

Rex Solaire Solar and Storage Project

Initial Study

prepared for

Kings County
Community Development Agency
1400 West Lacey Boulevard, Building #6
Hanford, California 93230
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prepared with the assistance of

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RINCON CONSULTANTS

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Initial Study

1. Project Title

Rex Solaire Solar and Storage Project

2. Lead Agency Name and Address

Kings County
Community Development Agency
1400 West Lacey Boulevard, Building #6
Hanford, California 93230

3. Contact Person and Phone Number

Kings County, Community Development Agency
Contact: Ms. Noelle Tomlinson, Planner
Telephone: 559-852-2670

4. Project Sponsor's Name and Address

Rex Solaire Solar and Storage, LLC
4275 Executive Square, Suite 400
La Jolla, California 92037
Contact: Simon Day, Vice President

5. Project Location

The Project site is located on approximately 20,600 acres of agricultural land in unincorporated southern Kings County (County), along the eastern edge of Interstate 5 (I-5). A list of Assessor's Parcel Numbers (APNs) is included as Appendix A. The Project site is located approximately 10 miles southeast of the community of Kettleman City, approximately 10 miles west of the community of Alpaugh, approximately 18 miles southeast of the city of Avenal, approximately 13 miles north of the community of Lost Hills, and approximately 3 miles east of the California Aqueduct. State Route 41 is located approximately 8 miles northwest of the Project site, State Route 33 is located approximately 9 miles southwest of the Project site, State Route 46 is located approximately 12 miles south of the Project site, and State Route 43 is located approximately 15 miles to the east of the Project site. The Project site borders Kern County to the south. The Project site is accessible from I-5 via Utica Avenue, located adjacent to the north of the Project site. Figure 1 shows the regional location of the Project site. Figure 2 shows the Project site in a local context.

Figure 1 Regional Location

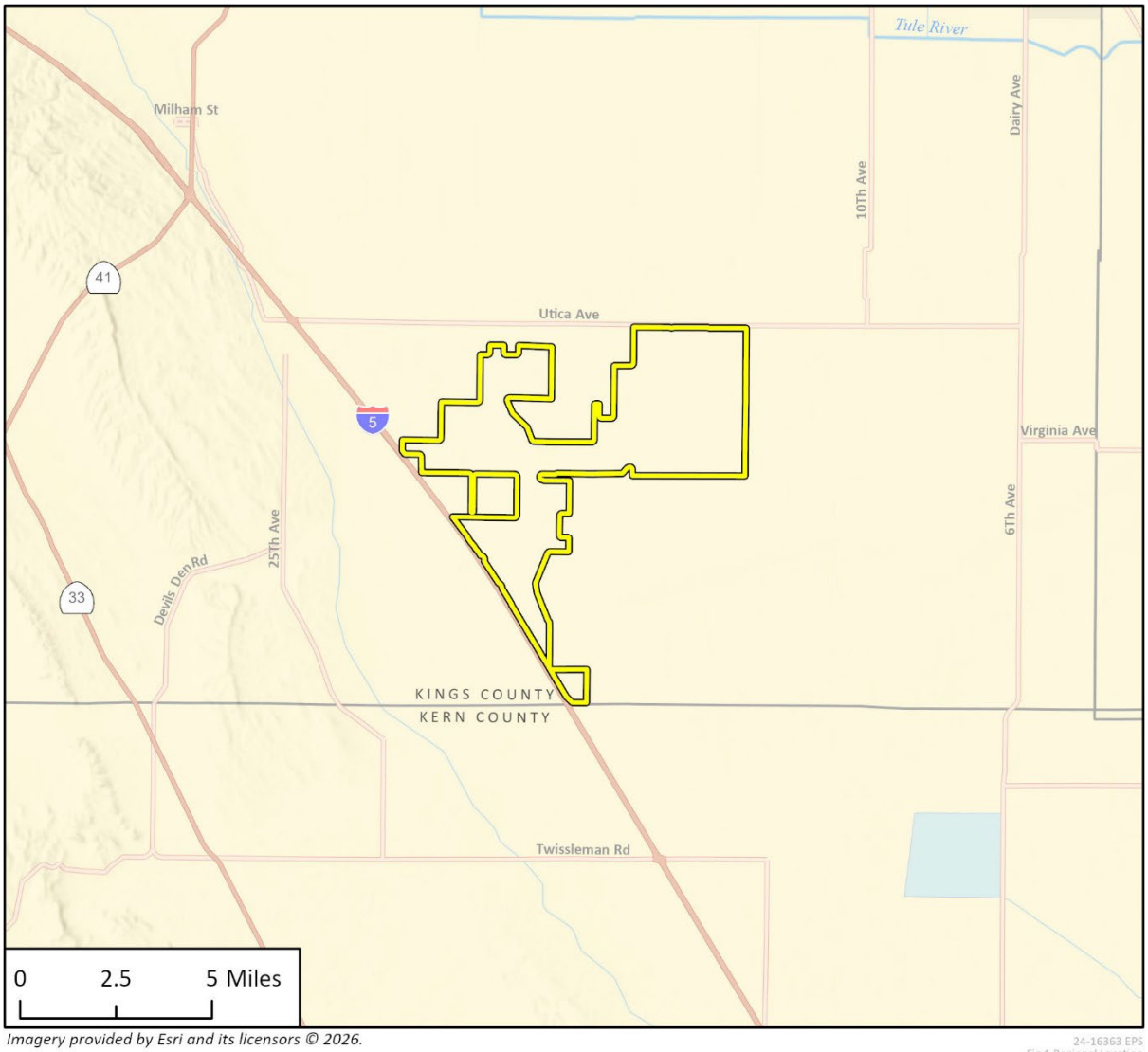
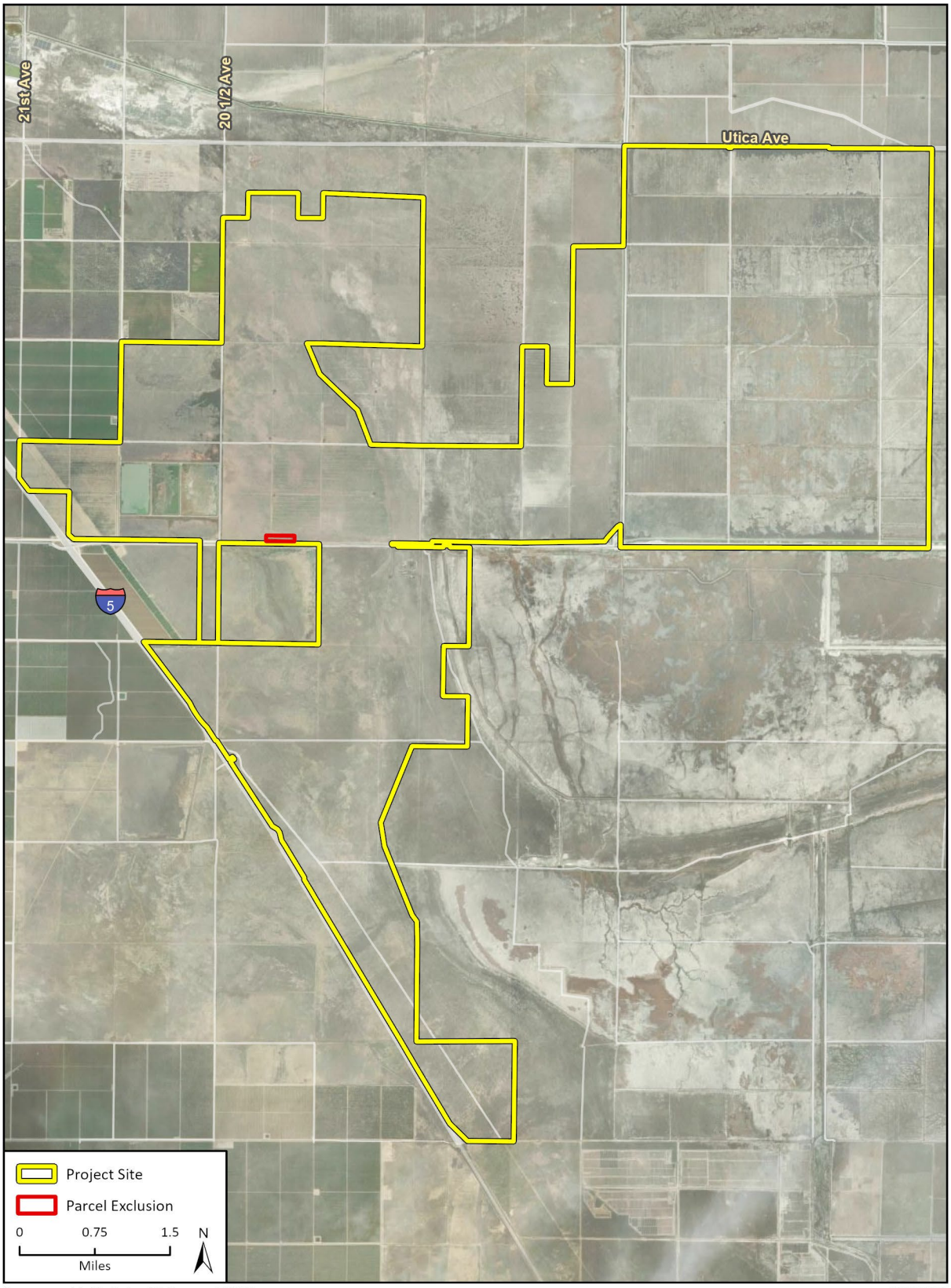


Figure 2 Project Site



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24-16363 EPS
Fig 2 Project Site

6. Surrounding Land Uses and Setting

Onsite, adjacent, and nearby uses, including land use designations and zoning, are summarized in Table 1. The entire Project site and the surrounding half-mile radius are designated General Agriculture – 40 Acre by the Kings County General Plan (Kings County 2010a).

Table 1 Project Site and Surrounding Land Uses

Location	General Plan Designations	Zone Classifications	Land Use
Project Site	General Agriculture – 40 acre	AG-40 (Agricultural, 40-acre minimum parcel size)	Land uses within the Project site include grazing grasslands, fallow fields, irrigation ditches/canals, and undeveloped land.
North	General Agriculture – 40 acre	AG-40 (Agricultural, 40-acre minimum parcel size)	Utica Avenue is located directly north of the Project site. Land uses beyond Utica Avenue include grazing grasslands and irrigation ditches/canals.
East	General Agriculture – 40 acres	AG-40 (Agricultural, 40-acre minimum parcel size)	Land uses to the east of the Project site include fallow fields, undeveloped land, and irrigation ditches/canals.
South	Kern County Map Code 8.1 (Intensive Agriculture), & Map Code 8.3 (Extensive Agriculture)	County (A) Exclusive Agriculture District, 20-acre minimum parcel size	Land uses south of the Project site are located in Kern County and include shrublands, grasslands, and seasonal wetlands.
West	General Agriculture – 40 acres	AG-40 (Agricultural, 40-acre minimum parcel size)	Interstate 5 is located directly west of the Project site. Just beyond, the California Aqueduct runs parallel to the interstate. The Jackson Ranch Specific Plan area is located between the aqueduct and interstate. Land uses west of the Project site include active agricultural fields/orchards, irrigation canals, and undeveloped land.

7. General Plan Designation

The entirety of the Project site has a General Plan land use designation of General Agriculture – 40 acre.

8. Zoning

The entirety of the Project site is zoned AG-40 (Agricultural, 40-acre minimum parcel size).

9. Description of Project

Overview

The Project is a utility-scale photovoltaic (PV) solar power generation and energy storage facility that consists of tracking solar PV panel arrays with a nominal output of up to 2,800 megawatts[1] (MW) alternating current (AC) at the point of interconnection and battery energy storage with a capacity of up to 11.2 gigawatt-hours (GWh).

The Project includes the following components, which are described in more detail in the following subsections:

- **Solar PV Generating Facilities:** solar PV modules made of thin film or crystalline silicon material, covered by anti-reflective glass, and mounted on single-axis racking systems; and inverters which convert direct-current (DC) energy produced by the solar modules to alternating-current (AC) used by the grid.
- **Energy Storage:** a battery energy storage system (BESS) and accessory infrastructure within a graveled area, with dedicated inverters, medium voltage collector lines, potentially with a long duration energy storage component, in a centralized energy storage facility located next to Project substations.
- **Project Substations:** up to seven onsite substation facilities that are the termination points of the collection system of medium voltage (34.5 kilovolt [kV] AC) electricity. At the substations, the output of the field of interconnected arrays and batteries are passed through an interconnection step-up transformer to convert it from medium voltage (34.5 kV) to high voltage (230 kV and/or 500 kV).
- **Power Collection, Interconnection, Switchyard, and Transmission:** overhead and underground collection system to collect power from the solar arrays and inverters. This power would be stepped up in voltage from medium voltage (34.5 kV) to high voltage, either 230 kV or 500 kV, at Project substation(s). A gen-tie line would connect the Project substation(s) to a switching station located adjacent to the PG&E transmission corridor. The switching station would connect the Project to the PG&E transmission system while also allowing the Project to be electrically isolated from the transmission system for maintenance of the Project or connected transmission facilities.
- **Telecommunication Facilities:** telecommunication equipment, including underground and overhead fiber optic cables and meteorological data collection systems (supervisory control and data acquisition [SCADA]). The SCADA system allows for remote monitoring and control of Project facilities.
- **Operations & Maintenance (O&M) Facility:** facility used for maintenance of Project facilities including permanent laydown areas, onsite office space, parking, and equipment storage and maintenance. This area may also include tanks to store onsite water for fire suppression.
- **Site Access and Security:** onsite access roads, perimeter security fencing, and nighttime directional lighting.
- **Temporary Construction Facilities:** onsite laydown areas, parking, and office trailers.

