250		
0%	0		
75%	5250		
86%	6000		
100%	7000		
0%	0		



Lateral Load Test Result for PLT-3, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

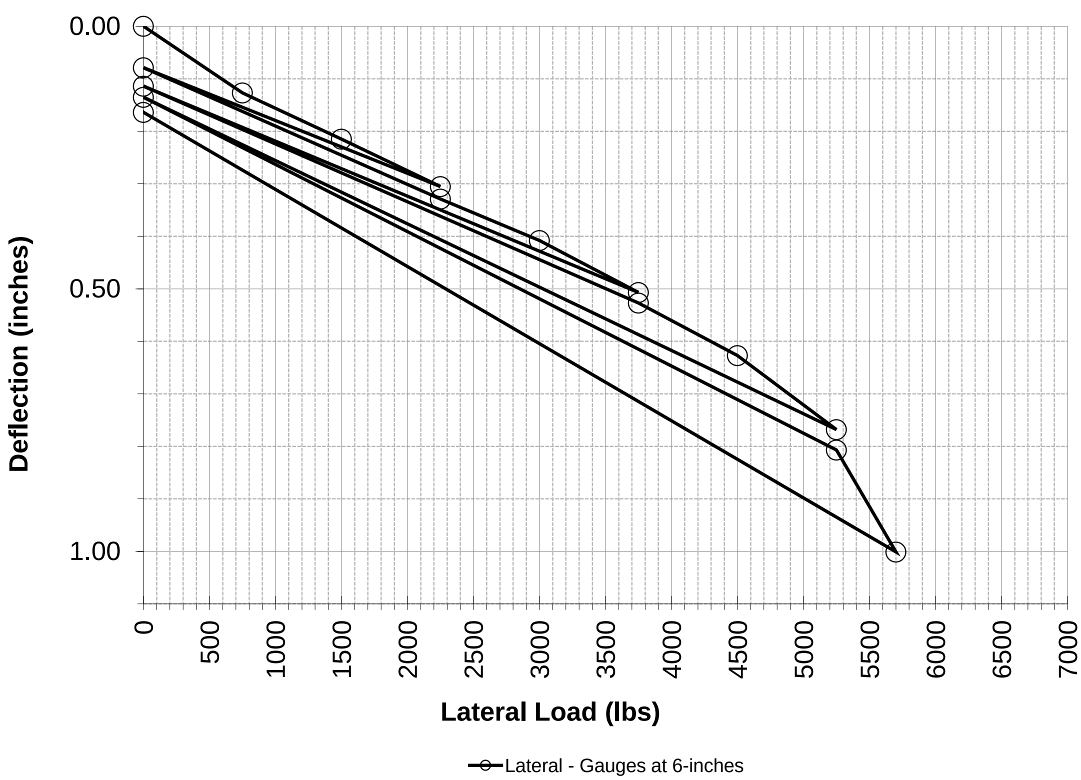
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-3, 12ft
Latitude:	43.18648
Longitude:	-120.45688
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	17

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.127	
21%	1500	0.215	
32%	2250	0.305	
0%	0	0.079	
32%	2250	0.329	
43%	3000	0.408	
54%	3750	0.507	
0%	0	0.114	
54%	3750	0.527	
64%	4500	0.627	
75%	5250	0.768	
0%	0	0.135	
75%	5250	0.807	
81%	5700	1.001	
100%	7000		
0%	0	0.164	



Lateral Load Test Result for PLT-4, 5ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