[1] The megawatt capacity is an estimate based on currently available technology. The solar PV industry has evolved substantially in the last few years and is anticipated to continue to evolve. While the components and total capacity of the Project may change, the overall size of the Project (up to 20,528-acres) would remain consistent.

Solar PV Generating Facilities

The Project would include a PV solar technology designed in a collection of subarrays which would convert solar energy directly to electrical power for eventual export to the electrical grid. The Project would include installation of PV modules that would be mounted on steel support posts that would be pile-driven into the ground. The modules would be made of a thin film material or crystalline silicon material covering the glass panes, which would be dark in color, highly absorptive, and have minimum reflectivity. The modules would be manufactured at an offsite location and transported to the site. Each sub-array of solar panels, often called "blocks" would be connected to an inverter.

Depending on the type of modules used, the panels would measure approximately 4 feet wide by 7.5 feet long, and the total height of the panel system, measured from the ground surface, would be up to 12 to 13 feet in height, with 8 to 10 feet being more common. The length of each row of panels would vary based on final site configuration and detailed engineering. Panels would be oriented in the north-south direction to maximize efficacy. As technology continues to advance, upgraded system dimensions that enable enhanced efficiency and output without expanding the Project development footprint may be utilized.

Spacing between each row would be approximately 10 to 30 feet to allow access for maintenance and cleaning, and to reduce shading between rows. Single-axis tracking systems would include a motor mechanism that would allow the arrays to track the path of the sun from east to west throughout the day with a range of up to 110 degrees. The motors would be installed after the horizontal cross-members are in place. In the morning, the panels would face east. The panels would slowly move to the horizontal position at noon and on to the west at sundown. The panels would stow horizontally overnight and as operational conditions dictate. The panels would reset each morning to receive direct sunlight at sunrise.

Energy Storage

The proposed BESS would use DC-coupled and/or AC-coupled battery storage. The DC BESS would be distributed and located throughout the solar arrays, typically near the inverter for each block (sub-array) of panels. The AC BESS would be centralized, potentially including long-duration energy storage facilities, located close to the Project substations.

The BESS would be comprised of batteries housed in outdoor cabinet-type enclosure. Typically, such BESS systems are based on lithium-ion battery modules installed in racks and housed in outdoor BESS cabinets. A typical BESS cabinet would house battery modules collectively capable of storing between 0.4 to 5 megawatt-hours of energy. The modules within each cabinet would be accessed for maintenance from the exterior via cabinet doors.

To ensure safe and efficient operations, each individual battery module within a cabinet would be monitored and controlled both locally by a UL listed battery management system and remotely via a SCADA system discussed in further detail below, and every cabinet would be equipped with integrated operational management systems and fire and life safety systems, such as heating ventilation and cooling (HVAC); gas and smoke detection; deflagration prevention; and alarms.

The battery cabinets are attached to inverters which convert DC to AC (and vice-versa when charging). Starting approximately 5 years after original installation, the Project Sponsor may be required to install additional battery modules and cabinets to augment lost storage capacity, which occurs naturally as the batteries are used and degraded. BESS augmentation would occur within

areas designated for BESS, as shown on the site plan (Appendix B), and would ensure the Project can maintain the full energy storage capacity.

The dimensions of a typical BESS cabinet vary between manufacturers and are typically arranged in repeated "blocks" across the BESS site. System blocks may consist of a single "large" cabinet, or several smaller cabinets set side-by-side to create banks of batteries with similar overall dimensions (i.e., a "skid). Smaller cabinets are typically closely spaced and may be physically attached at the time of construction or during assembly at the Original Equipment Manufacturer (OEM) factory; larger cabinets are placed in smaller groupings or individually. A cabinet can measure up to 22 feet in height, though most current OEM products are around 10 feet in height. The number, size, layout, spacing, and capabilities of the cabinets would vary depending on the battery cabinet manufacturer design, large scale fire testing results, EPC design, and BESS system manufacturer(s) selected for the Project. Regardless of the system manufacturer, the Project's developed footprint and overall capability would remain substantially the same throughout the life of the Project. The associated inverters, transformers, and switchgear would be located immediately adjacent to the structure on concrete pads or on driven pier foundations.

The Project and its systems would be designed, constructed, maintained and operated pursuant to the current California and local building code, California Fire Code with any local amendments, and the National Fire Protection Association (NFPA) 855 requirements. The California Fire Code and NFPA 855 specifies, but is not limited to, maximum limits on sizing for battery systems, seismic and structural design, spacing, product testing, product listings, commissioning and decommissioning, maintenance and repairs, fire alarm system requirements, fire water supply, and deflagration control requirements.

Power to the BESS auxiliary systems (e.g., HVAC, fire alarm, SCADA, etc.) would be provided via a connection to an onsite Project substation with connection lines installed above ground and/or below ground. Backup generators for the BESS may be required by the Authority Having Jurisdiction to provide backup emergency electricity for critical safety systems. If required, the generators would use diesel fuel and would need to be intermittently run for maintenance and testing purposes. The generators would operate in accordance with applicable California Air Resources Board and San Joaquin Valley Air Pollution Control District (SJVAPCD) regulations and appropriate air permits would be obtained from SJVAPCD. The BESS would generally be unstaffed, with remote operational control and periodic inspections and maintenance performed, as necessary. Fire and life safety systems would be inspected, tested, and maintained in accordance with the fire code and relevant NFPA Standards such as NFPA 72, NFPA 69, and NFPA 25.

Project Substations

The onsite Project substations are the termination points of the collection system of medium voltage (34.5 kV AC) electricity. At the substations, the output of the field of interconnected arrays and batteries are passed through an interconnection step-up transformer to convert it from medium voltage (34.5 kV) to high voltage 230 kV and/or 500 kV. Up to seven onsite substations may be needed for the Project, pending final engineering. Potential locations of Project substations are shown within the Preliminary Site Plan (Appendix B). Project substations would host the grid intertie safety equipment and switches required to interconnect the Project to the high-voltage transmission system. Generators for backup emergency power may be required at the Project substations. Generators would operate in accordance with applicable California Air Resources Board and SJVAPCD regulations and appropriate air permits would be obtained from SJVAPCD. The

footprint for each Project substation would be sized to accommodate all required equipment with safety-related setbacks.

Power Collection, Interconnection, Switchyard, and Transmission

An additional 230 kV and/or 500 kV switchyard is anticipated to be required to connect the Project gen-tie circuits into the PG&E transmission system. This switchyard would be located in proximity to the existing PG&E transmission lines and is likely to be located three to five miles southeast of the Project substation. A number of 230 kV and/or 500 kV high-voltage gen-tie lines would exit the Project substation and travel southeast to join the proposed switchyard where power would be delivered to the electrical grid. The switching station would require several transmission structures to "break" the existing 230 kV and/or 500 kV lines, route them through the switchyard, and reconnect the PG&E transmission system. The switchyard would consist of equipment, buildings and related appurtenances commonly found to facilitate the connection to a high-voltage transmission system including but not limited to operational buildings, disconnect switches, station service voltage transformers, coupling capacitor voltage transformers, surge arrestors, capacitor banks, breakers, communication equipment, fiber, and back-up generators. The switchyard is anticipated to require up to 100 acres and would ultimately be owned and operated by PG&E following construction. Any new transmission structures are anticipated to be predominately less than 200 feet tall, with the potential for up to 300 feet; and generally would match the height and arrangement of PG&E's existing facilities. The final design, layout, and required interconnection components would be determined during the detailed engineering design stage.

Telecommunications Facilities

The SCADA system functions as a remote start, stop, reset, and safety lock out for the facility, and provides site diagnostic information from the energy generating facility. The SCADA system would also control the Project substations, allowing for fully centralized operation of the Project to meet all California Independent System Operator and utility interconnection requirements. The SCADA system is critical to the California Independent System Operator and PG&E utility interconnection and provides for the proper operation and maintenance of the Project. The system uses proprietary software; fiberoptic communication lines; and communication systems such as telephone and/or radio link.

The Project would use local exchange carrier services for telecommunication to support remote monitoring requirements. The cabinet holding the connection equipment would have a base of approximately 4 feet by 2 feet and would be approximately 5 feet in height. From the point of demarcation, an underground or overhead fiberoptic cable would be installed within the Project footprint to connect the cabinet to the SCADA equipment within the onsite Project substations.

Operations and Maintenance Facility

It is anticipated that approximately 10 to 20 permanent, full-time employees would be needed to maintain the Project. A permanent, onsite O&M building would be built to facilitate rapid repair of equipment. The O&M building would be approximately 8,000 square feet and up to 25 feet tall and would include offices, workshops, and conference rooms. The O&M building is expected to be a prefabricated commercial structure. Permanent restroom facilities with a 1,500 gallon septic tank and/or portable toilets would be used for sanitary purposes, and a permanent water source in the form of trucked water, well water (from onsite and/or offsite wells), or bottled water would be provided for staff. The proposed O&M facility would include the requisite number of parking spaces

for staff and visitor vehicles and O&M equipment and would comply with Americans with Disability Act requirements. Power for the O&M building would be provided by either the Project's electrical generation or supplied by the local power provider. A maintenance shop of up to 20,000 square feet and outdoor storage yard may also be built. A back-up generator for critical and emergency facility services with a fuel storage tank may be installed, if required. The O&M building would also include approved, non-occupied storage outbuildings for hazardous waste, including lithium-ion battery storage and associated containment for oil and fuels and vehicle maintenance. The O&M building would likely be sited near the Project entrance along Utica Avenue or near one of the Project substations. The precise location of the O&M building within this footprint would be determined at a later stage of design.

Site Access and Security

Access to the Project site would be provided via the existing Utica Avenue exit from I-5. Although existing roadways would be used to the greatest extent possible, new unpaved Project access roads would be constructed to provide internal access from the existing road network. Project access roads would be up to 30 feet wide and would be cleared and compacted for travel and access to the solar blocks. Project access roads will comply with Kings County Fire Department requirements. These Project access roads would remain in place for ongoing O&M activities after construction is completed. Dirt roads that are located within the boundaries of the Project, and that are not the subject of a written easement, would be closed to third-party use for vehicular or other access. Any such third parties that may currently use these dirt roads would be rerouted to public access routes, which would remain open.

Final Project access road alignments would depend on the final placement of the solar panels and on the results of field investigations, including topography and any other site-specific details to be incorporated into the final design. Should Project access roads be required to cross waters of the State under the jurisdiction of the Regional Water Quality Control Board and/or the California Department of Fish and Wildlife, appropriate crossings would be installed to minimize impacts to these jurisdictional areas and would comply with all regulatory requirements, including avoidance, minimization, and compensatory mitigation requirements set forth in any permit issued to the Project by said agencies.

Fencing would be installed around the perimeter of the Project site, substation, switchyard, and other areas requiring controlled access, as needed for safety and security purposes. Fencing would remain for the life of the Project. Final fence design will be developed in consultation with appropriate agencies.

Parcel maps would be recorded to eliminate public access easements on properties within the Project site. At this time, the Project Sponsor is not proposing street vacations beyond those public access easements included in the parcel maps. These parcel map requests would not impede public egress or ingress to any adjacent, non-Project properties.

Permanent motion-sensitive, directional security lights would be installed to provide adequate illumination around the substation areas and points of ingress/egress. All lighting would be shielded and directed downward to minimize the potential for glare or spillover onto adjacent properties. Lighting would be used from dusk to dawn once the Project is operational. Motion-sensitive cameras may also be installed throughout the solar arrays at the inverters for added security. If installed, cameras would be mounted on poles approximately 8-feet in height.

Project Construction

Temporary Construction Facilities

LAYDOWN AREAS

Construction laydown areas would be located at convenient locations within the Project site for staging of construction equipment and job site trailers, the offloading and temporary storage of equipment and materials, and equipment fueling. All construction laydown areas would be located within the Project site boundary. Construction laydown areas would be cleared of vegetation and soil would be compacted to support heavy equipment. Upon completion of Project construction, temporary disturbance associated with the laydown areas would be revegetated (refer to the Project Decommissioning discussion below) if not used for solar arrays or facility operations.

TEMPORARY OFFICE TRAILERS

It is anticipated that temporary office trailers would be utilized at construction laydown areas. Construction staff would use these trailers for daily operations such as plan review; meetings; environmental, health and safety trainings; and other general office activities. The trailers would be approximately 20 feet by 20 feet (400 square feet) and 10 feet tall and would rest on temporary foundation structures. Temporary sanitary facilities would be brought into the office trailer sites, routinely maintained by a licensed sanitary waste hauler, and waste would be hauled offsite as needed to an approved disposal facility.

PARKING

Parking for construction workers would be provided near the temporary office trailers and at conveniently accessed areas of the Project site.

Construction Sequence and Equipment

SCHEDULE AND WORKFORCE

Project construction is anticipated to commence in 2028 with the initial 900 to 1,500 MW phase to be completed over a period of approximately 24 to 36 months. The Project would develop up to approximately 20,600 acres in multiple phases. Construction activities for the Project generally fall into three main categories: site preparation; system installation; and testing, commissioning, and cleanup.

The Project would be constructed by several specialized construction contractors. The onsite construction workforce is expected to peak at approximately 660 individuals; however, the average daily construction workforce is expected to be approximately 470 construction, supervisory, support, and construction management personnel onsite. It is anticipated that the construction workforce would generally commute to the site each day from local communities and report to designated construction laydown areas prior to the beginning of each workday. Construction staff not drawn from the local labor pool would stay in local hotels in the nearby communities of Kettleman City or Lost Hills, or potentially in other local areas such as Coalinga, Wasco, or Lemoore.

One or more of the construction laydown areas may be used as a parking and meeting area for construction staff. It is anticipated that the construction workforce could use Utica Avenue, or other local roads as points of ingress/egress to the Project site via the I-5 and that, once onsite, would access various areas via internal Project and existing dirt access roads.

Construction would primarily occur during daylight hours, Monday through Friday; however, some work may occur on Saturday and Sunday and after hours, as required, to meet the construction schedule and/or to promote the health and safety of workers. Project construction work would conform to the Kings County Noise Ordinance (Ord. No. 574, § 1, 6-23-98) and with all applicable policies contained within the Noise Element of the Kings County General Plan.

SITE PREPARATION

Site preparation activities would include trash removal, mowing, clearing and selective grading of the site. All applicable local, state, and federal requirements and best management practices (BMP) would be incorporated into site preparation and construction activities at the Project site. Initial work on the site would involve preparing the land for installation of arrays, energy storage facilities, related infrastructure, access points, and temporary construction laydown areas. The construction contractor would be required to incorporate plans consistent with the County zoning ordinance including preparation of a Stormwater Pollution Prevention Plan and a soil erosion and sedimentation control plan which specifies BMPs for minimizing erosion and water quality degradation during construction. Prior to initial construction mobilization, preconstruction surveys would be performed, and sediment and erosion controls would be installed in accordance with the approved Stormwater Pollution Prevention Plan. Stabilized construction entrances and exits would be installed at access points to reduce tracking of sediment onto adjacent public roadways.

Site preparation would be consistent with the applicable California Air Resources Board, County regulations and SJVAPCD rules for dust control. Project construction personnel would use construction techniques that minimize soil disturbance and allow water to continue to flow and percolate naturally across the site. Such techniques include clearing and mowing of the site, with minimal overall mass grading; limited use of disk-and-roll and micro-grading techniques; and reducing grading to the greatest extent possible to areas where site topography requires smoothing for external fence lines, roads, and Project equipment. These construction techniques avoid and minimize environmental impacts, including impacts to air quality, noise, biological resources, and water quality.

Site preparation would involve the removal and proper disposal of existing vegetation and debris that would interfere with Project construction or the health and safety of onsite personnel. Conventional grading would be performed throughout the site but minimized to the maximum extent possible to reduce unnecessary soil movement that may result in dust. Earthwork scrapers, excavators, dozers, water trucks, haul vehicles, and graders may all be used to perform grading. Land-leveling equipment, such as a smooth steel drum roller, would be used to even the surface of the ground and to compact the upper layer of soil to a value recommended by a geotechnical engineer for structural support. Access roads may be additionally compacted to 90 percent or greater, as required, to support construction and emergency vehicles. Certain access roads may also require the use of aggregate or decomposed granite to meet emergency access requirements.

Trenching would be required for placement of underground electrical and communication lines, and may include the use of trenchers, backhoes, excavators, haul vehicles, compaction equipment, and water trucks. After preparation of the site, the pads for structures, equipment enclosures, and equipment vaults would be prepared per geotechnical engineer recommendations. The area for the substations would have a grounding grid installed and would be covered with aggregate surfacing for safe operation. Collection and transmission structures from Project substations to the transmission line would require drilling for foundation support, and the soils removed would be spread across the site.

Site preparation also includes the demarcation of environmentally sensitive areas identified through the environmental review processes in, and adjacent to, the Project site. Demarcation would include installing flags, stakes, or fencing to discourage workers from entering sensitive areas.

SOLAR PV SYSTEM INSTALLATION

Erection of the solar arrays would include support structures and associated electrical equipment and cabling. Steel foundation piles would be driven into the soil using vibratory techniques. The piles are typically spaced approximately 10 to 15 feet apart and installed to a revealed height of approximately 24 inches above maximin flood height (typically 4 feet above grade). Once the piles have been installed, the horizontal cross-members would be placed and secured. The arrays would consist of a motorized single-axis tracking system. For a single-axis tracking system, the trackers and their associated motors would be mounted to the horizontal crossmembers.

Concrete would be required for the footings and pads for the substations transformers and switchyard equipment. Concrete may also be required for pile foundation support in the event soils prevent some piles from being driven to appropriate depths or for inverter foundations. Final concrete specifications would be determined during detailed design engineering. Concrete would be purchased from an offsite supplier and trucked into the site. No concrete batch plants would be required onsite.

During this work, there would be multiple crews working onsite with various equipment and vehicles, including special vehicles for transporting the modules and other equipment. As the solar arrays are installed, the Project substations, high-voltage components, and O&M building would be constructed, and the electrical collection and communication systems would be installed. Within the solar fields, the electrical and communication wiring would be installed in underground trenches or placed in aboveground cable trays or cable support systems, and some of the medium voltage collection circuits and communication systems may be on overhead lines. The wiring would connect to the appropriate electrical and communication terminations and would be checked and commissioned prior to operation.

ENERGY STORAGE SYSTEM INSTALLATION

Installation of the BESS would be specific to the manufacturer and model of the BESS selected. A typical lithium-ion, battery-based BESS would include installation of pour-in-place concrete footings, pad foundations, piers, and/or driven pilings. The foundations would then be backfilled and compacted as necessary prior to installation of power conversion systems, power distribution systems, pad-mounted transformers, and battery storage enclosures. Electrical cables would be installed to connect the equipment and above-ground utilities. The circuits and control systems would be checked and tested for proper installation and functionality during commissioning prior to operation.

TESTING, COMMISSIONING, STARTUP, AND CLEANUP

After each major electrical system component is completed, including interconnection, high-voltage transmission, electrical collection system, and the solar arrays, the system would be thoroughly inspected and tested to ensure it is ready to carry energy. The system would then be energized, and additional testing would be done to ensure safe operation.

“Startup” refers to commissioning activities that would occur after mechanical completion of the solar modules and commissioning of the electrical system.

During construction, the site would be kept as clean as possible. After construction of the Project is complete, all waste, debris, and construction equipment would be removed from the site. Laydown areas or other areas disturbed during construction that are not needed for solar arrays or facility operations would be revegetated.

Construction Utility Requirements, Materials, and Equipment

ELECTRICAL SUPPLY

Power required for construction activities would be provided via portable generators or temporary local distribution. Generators used onsite for construction, backup power supply, or for emergency fire protection would abide by California Air Resources Board and SJVAPCD regulations.

WATER AND WASTEWATER

During Project construction, water would be required for common construction-related purposes, including but not limited to dust suppression, soil compaction, and grading. Dust-control water may be used for ingress and egress of onsite construction and delivery vehicle traffic. A sanitary water supply would not be required during construction because restroom facilities would be provided by portable units to be serviced by licensed providers. Construction water usage is anticipated to be approximately 1,000 acre-feet every 18 to 24 months of the construction period. During construction, the water used is anticipated to be purchased from local water purveyors or existing onsite water wells. This is anticipated to be sourced from the San Joaquin Valley – Tulare Lake Groundwater Basin (Basin 5-022.12). Water demand during construction is expected to be the same if the Project is constructed during a year with normal precipitation, a year with less-than-average precipitation, or a multiyear period of less-than-average precipitation.

Solid and Nonhazardous Waste

A small amount of solid waste would be produced from Project construction activities. This may include paper, wood, glass, plastics from packing material, waste lumber, insulation, scrap metal and concrete, empty nonhazardous containers, and vegetation waste. These wastes would be segregated, where practical, for recycling. Non-recyclable wastes would be placed in covered dumpsters and removed on a regular basis by a certified waste-handling contractor for disposal at a Class III landfill. Vegetation waste generated by site clearing and grubbing would be chipped/mulched and spread onsite or hauled offsite to an appropriate green waste facility. The closest Class III municipal landfill is the Avenal Regional Landfill, located approximately 17 miles northwest of the northwestern corner of the Project site boundary at 201 North Hydril Road, Avenal. Avenal Regional Landfill is a modern municipal solid waste disposal facility permitted by Cal Recycle and is in full compliance with state and local rules and regulations.

Hazardous Materials

The proposed BESS is anticipated to use lithium-ion batteries. If utilized, the batteries would be delivered to the Project site in U.S. Department of Transportation–certified vehicles and in compliance with all applicable requirements of the U.S Department of Transportation, California Highway Patrol, and the California Department of Motor Vehicles.

Other hazardous materials used for construction would be typical for most construction projects of this type. Materials would include small quantities of gasoline, diesel fuel, oils, lubricants, solvents,

detergents, degreasers, paints, ethylene glycol, dust palliative, herbicides, and welding materials/supplies.

A Hazardous Materials Business Plan would be provided to the Kings County Department of Public Health that would include a complete list of all materials used onsite, information regarding how the materials would be transported, and emergency response/contingency plans and safety procedures. This information would be recorded to maintain safety and prevent possible environmental contamination or worker exposure from a release or threatened release of a hazardous material. During Project construction, material Safety Data Sheets for all applicable materials present at the Project site would be made readily available to onsite personnel.

Hazardous Waste

Small quantities of hazardous waste, which would be typical for most construction projects of this type, would most likely be generated over the course of construction. These wastes may include waste paint, spent construction solvents, waste cleaners, waste oil, oily rags, waste batteries, and spent welding materials. Workers would be trained to properly identify and handle all hazardous materials. Hazardous waste would be either recycled or disposed of at a permitted and licensed treatment or disposal facility. All hazardous waste shipped offsite for recycling or disposal would be transported by a licensed and permitted hazardous waste hauler and disposed of at an approved location.

Operations and Maintenance

Schedule and Workforce

The facility would operate year-round. It is anticipated that approximately 10 to 20 permanent, full-time employees would be needed to maintain the facility. Typical O&M activities that would occur at the site during operation include, but are not limited to, liaison and remote monitoring; administration and reporting; semi-annual and annual services; remote operations of inverters and BESS; site security and management; additional communication protocol; repair and maintenance of solar facilities, the battery storage facility, the substation, electrical transmission lines, and other Project facilities; battery storage facility supplemental phasing and/or augmentation; mowing and vegetation control; and periodic panel washing. As technology continues to advance, existing systems may be upgraded or replaced during operations to enhance efficiency and output without expanding the Project development footprint.

Electrical Supply

Power for O&M activities would be provided by the Project's electrical generation, supplied by the local power provider or via portable generators. The Project would require power for the electrical enclosures, substation equipment, tracker motors (if non-self-powered single-axis trackers are used), battery storage, and plant lighting and security. Batteries in the substations buildings would supply backup DC power for the protection equipment. In the event emergency backup generators are required for Project facilities such as Project substations and the BESS, these diesel-powered generators would meet applicable best available control technology and be operated in accordance with applicable California Air Resources Board and SJVAPCD rules and regulations.

Water Use

During Project operation, the total annual water supply for the Project would be approximately 86 acre-feet per year (AFY) for solar panel washing, equipment washing, general maintenance, and other miscellaneous water uses including fire water supply and O&M building use. Water is anticipated to be sourced from groundwater and/or local water providers.

Panel washing for a Project of this size would require approximately 30 to 60 days to complete per wash cycle. PV panel washing is typically expected to occur twice per year.

The Project would also include a number of aboveground fire water protection tanks for operations, exposure cooling, and non-battery fire suppression, which would be placed at designated locations throughout the Project site, including near BESS locations. Final locations and tank volumes would be subject to review and approval by Kings County Fire Department.

The O&M building facility would include permanent restroom facilities for up to 20 O&M staff and a water source in the form of trucked water, well water (from offsite or onsite wells), or bottled water would be provided for staff. The Project would not include a public water system, as defined in the California Health and Safety Code (Health & Saf. Code) § 116275 subd. [h].

Wastewater

During Project operation, wastewater production would be associated with permanent toilet and sanitary facilities. Sanitary facilities would either consist of portable sinks and toilets that would be regularly emptied by a permitted provider, or permanent facilities with an Onsite Wastewater Treatment System (septic system), subject to oversight and approval by the Kings County Environmental Health Services, a designated Local Agency Management Program agency under the State Water Resources Control Board/Regional Water Quality Control Board Onsite Wastewater Treatment System program. Anticipated maximum peak flow is 1,500 gallons into the leach field per day during Project operation. A Waste Discharge Permit would not be required from the Regional Water Quality Control Board because the Project would not exceed 2,500 gallons per day of sewage.