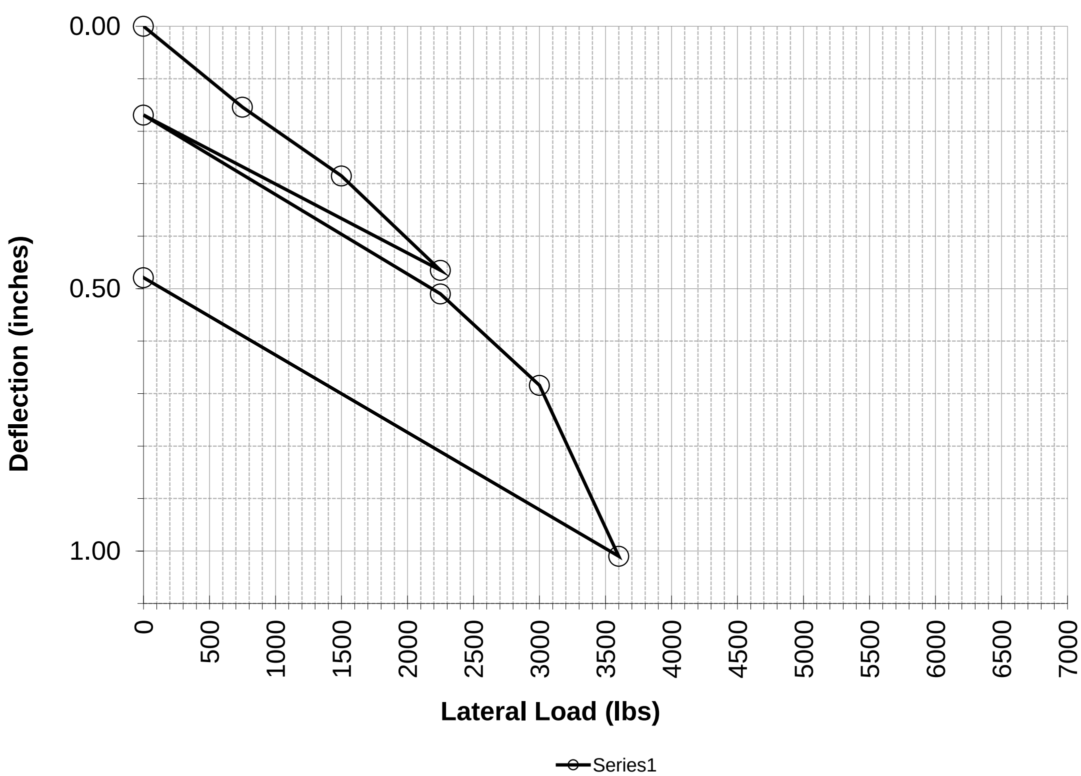
Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-4, 9ft
Latitude:	43.16221
Longitude:	-120.45297
Pile Type:	W6x9
Pile Embedment Depth [in]:	60
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	14

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.154	
21%	1500	0.285	
32%	2250	0.465	
0%	0	0.169	
32%	2250	0.510	
43%	3000	0.684	
51%	3600	1.010	
0%	0	0.479	
54%	3750		
64%	4500		
75%	5250		
0%	0		
75%	5250		
86%	6000		
100%	7000		
0%	0		



Lateral Load Test Result for PLT-4, 8ft. Embedment

Project Information

Project Name:	Archway Solar
Project Location:	Christmas Valley, Oregon
Project Number:	82185058

Lateral Load Test Set Up

Number of Top Gauges:	0
Number of Bottom Gauges:	1
Height of Top Gauges [in]:	60
Height of Bottom Gauges [in]:	6
Height of Applied Load [in]:	48
Load Cell:	25000

Test Date and Representative

Tested By Terracon Rep:	Jachin
Date Tested:	1/29/2020

Pile Information

Pile ID:	PLT-4, 12ft
Latitude:	43.16221
Longitude:	-120.45297
Pile Type:	W6x9
Pile Embedment Depth [in]:	96
Pile Stick-Up [in]:	48
Lateral Design Load [lbs]:	7000
Drive Time [sec]:	30

% of Design Load	Lateral Load [lbs]	Deflection Δ (in.) Gauge #1	Comments
0%	0	0.000	
11%	750	0.109	
21%	1500	0.205	
32%	2250	0.334	
0%	0	0.059	
32%	2250	0.364	
43%	3000	0.470	
54%	3750	0.604	
0%	0	0.094	
54%	3750	0.684	
64%	4500	0.718	
73%	5100	1.001	
0%	0	0.159	
75%	5250		
86%	6000		
100%	7000		
0%	0