No wastewater generated through Project operations would be disposed of through discharge directly to open waterbodies.

Stormwater

Stormwater facilities would be constructed in accordance with County standards. The Project would include detention basins placed throughout the Project site to control the rate and amount of stormwater runoff associated.

Solid and Nonhazardous Waste

Project operation and maintenance activities would produce a small amount of waste associated with maintenance activities, which could include broken and rusted metal, defective or malfunctioning modules, electrical materials, empty containers, and other miscellaneous solid waste, including the typical refuse generated by workers. Most of these materials would be collected and returned to the manufacturer or delivered to recyclers. Nonrecyclable waste would be placed in covered dumpsters and removed on a regular basis by a certified waste-handling contractor for disposal at a Class III landfill. As previously described, the closest Class III municipal landfill is the Avenal Landfill, located approximately 17 miles northwest of the northwestern corner of the Project site boundary. Shipping materials and other general solid wastes would be separated for recycling where possible/available.

Hazardous Materials

Limited amounts of hazardous materials would be stored or used on the site during Project operations, including diesel fuel, gasoline, and motor oil for vehicles; mineral oil to be sealed within the transformers; and lead-acid-based or lithium-ion-based batteries for emergency backup. If the quantities stored are estimated to be in excess of 1,320 gallons,[2] storage would be undertaken in compliance with the Spill Prevention, Control, and Countermeasure Rule. However, quantities are not likely to be in excess of 1,320 gallons. In addition, if the quantities stored are estimated to be 55 gallons or 500 pounds, a Hazardous Materials Business Plan would be developed prior to construction for submission to the Kings County Department of Public Health. The Hazardous Materials Business Plan would include a complete inventory of all materials used onsite, information regarding how the materials would be transported, and response/contingency plans and safety procedures. Safety Data Sheets for all applicable materials present at the site would be maintained and made readily available to onsite personnel. Appropriate spill containment and cleanup kits would be maintained during operation of the Project.

Hazardous Waste

Project operation and maintenance activities would produce a small amount of hazardous waste associated with maintenance activities, which could include, but not be limited to, defective or malfunctioning modules, electrical materials, unused paint, solvents, cleaners, waste oil, oily rags, and batteries. Workers would be trained to properly identify and handle all hazardous waste. Fuels and lubricants used in operations would be subject to the Spill Prevention, Containment, and Countermeasures Plan to be prepared for the Project. Solid waste, if generated during operations, would be subject to the Solid Waste Management Plan to be prepared for the Project.

Hazardous waste would be recycled or disposed of at a permitted and licensed treatment and/or disposal facility. All hazardous waste shipped offsite for recycling or disposal would be transported by a licensed and permitted hazardous waste hauler and disposed of at an approved location.

Security and Lighting

Fencing would be installed around the perimeter of most Project facilities, including all areas requiring controlled access for safety and security purposes as needed. Locking gates would be installed at access roads entering the site. A 3-to-5-inch gap may be included at the bottom of the fence to allow for wildlife passage under the fence. Limiting access to the site would be necessary both to ensure the safety of the public and to protect the equipment from potential theft and vandalism. Fencing would remain for the life of the Project. Final fence design would be developed in consultation with appropriate agencies.

The Project's lighting system would provide O&M personnel with illumination for both normal and emergency conditions. Lighting would be designed to provide the minimum illumination needed to achieve safety and security objectives. Additionally, lighting would be directed downward and shielded to focus illumination on the desired areas only and to minimize light trespass in accordance with applicable County requirements. Motion-sensitive cameras may also be installed throughout the solar arrays at the inverters for added security.

[2] Effective January 1, 2008, the Certified Unified Program Agencies are vested with the responsibility and authority to implement the Aboveground Petroleum Storage Act. Owners or operators of aboveground petroleum storage tanks are required to file a storage statement and implement spill prevention measures according to the Aboveground Petroleum Storage Act of 1990. Facilities with a single tank or cumulative aboveground storage capacities of 1,320 gallons or greater of petroleum are covered by this law.

The Project Sponsor would prepare a Fire Safety Plan for the Project in compliance with applicable Kings County regulations and an Emergency Action and Response Plan pursuant to Senate Bill (SB) 38.

Decommissioning

The Project has an anticipated operational life of up to 40 years, after which the Project Sponsor may choose to update site technology and recommission, or to decommission the site and remove the systems and their components, except for the PG&E-owned and operated switchyard and connection lines to their transmission system, which would not be decommissioned. All decommissioning would occur within the Project site and disturbance limits, and would involve similar, though reduced construction equipment and activities.

Because the tracking system and supporting equipment for the PV arrays would be supported by steel posts that would be pile-driven into the ground, the land would be largely unaltered from its natural state when the arrays are removed after the Project's lifetime. As discussed in more detail below, portions of the Project site would require reclamation. The Project site could then be converted to other uses in accordance with applicable land use regulations in effect at that time.

Decommissioning and restoration activities would adhere to the requirements of the appropriate governing authorities as well as all applicable local, state, and federal requirements and BMPs. The Project would include BMPs to ensure the collection and recycling of modules and batteries and to avoid the potential for modules or batteries to be disposed of as municipal waste. The Project is anticipated to need approximately 1,100 acre-feet of water during decommissioning.

It is anticipated that during Project decommissioning, Project structures, including an onsite O&M building, would be removed from the site. Aboveground equipment that would be removed includes module posts and support structures, onsite transmission poles that are not shared with third parties, and the overhead collection system within the site and inverters, transformers, electrical wiring, battery storage equipment, and equipment on the inverter pads. The substations would be removed unless a public or private utility assumes ownership of the substations, in which case the equipment may remain onsite to be used as part of the utility service to supply other applications.

Equipment would be de-energized prior to removal, salvaged (where possible), placed in appropriate shipping containers, and secured in a truck transport trailer for shipment offsite to be recycled or disposed of at an appropriately licensed disposal facility. Some materials may also be returned to the manufacturer.

Removal of the solar modules would include removing the racks on which the solar panels are attached and placing them in secure transport crates and a trailer for storage. Removal of the energy storage facility equipment would include removing the batteries, enclosures, and associated inverters and control panels. Batteries would be placed in appropriate hazardous materials containers for proper handling and removal from the site. The equipment would ultimately be transported to another facility (possibly including an equipment manufacturer) or to a recycling or disposal facility. Once the solar modules have been removed, the racks would be disassembled and the structures supporting the racks would be removed. Site infrastructure would be removed, including the fences and the concrete pads that may support the inverters, transformers, and related equipment, and concrete posts would be removed to grade level. The demolition debris and removed equipment may be cut or dismantled into pieces that can be safely lifted or carried with the equipment being used. The fencing and gates would be removed, and all materials would be

recycled to the extent feasible. Project roads would be restored to their preconstruction condition to the extent feasible, unless the landowner elected to retain the improved roads for access throughout that landowner's property. The area would be thoroughly cleaned, and all debris would be removed. A collection and recycling program would be executed to promote recycling of Project components and minimize disposal in landfills.

As mentioned above, reclamation activities would occur as part of Project decommissioning. Reclamation activities aim to bring the Project site back to similar conditions that existed prior to construction of the Project; the goal of reclamation activities is to return the land to a condition in which natural vegetation can successfully re-establish. Pursuant to Kings County Development Code Section 1112.B.2.d, the Applicant will submit a reclamation plan and financial assurance to the County to ensure the return of the land to a farmable state after completion of the Project life. While the reclamation plan details proposed reclamation activities in full, specific activities may include, but not be limited to, the following actions:

- Project drainage features (retention basins, etc.) would be reclaimed using suitable fill material.
- Roads, parking areas, and substations would be removed and reclaimed to their pre-construction conditions and topography.
- Soils would be de-compacted using a disking method, as needed.
- As needed, topsoil would be used to restore suitable conditions for vegetation growth.

For the purpose of this analysis, decommissioning is assumed to involve the use of similar equipment and personnel as those used during Project construction, and for the same duration.

10. Other Public Agencies Whose Approval is Required

The Project would require a Conditional Use Permit; therefore, Kings County is the lead agency for the Project. Other public agencies whose approval may be required for the Project include:

- Kings County Community Development Agency Building Division
- Kings County Fire Department
- California Department of Transportation
- San Joaquin Valley Air Pollution Control District
- United States Fish and Wildlife Service
- United States Army Corps of Engineers
- State Water Resources Control Board
- Regional Water Quality Control Board
- California Department of Fish and Wildlife

11. Have California Native American Tribes Traditionally and Culturally Affiliated with the Project Area Requested Consultation Pursuant to Public Resources Code Section 21080.3.1?

The County is conducting formal consultation pursuant to Assembly Bill 52 in order to determine the potential for the Project to result in impacts to tribal cultural resources.

A consultation notice packet was sent to the Santa Rosa Rancheria Tachi Yokut Tribe on August 7, 2025. To this date, no requests for consultation have been received. The tribe will be included on all further public notices for the Project.

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Environmental Factors Potentially Affected

This Project would potentially affect the environmental factors checked below, involving at least one impact that is "Potentially Significant" or "Less than Significant with Mitigation Incorporated" as indicated by the checklist on the following pages. By identifying an impact as potentially significant in this Initial Study, the Lead Agency is indicating that further analysis is warranted in an EIR; it is not an indication that there is a significant impact. The EIR will fully evaluate potential impacts in detail and reach significance determinations based on further evidence.

■ Aesthetics	■ Agriculture and Forestry Resources	■ Air Quality
■ Biological Resources	■ Cultural Resources	□ Energy
■ Geology and Soils	■ Greenhouse Gas Emissions	■ Hazards and Hazardous Materials
■ Hydrology and Water Quality	■ Land Use and Planning	□ Mineral Resources
■ Noise	□ Population and Housing	■ Public Services
□ Recreation	■ Transportation	■ Tribal Cultural Resources
■ Utilities and Service Systems	■ Wildfire	■ Mandatory Findings of Significance

Determination

Based on this initial evaluation:

□ I find that the proposed project COULD NOT have a significant effect on the environment, and a NEGATIVE DECLARATION will be prepared.

□ I find that although the proposed project could have a significant effect on the environment, there will not be a significant effect in this case because revisions to the project have been made by or agreed to by the project proponent. A MITIGATED NEGATIVE DECLARATION will be prepared.

■ I find that the proposed project MAY have a significant effect on the environment, and an ENVIRONMENTAL IMPACT REPORT is required.

□ I find that the proposed project MAY have a "potentially significant impact" or "less than significant with mitigation incorporated" impact on the environment, but at least one effect (1) has been adequately analyzed in an earlier document pursuant to applicable legal standards, and (2) has been addressed by mitigation measures based on the earlier analysis as described on attached sheets. An ENVIRONMENTAL IMPACT REPORT is required, but it must analyze only the effects that remain to be addressed.

☐ I find that although the proposed project could have a significant effect on the environment, because all potential significant effects (a) have been analyzed adequately in an earlier EIR or NEGATIVE DECLARATION pursuant to applicable standards, and (b) have been avoided or mitigated pursuant to that earlier EIR or NEGATIVE DECLARATION, including revisions or mitigation measures that are imposed upon the proposed project, nothing further is required.



Signature

June 23, 2026

Date

Noelle Tomlinson

Printed Name

Planner III

Title

Environmental Checklist

1 Aesthetics

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Except as provided in Public Resources Code Section 21099, would the project:				
a. Have a substantial adverse effect on a scenic vista?	☐	☐	☐	■
b. Substantially damage scenic resources, including but not limited to, trees, rock outcroppings, and historic buildings within a state scenic highway?	☐	☐	☐	■
c. In non-urbanized areas, substantially degrade the existing visual character or quality of public views of the site and its surroundings? (Public views are those that are experienced from a publicly accessible vantage point). If the project is in an urbanized area, would the project conflict with applicable zoning and other regulations governing scenic quality?	■	☐	☐	☐
d. Create a new source of substantial light or glare that would adversely affect daytime or nighttime views in the area?	■	☐	☐	☐

a. Would the project have a substantial adverse effect on a scenic vista?

Impacts on scenic vistas are analyzed from points or corridors that are accessible to the public and that provide a view of a scenic vista. Development within a viewer's line-of-sight of scenic areas may interfere with a public view of a scenic vista, either by physically blocking or screening the vista from view or by impeding or blocking access to a formerly available viewing position. The Kings County General Plan Open Space Element identifies the Kings River and other natural watercourses, and foothill and mountain terrain as scenic resources (Kings County 2010b). In addition, the County identifies the following as scenic vistas: the Coast Ranges with formations of the Chalk Buttes-Reef Ridge portion of the Kreyenhagen Hills, the Pyramid Hills, and Sunflower Valley. Other scenic resources include the various ridgelines located west of the County in adjacent Fresno County, which are visible along State Route 41 from the northern county line to Kettleman City. Kings River is located approximately 17.5 miles north of the Project site, and the Project site is not proximate to natural waterways. The nearest foothills to the Project site are located approximately 3.2 miles west. The Coast Ranges are located approximately 25 miles southwest of the Project site. The Kreyenhagen Hills are located approximately 27 miles northwest of the Project site. Pyramid Hills are located approximately 12 miles southwest of the Project site. Sunflower Valley is located

approximately 12 miles southwest of the Project site. Given the distance between these scenic resources and the Project site, the Project would not have a substantial adverse effect on these scenic vistas. No impact would occur.

NO IMPACT

b. Would the project substantially damage scenic resources, including but not limited to, trees, rock outcroppings, and historic buildings within a state scenic highway?

Based on the California Department of Transportation's (Caltrans) California State Scenic Highway Map, the closest officially designated state scenic highway to the Project site is State Route 1 located approximately 68 miles west of the Project site (Caltrans 2018). Given the distance between State Route 1 and the Project site, the Project would not substantially damage scenic resources within a state scenic highway. No impact would occur.

NO IMPACT

c. Would the project, in non-urbanized areas, substantially degrade the existing visual character or quality of public views of the site and its surroundings? (Public views are those that are experienced from a publicly accessible vantage point). If the project is in an urbanized area, would the project conflict with applicable zoning and other regulations governing scenic quality?

The Project site is located in a non-urbanized area. The visual character of the Project site and surrounding area is generally open and rural in nature, with public views available to motorists driving I-5. This is characterized by uses such as grazing, open space, and cultivated agriculture.

The Project includes the construction, operation and maintenance, and decommissioning of an up to 2,800 MW solar PV electricity generating facility with up to 11.2 GWh of battery energy storage capacity, and associated infrastructure on a site that is primarily agriculture. The Project would alter the existing rural and agricultural character of the site as a result of converting open agricultural land to a solar power-generation and energy storage facility. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

d. Would the project create a new source of substantial light or glare that would adversely affect daytime or nighttime views in the area?

The Project site is located in a rural area in Kings County where existing levels of light and glare are limited to minimal lighting associated with agricultural equipment and lights from vehicles travelling on I-5. There are no existing sources of light or glare on the Project site.

Project operations would require minimal lighting for safety and security functions and for the O&M building. As required by Article 4, Section 418, E of the Kings County Development Code, lighting would be directed downward, hooded, and shielded to focus illumination on the desired areas only and prevent glare.

The BESS facility components are not reflective and would not have the potential to result in substantial glare. The solar PV module faces would be minimally reflective, dark in color, and highly absorptive of light. Therefore, they would not result in substantial glare. However, given there are no existing sources of light or glare on the Project site, this issue will be studied in detail in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

2 Agriculture and Forestry Resources

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Convert Prime Farmland, Unique Farmland, or Farmland of Statewide Importance (Farmland), as shown on maps prepared pursuant to the Farmland Mapping and Monitoring Program of the California Resources Agency, to non-agricultural use?	■	□	□	□
b. Conflict with existing zoning for agricultural use or a Williamson Act contract?	■	□	□	□
c. Conflict with existing zoning for, or cause rezoning of, forest land (as defined in Public Resources Code Section 12220(g)); timberland (as defined by Public Resources Code Section 4526); or timberland zoned Timberland Production (as defined by Government Code Section 51104(g))?	□	□	□	■
d. Result in the loss of forest land or conversion of forest land to non-forest use?	□	□	□	■
e. Involve other changes in the existing environment which, due to their location or nature, could result in conversion of Farmland to non-agricultural use or conversion of forest land to non-forest use?	■	□	□	□

a. Would the project convert Prime Farmland, Unique Farmland, or Farmland of Statewide Importance (Farmland), as shown on maps prepared pursuant to the Farmland Mapping and Monitoring Program of the California Resources Agency, to non-agricultural use?

The California Department of Conservation (DOC) identifies the Project site primarily as grazing land and urban and built-up land. However, a portion of the Project site adjacent to Utica Avenue contains approximately 230 acres of land designated as Prime Farmland and approximately 191 acres of land designated as Farmland of Statewide Importance (DOC 2025a). Portions of the Project site also are considered "medium-high priority" agricultural land under the County's Resource Conservation Element. Accordingly, construction of the Project would convert Farmland to non-agricultural use. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project conflict with existing zoning for agricultural use or a Williamson Act contract?

Several Project site parcels are subject to existing Williamson Act and Farmland Security Zone contracts (DOC 2025b). Therefore, the Project may conflict with existing Williamson Act contracts. In addition, the Project site is zoned AG-40 (Agricultural, 40-acre minimum parcel size). Solar PV energy facilities may be allowed in this zone by discretionary approval of a Conditional Use Permit and must comply with Article 11, Section 1112 of the Development Code. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project conflict with existing zoning for, or cause rezoning of, forest land (as defined in Public Resources Code Section 12220(g)); timberland (as defined by Public Resources Code Section 4526); or timberland zoned Timberland Production (as defined by Government Code Section 51104(g))?

The Project site and immediate vicinity does not contain any land defined as forest land (as defined in Public Resources Code Section 12220(g)), timberland (as defined by Public Resources Code Section 4526), or timberland zoned Timberland Production (as defined by Government Code Section 51104(g)). Therefore, the Project would not conflict with existing zoning for, or cause rezoning of, forest land, timberland, or timberland zoned Timberland Production. No impact would occur as a result of the Project.

NO IMPACT

d. Would the project result in the loss of forest land or conversion of forest land to non-forest use?

The Project site and immediate vicinity does not contain any land defined as forest land (as defined in Public Resources Code Section 12220(g)), timberland (as defined by Public Resources Code Section 4526), or timberland zoned Timberland Production (as defined by Government Code Section 51104(g)). Therefore, the Project would not result in the loss or conversion of forest land to non-forest use. No impact to forest land would occur.

NO IMPACT

e. Would the project involve other changes in the existing environment which, due to their location or nature, could result in conversion of Farmland to non-agricultural use or conversion of forest land to non-forest use?

See thresholds 2a and 2c above. Construction of the Project would convert Farmland to non-agricultural use. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

3 Air Quality

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Conflict with or obstruct implementation of the applicable air quality plan?	■	□	□	□
b. Result in a cumulatively considerable net increase of any criteria pollutant for which the project region is non-attainment under an applicable federal or state ambient air quality standard?	■	□	□	□
c. Expose sensitive receptors to substantial pollutant concentrations?	■	□	□	□
d. Result in other emissions (such as those leading to odors) adversely affecting a substantial number of people?	□	□	□	■

a. Would the project conflict with or obstruct implementation of the applicable air quality plan?

The Project site is located in the San Joaquin Valley Air Basin (Air Basin), which is under the jurisdiction of the SJVAPCD and California Air Resources Board. The Air Basin is in nonattainment for the federal and state standards for ozone and particulate matter ($PM_{2.5}$) and state standards for PM_{10}. The Air Basin is unclassified or classified as attainment for all other pollutant standards (SJVAPCD 2025). Implementation of the Project would generate both short-term construction-related emissions and long-term operational emissions with the potential to violate SJVAPCD standards or result in a cumulatively considerable increase in criteria pollutants. This impact would be potentially significant. An air quality study will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project result in a cumulatively considerable net increase of any criteria pollutant for which the project region is non-attainment under an applicable federal or state ambient air quality standard?

See threshold 2a above. Implementation of the Project would generate both short-term construction-related emissions and long-term operational emissions with the potential to violate SJVAPCD standards or result in a cumulatively considerable increase in criteria pollutants. This impact would be potentially significant. An air quality study will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project expose sensitive receptors to substantial pollutant concentrations?

The nearest sensitive receptors to the Project site include a residence north of Utica Avenue located approximately 2.5 miles to the east of the easternmost Project site boundary and three potential residences located approximately 2.5 miles west of the Project site boundary. Although there are no sensitive receptors in the immediate vicinity of the Project site, *Coccidioides immitis* has been found in the soils in the region. The *Coccidioides* fungus can cause valley fever if spores become airborne and are inhaled. Although not anticipated to be a significant impact, Project construction and operational activities have the potential to expose construction and operations and maintenance workers to valley fever through the generation of airborne dust. Therefore, an air quality study will be prepared for the Project and will evaluate in detail the potential for exposure of receptors to *Coccidioides* fungus, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

d. Would the project result in other emissions (such as those leading to odors) adversely affecting a substantial number of people?

Substantial objectionable odors are normally associated with agriculture, wastewater treatment, industrial uses, or landfills. The Project involves the construction, operation and maintenance, and decommissioning of a solar power-generation and energy storage facility and associated infrastructure that do not produce objectionable odors. No impact would occur from implementation of the Project.

NO IMPACT

4 Biological Resources

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Have a substantial adverse effect, either directly or through habitat modifications, on any species identified as a candidate, sensitive, or special status species in local or regional plans, policies, or regulations, or by the California Department of Fish and Wildlife or U.S. Fish and Wildlife Service?	■	□	□	□
b. Have a substantial adverse effect on any riparian habitat or other sensitive natural community identified in local or regional plans, policies, or regulations, or by the California Department of Fish and Wildlife or U.S. Fish and Wildlife Service?	■	□	□	□
c. Have a substantial adverse effect on state or federally protected wetlands (including, but not limited to, marsh, vernal pool, coastal, etc.) through direct removal, filling, hydrological interruption, or other means?	■	□	□	□
d. Interfere substantially with the movement of any native resident or migratory fish or wildlife species or with established native resident or migratory wildlife corridors, or impede the use of native wildlife nursery sites?	■	□	□	□
e. Conflict with any local policies or ordinances protecting biological resources, such as a tree preservation policy or ordinance?	■	□	□	□
f. Conflict with the provisions of an adopted Habitat Conservation Plan, Natural Community Conservation Plan, or other approved local, regional, or state habitat conservation plan?	□	□	□	■

a. Would the project have a substantial adverse effect, either directly or through habitat modifications, on any species identified as a candidate, sensitive, or special status species in local or regional plans, policies, or regulations, or by the California Department of Fish and Wildlife or U.S. Fish and Wildlife Service?

The Project site is located on undeveloped land and, although previously-disturbed due to intensive historical agricultural use, has the potential to support special status species and/or habitats. The Project site is in the vicinity of the Pacific Flyway, a major avian migration route. In addition, wetland habitat is present on the Project site. Based on a review of the USFWS Critical Habitat for Threatened and Endangered Species GIS Database (USFWS 2024), no federally designated critical habitat occurs within the Project site or vicinity.

Construction, operation and maintenance, and decommissioning of the Project could adversely affect special status species and/or habitats, protected wetlands, movement of wildlife, and conflict with local policies protecting biological resources. These impacts are potentially significant. A Biological Resources Assessment will be prepared for the Project and further study in an EIR is warranted.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project have a substantial adverse effect on any riparian habitat or other sensitive natural community identified in local or regional plans, policies, or regulations, or by the California Department of Fish and Wildlife or U.S. Fish and Wildlife Service?

See threshold 4a above.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project have a substantial adverse effect on state or federally protected wetlands (including, but not limited to, marsh, vernal pool, coastal, etc.) through direct removal, filling, hydrological interruption, or other means?

See threshold 4a above.

POTENTIALLY SIGNIFICANT IMPACT

d. Would the project interfere substantially with the movement of any native resident or migratory fish or wildlife species or with established native resident or migratory wildlife corridors, or impede the use of native wildlife nursery sites?

See threshold 4a above.

POTENTIALLY SIGNIFICANT IMPACT

e. Would the project conflict with any local policies or ordinances protecting biological resources, such as a tree preservation policy or ordinance?

See threshold 4a above.

POTENTIALLY SIGNIFICANT IMPACT

f. Would the project conflict with the provisions of an adopted Habitat Conservation Plan, Natural Community Conservation Plan, or other approved local, regional, or state habitat conservation plan?

The Project site is not located within the boundaries of any adopted Habitat Conservation Plan or Natural Community Conservation Plan (California Department of Fish and Wildlife 2023). Therefore, the Project would not conflict with the provisions of an adopted Habitat Conservation Plan or Natural Community Conservation Plan and no impact would occur.

NO IMPACT

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5 Cultural Resources

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Cause a substantial adverse change in the significance of a historical resource pursuant to §15064.5?	■	□	□	□
b. Cause a substantial adverse change in the significance of an archaeological resource pursuant to §15064.5?	■	□	□	□
c. Disturb any human remains, including those interred outside of formal cemeteries?	■	□	□	□

a. Would the project cause a substantial adverse change in the significance of a historical resource pursuant to §15064.5?

Two previously-identified cultural resources are mapped within the Project site, and the Project site is located partially within the mapped boundary of the historic Tulare Lake, an area that is known to have supported indigenous communities before the water was rerouted and drained for agricultural use. There is potential that grading and earth moving activities during proposed construction and decommissioning have the potential to disturb known and previously undiscovered cultural or archaeological resources, which could cause a substantial adverse change in the significance of a historical or archaeological resource. This impact would be potentially significant. A Cultural Resources Assessment will be prepared for the Project and further study in an EIR is warranted.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project cause a substantial adverse change in the significance of an archaeological resource pursuant to §15064.5?

Two previously-identified cultural resources are mapped within the Project site, and the Project site is located partially within the mapped boundary of the historic Tulare Lake, an area that is known to have supported indigenous communities before the water was rerouted and drained for agricultural use. There is potential that grading and earth moving activities during proposed construction and decommissioning have the potential to disturb known and previously undiscovered archaeological resources, which could cause a substantial adverse change in the significance of a historical or archaeological resource. This impact would be potentially significant. A Cultural Resources Assessment will be prepared for the Project and further study in an EIR is warranted.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project disturb any human remains, including those interred outside of formal cemeteries?

While no human remains are known to be present within the Project site, the Project site is located partially within the mapped boundary of the historic Tulare Lake, an area that is known to have supported indigenous communities before the water was rerouted and drained for agricultural use. There is potential that grading and earth moving activities during proposed construction and decommissioning have the potential to disturb known and previously undiscovered human remains, which could cause a substantial adverse change in the significance of a historical or archaeological resource. In the event human remains are inadvertently encountered during ground disturbing activities, they would be treated consistent with state and local regulations including California Health and Safety Code Section 7050.5, California Public Resources Code Section 5097.98, and California Code of Regulations Section 15064.5(e). In accordance with these regulations, if human remains are found, the County Coroner must be immediately notified of the discovery. No further disturbance can occur until the County Coroner has made a determination of origin and disposition pursuant to California Public Resources Code Section 5097.98. If the County Coroner determines the remains are, or believed to be Native American in origin, he or she is required to notify the Native American Heritage Commission. The Native American Heritage Commission will then notify those persons believed to be the most likely descendant. The most likely descendant has 48 hours from being granted site access to make recommendations for the disposition of the remains. If the most likely descendant does not make recommendations within 48 hours, the landowner must reinter the remains in an area of the property secure from subsequent disturbance.

Due to the anthropogenic history of the region encompassing the Project site, this impact would be potentially significant. A Cultural Resources Assessment will be prepared for the Project and further study in an EIR is warranted.

POTENTIALLY SIGNIFICANT IMPACT

6 Energy

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Result in a potentially significant environmental impact due to wasteful, inefficient, or unnecessary consumption of energy resources, during project construction or operation?	☐	☐	■	☐
b. Conflict with or obstruct a state or local plan for renewable energy or energy efficiency?	☐	☐	☐	■

a. Would the project result in a potentially significant environmental impact due to wasteful, inefficient, or unnecessary consumption of energy resources, during project construction or operation?

Construction and decommissioning of the Project is expected to require the use of non-renewable resources. Temporary electric power for as-necessary lighting and electronic equipment (such as computers inside temporary construction trailers and HVAC) would be provided by portable generators or temporary local distribution. The amount of electricity used during construction and decommissioning would be minimal; typical demand would stem from the use of electrically powered hand tools and several construction trailers by managerial staff during the hours of construction activities. The electricity used for construction and decommissioning activities would be temporary and minimal; therefore, potential impacts would be less than significant.

Natural gas is not anticipated to be required during construction or decommissioning of the Project. Any minor amounts of natural gas that may be consumed as a result of Project construction or decommissioning would be temporary and negligible, and would not have an adverse effect; therefore, potential impacts would be less than significant.

Construction and decommissioning of the Project would also require gasoline and diesel to power off-road construction equipment and on-road vehicles. Fuel consumption associated with the transportation of construction materials and construction worker commutes would be the primary energy resource expended over the course of construction. It is assumed that construction workers would travel to and from the Project site in gasoline-powered vehicles. Heavy duty construction equipment associated with construction activities and onsite trucks such as water trucks and trucks involved in relocating dirt around the Project site would rely on diesel fuel. In addition, construction contractors would be required to comply with the provisions of California Code of Regulations Title 13 Sections 2449 and 2485, which prohibit off-road diesel vehicles and diesel-fueled commercial motor vehicles, respectively, from idling for more than five minutes and would minimize unnecessary fuel consumption. Construction equipment would be subject to the United States Environmental Protection Agency Construction Equipment Fuel Efficiency Standard, and trucks would be subject to the California Air Resources Board Advanced Clean Trucks regulation, both of

which would also minimize inefficient, wasteful, or unnecessary fuel consumption. These regulations would result in the efficient use of energy necessary to construct the Project. Therefore, Project construction and decommissioning would not result in in wasteful, inefficient, or unnecessary consumption of energy resources, and impacts during construction and decommissioning would be less than significant.

The Project includes the development of a solar facility and associated infrastructure with the capacity to generate up to 2,800 MW of renewable electric energy with up to 11.2 GWh of battery energy storage capacity for delivery to the electrical grid, which would be a beneficial effect. Fuel during the operational phase would be required for use of emergency backup generators, employee vehicle trips, and vendor deliveries. However, the associated fuel consumption would be minimal and would not be considered an inefficient use of fuel, given that it would be associated with maintaining a renewable energy facility. Thus, although irreversible commitments of small quantities of nonrenewable resources would occur during operation of the Project, the amount of diesel and gasoline fuel consumed during Project operation would be relatively minimal and would not be considered an inefficient use.

The Project would generate approximately 286,000 gigawatt hours of carbon-free electricity annually over a 40-year lifespan. This production would more than offset the minimal amount of energy consumed annually to operate the Project. In order to meet the Assembly Bill 32 GHG emissions reduction mandate, the Climate Change Scoping Plan relies on achievement of the 33 percent State Renewable Portfolio Standard (RPS) by 2020 and 50 percent by 2030. The Project's electricity generation, and other similar projects, are essential to achieving the RPS goal and would assist State investor-owned utilities in meeting their obligations under RPS guidelines by providing a renewable energy alternative to the utilities' existing power mix. The Project would not result in a wasteful use of energy and potential impacts would be considered beneficial; therefore, no further discussion in an EIR is warranted.

LESS-THAN-SIGNIFICANT IMPACT

b. Would the project conflict with or obstruct a state or local plan for renewable energy or energy efficiency?

State and local plans related to renewable energy and energy efficiency include the California Air Resources Board's 2022 Scoping Plan Update and SB 100. Although these plans are primarily associated with reductions in greenhouse gas (GHG) emissions, the strategies and policies contained within these plans include those which relate to increasing renewable energy production and energy efficiency. The 2022 Scoping Plan's strategies that apply to the Project include the following:

- Reducing fossil fuel use, energy demand and vehicle miles traveled.

SB 100 accelerated the state's Renewables Portfolio Standard Program by increasing California's procurement of electricity from renewable sources to 33 percent of total retail sales by 2020, 60 percent by 2030, and 100 percent by 2045. The Project would be consistent with 2022 Scoping Plan strategies and assist the state to meet SB 100 targets by implementing a solar power generation and energy storage facility and thereby assisting in the increased supply of renewable solar energy within the state. The Project would also be consistent with SB 100. The Project would generate 2,800 MW of renewable energy and store up to 11.2 GWh of electricity to be used during peak activity. Overall, the Project would increase the efficiency of the existing transmission network. Increasing the efficiency of the existing transmission network would improve California's ability to supply renewable energy to end-use customers to achieve statewide renewable energy goals. The

additional renewable energy generation and energy storage provided by the Project would directly support energy goals under SB 100.

At the local level, Objective G1.2 contained in the Kings County General Plan Resource Conservation Element addresses renewable energy or energy efficiency. Objective G1.2 is listed below:

- **Objective G1.2:** Promote the development of sustainable and renewable alternative energy sources, including wind, solar, hydroelectric and biomass energy.

The Project would advance the implementation of this objective by developing a solar power generation and energy storage facility. Overall, the Project would not conflict with or obstruct a state or local plan for renewable energy or energy efficiency. No impact would occur.

NO IMPACT

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7 Geology and Soils

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:				
i. Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?	☐	☐	■	☐
ii. Strong seismic ground shaking?	☐	☐	■	☐
iii. Seismic-related ground failure, including liquefaction?	■	☐	☐	☐
iv. Landslides?	☐	☐	☐	■
b. Result in substantial soil erosion or the loss of topsoil?	■	☐	☐	☐
c. Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?	■	☐	☐	☐
d. Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property?	■	☐	☐	☐
e. Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater?	■	☐	☐	☐
f. Directly or indirectly destroy a unique paleontological resource or site or unique geologic feature?	■	☐	☐	☐

This section is based on the Preliminary Geotechnical Engineering Report prepared by Barr Engineering Co. for the Project in October 2024 (Appendix C).

a.1. Would the project directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault?

Alquist-Priolo Earthquake Fault Zones are regulatory zones established throughout California by the California Geological Survey. These zones identify areas where potential surface rupture along an active fault could prove hazardous and identify where special studies are required to characterize the fault rupture hazard potential to habitable structures. The nearest active faults to the Project site are the Great Valley thrust fault system, located approximately 8.8 miles northwest of the Project site, the Poso Creek fault located approximately 15 miles east of the Project site, and the Creeping Section of the San Andres Fault, located approximately 25 miles west of the Project site (DOC 2025c). The potential for surface fault rupture at the Project site is considered low. In addition, prior to the issuance of grading permits, the Project Sponsor would be required to design Project infrastructure to withstand probable seismically induced ground shaking in accordance with all applicable ordinances of the Kings County Building Code (Chapter 5) and the current California Building Code. The California Building Code has been adopted as the Kings County Building Code, which is implemented and enforced by the Kings County Building Official and Building Inspectors through building permit reviews, approvals, inspections, and final sign offs. Therefore, the Project would not directly or indirectly cause substantial adverse effects, including the risk of loss, injury, or death, involving rupture of a known earthquake fault and this impact would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

a.2. Would the project directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving strong seismic ground shaking?

Consistent with its location in a seismically active region, the site may be subject to strong ground shaking resulting from a major earthquake on one or more faults in the area within the lifetime of the Project. Seismic ground shaking from major earthquakes in the region is not anticipated to be greater than at any other sites in the region. The potential for strong ground shaking is an existing seismic hazard that affects the site, and the Project would not exacerbate this condition. Also, the Project would not involve construction of structures whose height, mass, or materials would pose a hazard in the event of an earthquake. In addition, the Project would be designed and constructed in compliance with the seismic standards of the Kings County Building Code (Chapter 5), which adopts the California Building Code. Grading, excavation, construction and decommissioning are required to comply with the Kings County Building Code, as it relates to site preparation and construction; alteration; moving; demolition; repair; use and occupancy of buildings; structures and building service equipment within the County. The code is implemented and enforced by the Kings County Building Official and Building Inspectors through building permit reviews, approvals, inspections, and final sign offs. Impacts related to seismic ground shaking would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

a.3. Would the project directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving seismic-related ground failure, including liquefaction?

Liquefaction is a process in which soil is temporarily transformed to fluid form during intense and prolonged ground shaking or because of a sudden shock or strain. Liquefaction occurs typically in areas where the groundwater is less than 30 feet from the surface and where the soils are composed of poorly consolidated fine to medium sand. Based on the Preliminary Geotechnical Engineering Report (Appendix C), liquefaction could potentially be a risk onsite. Impacts related to liquefaction may be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

a.4. Would the project directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving landslides?

The Project site is located on relatively flat terrain in the San Joaquin Valley. Hillside and other geographic features associated with landslide hazards are not present at the Project site (Barr Engineering 2024; Appendix C). Therefore, the Project would not expose people or structures to risk of loss, injury, or death involving landslides.

NO IMPACT

b. Would the project result in substantial soil erosion or the loss of topsoil?

Construction and decommissioning of the Project would involve earth-disturbing activities that could expose soils to the effects of wind or water erosion. Although the Project site consists of relatively flat topography and would not involve substantive cut and fill operations, earthmoving and construction activities could loosen soil, and the removal of existing minimal vegetation could contribute to soil loss and erosion. In addition, implementation of the Project would result in a slight increase in impervious surface area associated with the O&M building and BESS foundations which has the potential to result in erosion. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?

See threshold 7a.3 above. Based on the Preliminary Geotechnical Engineering Report (Appendix C), liquefaction could potentially be a risk onsite. Impacts related to liquefaction may be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

d. Would the project be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property?

The Project site is underlain by soils with high percentage of clay composition, indicating a potential for expansive soil (Barr Engineering 2024 Appendix C; United States Department of Agriculture 2025). This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

e. Would the project have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater?

During Project construction, wastewater production would be limited to temporary toilet and sanitary facilities, which would be serviced by a third-party contractor; no wastewater would be discharged within or to the Project site. During Project operation, wastewater production would be associated with permanent toilet and sanitary facilities. The sanitary facilities would either consist of portable sinks and toilets that would be regularly emptied by a permitted provider, or permanent facilities with an Onsite Wastewater Treatment System (OWTS) (i.e., septic system), subject to oversight and approval by the Kings County Division of Environmental Health Services. Shallow groundwater is present throughout the Project site (Appendix C). The Project would be required to adhere to the Kings County septic system permit requirements, specifically the requirements of the Kings County Code of Ordinances Article VI Plumbing Code and the Kings County Local Area Management Program, which set out procedures for the design, installation, and operation of on-site wastewater treatment systems. As part of the requirements, applicants proposing to install septic systems must conduct test holes to identify subsurface conditions and conduct percolation tests to adequately demonstrate the range of percolation rates within the primary disposal area.

On-site septic systems must then be designed for site specific soil and percolation conditions. The septic system design would comply with the State Water Resources Control Board's OWTS regulations (Title 27 CCR) and Kings County OWTS permit requirements. If the site conditions are unfavorable to support a conventional leach field system, the applicant would work with the County to evaluate a viable alternative, such as portable sinks and toilets. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

f. Would the project directly or indirectly destroy a unique paleontological resource or site or unique geologic feature?

The potential to disturb paleontological resources or sites or unique geologic features would occur during earthmoving activities, such as grading, using an auger, trenching, excavating, etc., at depth. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

8 Greenhouse Gas Emissions

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Generate greenhouse gas emissions, either directly or indirectly, that may have a significant impact on the environment?	☐	☐	■	☐
b. Conflict with an applicable plan, policy, or regulation adopted for the purpose of reducing the emissions of greenhouse gases?	☐	☐	■	☐

a. Would the project generate GHG emissions, either directly or indirectly, that may have a significant impact on the environment?

Project-related activities would directly contribute to the generation of GHG emissions during construction, operation, and decommissioning and indirectly contribute to the reduction of GHG emissions during operation by providing low-GHG electricity to California customers and displacing higher GHG emitting resources. The Project consists of a solar power generation and energy storage facility that would meet the requirements of California Renewable Portfolio Standard Program and the California Global Warming Solutions Act (AB 32 and SB 32) for the reduction of GHG emissions through production of renewable energy and SB 100, as discussed in Section 6, *Energy*. Over the operational life of the Project, the Project is expected to provide a net benefit with respect to the reduction of GHG emissions because potentially displaced emissions would assist in the attainment of the State's goal to reduce GHG emissions. Given the nature of the Project being a solar power generation and energy storage facility that would meet the requirements of AB 32 and SB 32 for the reduction of GHG emissions through production of renewable energy and SB 100, the Project's impacts on GHG emissions, and consistency with applicable plans and policies for the purpose of reducing GHG emissions is anticipated to be less than significant; however the Project's GHG emissions will be quantified and analyzed in detail in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

b. Would the project conflict with an applicable plan, policy, or regulation adopted for the purpose of reducing the emissions of greenhouse gases?

See threshold 8a above. The Project consists of a solar power generation and energy storage facility that would meet the requirements of California Renewable Portfolio Standard Program and the California Global Warming Solutions Act (AB 32 and SB 32) for the reduction of GHG emissions through production of renewable energy and SB 100, as discussed in Section 6, Energy. Over the operational life of the Project, the Project is expected to provide a net benefit with respect to the reduction of GHG emissions because potentially displaced emissions would assist in the attainment of the State's goal to reduce GHG emissions. This impact would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

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9 Hazards and Hazardous Materials

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Create a significant hazard to the public or the environment through the routine transport, use, or disposal of hazardous materials?	■	□	□	□
b. Create a significant hazard to the public or the environment through reasonably foreseeable upset and accident conditions involving the release of hazardous materials into the environment?	■	□	□	□
c. Emit hazardous emissions or handle hazardous or acutely hazardous materials, substances, or waste within 0.25 mile of an existing or proposed school?	□	□	□	■
d. Be located on a site that is included on a list of hazardous material sites compiled pursuant to Government Code Section 65962.5 and, as a result, would it create a significant hazard to the public or the environment?	□	□	□	■
e. For a project located in an airport land use plan or, where such a plan has not been adopted, within two miles of a public airport or public use airport, would the project result in a safety hazard or excessive noise for people residing or working in the project area?	□	□	□	■
f. Impair implementation of or physically interfere with an adopted emergency response plan or emergency evacuation plan?	■	□	□	□
g. Expose people or structures, either directly or indirectly, to a significant risk of loss, injury, or death involving wildland fires?	□	□	■	□

a. Would the project create a significant hazard to the public or the environment through the routine transport, use, or disposal of hazardous materials?

The Project would involve the transport, use, and/or disposal of limited hazardous materials and wastes (e.g., batteries, hydraulic fluid, diesel fuel, gasoline, grease, lubricants, paints and thinners, solvents and cleaning solutions, glues, and adhesives) during construction, operation and maintenance, and decommissioning that could result in potential adverse health and environmental impacts if these materials were transported, used, or disposed of improperly. Potential direct and indirect impacts of such releases could degrade soil and water quality or expose humans and the environment to the harmful effects of hazardous materials.

Excavation and grading activities associated with Project construction and decommissioning could result in harmful exposures to subsurface materials, such as residual pesticides, Coccidioides (Valley Fever) fungus spores, or other unanticipated hazardous materials.

Under normal operations, energy storage facilities do not store or generate hazardous materials in quantities that would pose a risk to offsite receptors, and no reportable quantities of acutely or extremely hazardous materials will be transported, stored, or used at the site. Nevertheless, because battery facilities do store electrochemical energy, a battery thermal runaway can occur if a cell, or area within a cell, achieves elevated temperatures due to thermal or mechanical failure. Specialized emergency response protocols are necessary for effectively addressing such emergencies. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project create a significant hazard to the public or the environment through reasonably foreseeable upset and accident conditions involving the release of hazardous materials into the environment?

See threshold 9a above. Under normal operations, energy storage facilities do not store or generate hazardous materials in quantities that would pose a risk to offsite receptors, and no reportable quantities of acutely or extremely hazardous materials will be transported, stored, or used at the site. Nevertheless, because battery facilities do store electrochemical energy, a battery thermal runaway can occur if a cell, or area within a cell, achieves elevated temperatures due to thermal or mechanical failure. Specialized emergency response protocols are necessary for effectively addressing such emergencies. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project emit hazardous emissions or handle hazardous or acutely hazardous materials, substances, or waste within 0.25 mile of an existing or proposed school?

The nearest populated area to the Project site is the community of Alpaugh, located approximately 10 miles to the east. The Project site is not located within 0.25-mile of an existing or proposed school. The nearest school, Kettleman City Elementary School, to the Project site is located approximately 9.5-miles to the northwest. No impact would occur as a result of the Project.

NO IMPACT

d. Would the project be located on a site that is included on a list of hazardous material sites compiled pursuant to Government Code Section 65962.5 and, as a result, would it create a significant hazard to the public or the environment?

The Project is not located on a site included on a list of hazardous material sites compiled pursuant to Government Code Section 65962.5 and would not create a significant hazard to the public (California Environmental Protection Agency 2025; State Water Resources Control Board 2025). No impact would occur as a result of the Project.

NO IMPACT

e. For a project located within an airport land use plan or, where such a plan has not been adopted, within two miles of a public airport or public use airport, would the project result in a safety hazard or excessive noise for people residing or working in the project area?

The nearest airport to the Project site is the Delano Municipal Airport, located approximately 26 miles southeast of the Project site. The Project is not located in an airport land use plan and would not result in a safety hazard for people residing or working in the Project area. There would be no impact as a result of the Project.

NO IMPACT

f. Would the project impair implementation of or physically interfere with an adopted emergency response plan or emergency evacuation plan?

The County's General Plan Health and Safety Element identifies I-5 as a primary evacuation route within Kings County and Utica Avenue is designated as a secondary evacuation route (Kings County 2010c). Given the proximity of the Project to I-5, and due to temporary increased Project-related traffic on area roadways during Project construction and decommissioning, further study in an EIR is warranted regarding the potential for the Project to impair emergency evacuation routes included within the County's General Plan Health and Safety Element as this impact would be potentially significant.

The Project Sponsor is required to develop an Emergency Action and Emergency Response Plan pursuant to SB 38 in advance of construction to train local emergency response personnel during development and operation of the facility. The plan would be completed in accordance with existing state regulations (Health and Safety Code Section 25504(b); 19 Cal. Code Regs. Section 2731; 22 Cal. Code Regs. Section 66262.34(a)(4)). The contents of the Emergency Action and Emergency Response Plan would comply with existing state regulations, be developed in consultation with the Kings County Fire Department and Project Sponsor and would include defined roles and responsibilities and training for local first responders. The Emergency Action and Emergency Response Plan would be discussed further within the EIR. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

g. Would the project expose people or structures, either directly or indirectly, to a significant risk of loss, injury, or death involving wildland fires?

The Project site is not populated and is located in a sparsely vegetated, flat agricultural area with no nearby forested regions. According to the California Department of Forestry and Fire Protection (CAL FIRE 2025), the site is not within a High or Very High Fire Hazard Severity Zone. It lies approximately 16 miles east of the nearest Very High zone and 3 miles east of the nearest High zone (CAL FIRE 2025).

The Kings County Health and Safety Element (2010c) identifies most of the Project site as being within 1.5 miles (2,400 meters) of a Moderate Threat area for potential fire, with a portion also within 2,400 meters of a High Threat area (Figure HS–9).

The Project involves the construction, operation and maintenance, and decommissioning of a solar power-generation and energy storage facility and associated infrastructure. During Project construction and decommissioning, the primary fire hazards would be heat or sparks from vehicles and construction equipment. These hazards could potentially ignite dry vegetation at the Project site, especially during the warmer, dry months between June and October. Additionally, construction activities, such as welding and grinding, could generate sparks that would increase the likelihood of ignition. Thus, depending on the time of year and location of construction activities at the Project site, there could be a temporary exacerbated wildfire risk in the area due to these factors. However, site preparation would involve the removal of vegetation and Cal/OSHA Title 8 Regulations include Subchapter 4, Construction Safety Orders, Article 36, Fire Protection and Prevention, which contains standard practices to minimize the potential for fires during construction. The Project would adhere to all applicable fire protection and prevention regulations to limit the potential for fires during construction and decommissioning.

During operations, fire risks may exist due to heat or sparks from vehicles and equipment. To reduce this risk, the Project Sponsor maintains strict vegetation management at the Project site in accordance with all applicable fire codes and regulations, that will be outlined in a conditioned Fire Safety Plan that will be required for the Project, thus reducing fire risks. The Project also includes an energy storage system. While batteries are generally resistant to combustion, they can catch fire; proper vegetation management would minimize wildfire risk. The Project would comply with Section 1207.5.7 Vegetation Control of the 2022 California Fire Code, which requires clearing combustible vegetation within 10 feet of outdoor energy storage systems.

Additionally, the Project would provide a supply of firefighting water in tanks in accordance with National Fire Protection Association (NFPA) 1142 and as a condition of approval, develop and implement a Fire Safety Plan for all phases of the Project. The plan would be reviewed and approved by the Kings County Fire Department.

With the implementation these requirements—vegetation management, water supply, and a Fire Safety Plan—the Project is not anticipated to expose people or structures, either directly or indirectly, to a significant risk of loss, injury, or death involving wildland fires. While the impact is anticipated to be less than significant, wildfire-related risks will further be evaluated in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

10 Hydrology and Water Quality

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Violate any water quality standards or waste discharge requirements or otherwise substantially degrade surface or ground water quality?	■	□	□	□
b. Substantially decrease groundwater supplies or interfere substantially with groundwater recharge such that the project may impede sustainable groundwater management of the basin?	■	□	□	□
c. Substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner which would:				
(i) Result in substantial erosion or siltation on- or off-site;	■	□	□	□
(ii) Substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site;	■	□	□	□
(iii) Create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff; or	■	□	□	□
(iv) Impede or redirect flood flows?	■	□	□	□
d. In flood hazard, tsunami, or seiche zones, risk release of pollutants due to project inundation?	■	□	□	□
e. Conflict with or obstruct implementation of a water quality control plan or sustainable groundwater management plan?	■	□	□	□

a. Would the project violate any water quality standards or waste discharge requirements or otherwise substantially degrade surface or ground water quality?

Construction and decommissioning of the Project would involve mowing, ground disturbance and grading on portions of the Project site that could loosen existing surface soils and sediments, increasing the potential for erosion during storm events. Ground disturbance, grading, and the installation of impervious surfaces during construction could also alter the existing drainage and infiltration pattern of the site. Impervious and/or disturbed ground surfaces have the potential to increase the rate and volume of stormwater runoff, and thereby potentially increase possible on- or off-site flooding and erosion. Additionally, if groundwater dewatering is needed during construction due to shallow groundwater at the Project site, dewatering activities would discharge directly to the land surface in the vicinity of the construction site and potentially result in soil erosion. During Project construction, operations and maintenance, and decommissioning, any activity that results in the accidental release of pollutants, hazardous materials, or potentially hazardous materials could result in water quality degradation. Application of water for dust suppression or module washing could generate runoff that may transport pollutants (e.g., sediment, dissolved solids). These pollutants could be delivered to surface water bodies during storm events, and/or be infiltrated into groundwater and the underlying aquifer, resulting in the degradation of water quality standards. These impacts are potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project substantially decrease groundwater supplies or interfere substantially with groundwater recharge such that the project may impede sustainable groundwater management of the basin?

Project construction is anticipated to commence in 2028 and to be completed over a period of approximately 24 to 36 months. The Project would require approximately 1,000 acre-feet every 18 to 24 months of the construction period. During Project operations, assuming one solar panel wash cycle per year plus typical O&M activities, the annual water usage is expected to total 80-acre feet per year. This amount includes the water necessary for Project operations, fire suppression, and site maintenance. The water used is anticipated to be purchased from local water purveyors or existing or new onsite water wells. Project-related impacts on groundwater supplies would be potentially significant. A water supply assessment will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

c.(i) Would the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner which would result in substantial erosion or siltation on- or off-site?

See threshold 10a above. Construction and decommissioning of the Project would involve mowing, ground disturbance and grading on portions of the Project site that could loosen existing surface soils and sediments, increasing the potential for erosion during storm events. Ground disturbance, grading, and the installation of impervious surfaces during construction could also alter the existing drainage and infiltration pattern of the site. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c.(ii) Would the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner which would substantially increase the rate or amount of surface runoff in a manner which would result in flooding on- or off-site?

See threshold 10a above. Ground disturbance, grading, and the installation of impervious surfaces during construction could alter the existing drainage and infiltration pattern of the site. Impervious and/or disturbed ground surfaces have the potential to increase the rate and volume of stormwater runoff, and thereby potentially increase possible on- or off-site flooding. Additionally, if groundwater dewatering is needed during construction due to shallow groundwater at the Project site, dewatering activities would discharge directly to the land surface in the vicinity of the construction site. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c.(iii) Would the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner that would create or contribute runoff water which would exceed the capacity of existing or planned stormwater drainage systems or provide substantial additional sources of polluted runoff?

See threshold 10a above. During Project construction, operations and maintenance, and decommissioning, any activity that results in the accidental release of pollutants, hazardous materials, or potentially hazardous materials could result in water quality degradation. Application of water for dust suppression or module washing could generate runoff that may transport pollutants (e.g., sediment, dissolved solids). These pollutants could be delivered to surface water bodies during storm events, and/or be infiltrated into groundwater and the underlying aquifer, resulting in the degradation of water quality standards. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c.(iv) Would the project substantially alter the existing drainage pattern of the site or area, including through the alteration of the course of a stream or river or through the addition of impervious surfaces, in a manner which would impede or redirect flood flows?

The Project site is located approximately 79 miles east of the Pacific Ocean and is not subject to tsunami. There are no large, enclosed bodies of water proximate to the Project site such that the Project could be subject to seiche. The eastern and central portions of the Project site, as well as areas in the western portion of the Project site, are mapped within Flood Zone A, defined as having a 1 percent annual chance of being equaled or exceeded in a given year, also known as the 100-year flood. The remainder of the site is mapped as Flood Zone X, defined as having a minimal flood hazard. (Federal Emergency Management Agency 2025). Therefore, the Project could potentially risk release of pollutants due to inundation and impede or redirect flood flows. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

d. In flood hazard, tsunami, or seiche zones, would the project risk release of pollutants due to project inundation?

See threshold 10c.iv above. The Project site is located approximately 79 miles east of the Pacific Ocean and is not subject to tsunami. There are no large, enclosed bodies of water proximate to the Project site such that the Project could be subject to seiche. The eastern and central portions of the Project site, as well as areas in the western portion of the Project site, are mapped within Flood Zone A, defined as having a 1 percent annual chance of being equaled or exceeded in a given year, also known as the 100-year flood. The remainder of the site is mapped as Flood Zone X, defined as having a minimal flood hazard. (Federal Emergency Management Agency 2025). Therefore, the Project could potentially risk release of pollutants due to inundation. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

e. Would the project conflict with or obstruct implementation of a water quality control plan or sustainable groundwater management plan?

The Project site is located within the Tulare Lake Subbasin of the San Joaquin Valley Groundwater Basin (SJVGB) and is subject to the applicable requirements of the Water Quality Control Plan for the Tulare Lake Basin, administered by the RWQCB in accordance with the Porter-Cologne Water Quality Control Act. There are five Groundwater Sustainability Agencies (GSA) responsible for groundwater management in the Tulare Lake Subbasin, through implementation of a Groundwater Sustainability Plan (GSP) for compliance with the Sustainable Groundwater Management Act (SGMA). The Project site is located within the management area of the Southwest Kings GSA and the Tri-County Water Authority GSA.

The Tulare Lake Subbasin has been historically affected by overdraft conditions, which are being addressed through projects and management actions detailed in the GSP. Also as described in the GSP, there is poor groundwater yield and poor groundwater quality within the Southwest Kings GSA area where the Project site is located; as a result, minimal groundwater is pumped within this GSA, and groundwater levels, quality, and subsidence are maintained at current levels (Tulare Lake Subbasin GSAs 2022). No part of the Tulare Lake Subbasin is adjudicated. The subbasin is classified as a very high priority basin and has been identified as critically overdrafted.

Construction and decommission water usage is anticipated to be approximately 1,000-acre feet every 18 to 24 months of the construction and decommissioning periods. Once operational, the Project is anticipated to use 80-acre feet per year. The water used is anticipated to be purchased from local water purveyors or existing or new onsite water wells. A water supply assessment will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

11 Land Use and Planning

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Physically divide an established community?	☐	☐	■	☐
b. Cause a significant environmental impact due to a conflict with any land use plan, policy, or regulation adopted for the purpose of avoiding or mitigating an environmental effect?	☐	☐	■	☐

a. Would the project physically divide an established community?

The Project would be developed on primarily agricultural land (active and fallow agricultural fields with livestock grazing, irrigation canals and water storage reservoirs). Land uses surrounding the Project site are essentially the same as those on the Project site. The Project site is located in unincorporated Kings County, and nearest community is approximately 10 miles to the east (community of Alpaugh). Therefore, the Project would not physically divide an established community, and no impact would occur with respect to this issue.

Parcel maps would be recorded to eliminate public access easements on properties within the Project site. At this time, the Project Sponsor is not proposing street vacations beyond those public access easements included in the parcel maps. These parcel map requests would not impede public egress or ingress to any adjacent, non-Project properties. The Project also requests the removal of future road reservations at section and mid-section lines within the Project site. Potential impacts would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

b. Would the project cause a significant environmental impact due to a conflict with any land use plan, policy, or regulation adopted for the purpose of avoiding or mitigating an environmental effect?

The Project site General Plan land use designation is General Agriculture – 40 acre and zoning is AG-40 (Agricultural, 40-acre minimum parcel size). Solar energy generation facilities are allowed in "General Agriculture" zone districts (AG-20 and AG-40) by discretionary approval of a Conditional Use Permit. This impact would be less than significant; however, this will be discussed further in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

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12 Mineral Resources

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Result in the loss of availability of a known mineral resource that would be of value to the region and the residents of the state?	☐	☐	☐	■
b. Result in the loss of availability of a locally important mineral resource recovery site delineated on a local general plan, specific plan, or other land use plan?	☐	☐	☐	■

a. Would the project result in the loss of availability of a known mineral resource that would be of value to the region and the residents of the state?

The Project includes the construction, operation and maintenance, and decommissioning of a solar power generation and energy storage facility. The Project would not result in the loss of availability of known mineral resources or of a locally important mineral resource recovery site delineated in the Kings County General Plan Resource Conservation Element (Kings County 2010d). The Project site is not located in a mineral resource zone as defined by the DOC (DOC 2025d). No impact would occur.

NO IMPACT

b. Would the project result in the loss of availability of a locally important mineral resource recovery site delineated on a local general plan, specific plan, or other land use plan?

See threshold 12a above. The Project would not result in the loss of availability of known mineral resources or of a locally important mineral resource recovery site delineated in the Kings County General Plan Resource Conservation Element (Kings County 2010d). The Project site is not located in a mineral resource zone as defined by the DOC (DOC 2025d). No impact would occur.

NO IMPACT

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13 Noise

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project result in:				
a. Generation of a substantial temporary or permanent increase in ambient noise levels in the vicinity of the project in excess of standards established in the local general plan or noise ordinance, or applicable standards of other agencies?	■	□	□	□
b. Generation of excessive groundborne vibration or groundborne noise levels?	■	□	□	□
c. For a project located within the vicinity of a private airstrip or an airport land use plan or, where such a plan has not been adopted, within two miles of a public airport or public use airport, would the project expose people residing or working in the project area to excessive noise levels?	□	□	□	■

a. Would the project result in generation of a substantial temporary or permanent increase in ambient noise levels in the vicinity of the project in excess of standards established in the local general plan or noise ordinance, or applicable standards of other agencies?

Current noise sources at the Project site consist of noise typical of agricultural operations, and from noise generated from vehicles along the I-5. The Project includes the construction, operation and maintenance, and decommissioning of a solar power generation and energy storage facility that would generate noise. Construction and decommissioning would require the use of equipment that would generate noise and groundborne vibration; therefore, it is assumed that noise levels from decommissioning would be similar to those during construction. Primary operational noise sources from Project facilities would be inverter skids, substation transformers, and the BESS units as well as some O&M activities on the Project site. Noise and vibration generated by Project activities could potentially exceed established thresholds within the County's General Plan Noise Element. These impacts would be potentially significant. A noise study will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project result in generation of excessive groundborne vibration or groundborne noise levels?

See threshold 13a above. Noise and vibration generated by Project activities could potentially exceed established thresholds within the County's General Plan Noise Element. These impacts would be potentially significant. A noise study will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

c. For a project located within the vicinity of a private airstrip or an airport land use plan or, where such a plan has not been adopted, within two miles of a public airport or public use airport, would the project expose people residing or working in the project area to excessive noise levels?

The nearest airport to the Project site is the Delano Municipal Airport, located approximately 26 miles southeast of the Project site. The Project is not located in an airport land use plan; therefore, it would not expose people residing or working in the Project area to excessive noise levels. No impact would result from implementation of the Project.

NO IMPACT

14 Population and Housing

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Induce substantial unplanned population growth in an area, either directly (e.g., by proposing new homes and businesses) or indirectly (e.g., through extension of roads or other infrastructure)?	☐	☐	■	☐
b. Displace substantial numbers of existing people or housing, necessitating the construction of replacement housing elsewhere?	☐	☐	☐	■

a. Would the project induce substantial unplanned population growth in an area, either directly (for example, by proposing new homes and businesses) or indirectly (for example, through extension of roads or other infrastructure)?

The Project includes the construction, operation and maintenance, and decommissioning of a solar power generation and energy storage facility. The Project would generate short-term employment during construction and decommissioning, and up to 20 full-time jobs during Project operation. The maximum daily construction workforce required for the Project is anticipated to be 660 personnel; however, the average daily construction workforce is expected to be approximately 470 personnel. These jobs are not expected to induce substantial population growth because Project needs would be primarily met by the existing available Kings County construction labor pool. Non-local construction staff, if any, would stay in local hotels temporarily and would not result in substantial permanent residents in the area.

Based on the California Employment Development Department's unemployment data, the unemployment rate in Kings County is approximately 9.8 percent, which is approximately 15,093 people based on a current population of approximately 154,015 identified by the California Department of Finance (California Department of Finance 2025; California Employment Development Department 2025). The addition of 20 full-time positions in Kings County would assist to reduce unemployment in Kings County but would not result in additional unplanned population growth as the workforce need can be met by the existing available Kings County labor pool. In addition, the Project would not include any new homes or businesses and would not directly induce substantial population growth.

The Project would include internal access roads; however, these roads would be solely for the purpose of access to the Project site during construction, operation and maintenance, and decommissioning and would not increase access to urban areas that would remove obstacles to growth or indirectly increase population growth.

While the Project would contribute to energy supply, which can indirectly support population growth, development of the Project is a response to the State's need for renewable energy to meet

its Renewable Portfolio Standard and would interconnect with the State's infrastructure. The power generated would be added to the State's electricity grid, with the intent that it would displace fossil-fueled power plants and their associated GHG emissions. The Project is not proposed for development as a source of base-load power in response to growth in demand for electricity. Therefore, the Project would not indirectly induce substantial population growth. Impacts of the Project on population growth would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

b. Would the project displace substantial numbers of existing people or housing, necessitating the construction of replacement housing elsewhere?

The Project site is currently comprised of agricultural land, grazing land, fallow land, and scattered vegetation with no housing or other occupied structures present. Therefore, the Project would not displace any housing or people. Because no people or housing would be displaced by construction, operation and maintenance, or decommissioning of the Project, it would not be necessary to construct replacement housing elsewhere and no impact would occur.

NO IMPACT

15 Public Services

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
a. Would the project result in substantial adverse physical impacts associated with the provision of new or physically altered governmental facilities, or the need for new or physically altered governmental facilities, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios, response times or other performance objectives for any of the public services:				
1 Fire protection?	■	□	□	□
2 Police protection?	□	□	■	□
3 Schools?	□	□	□	■
4 Parks?	□	□	□	■
5 Other public facilities?	□	□	□	■

a.1. Would the project result in substantial adverse physical impacts associated with the provision of new or physically altered fire protection facilities, or the need for new or physically altered fire protection facilities, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios, response times or other performance objectives?

The Project site is served by the Kings County Fire Department, with the nearest fire station (Station 9) located approximately 9.6 miles northwest of the Project site in the city of Avenal. As a large-scale electrical infrastructure Project with onsite batteries and high voltage electrical equipment, the Project could pose an increased fire risk compared to existing conditions. However, modern battery systems have seen a 98% reduction in failures per megawatt installed since 2018 (EPRI Product ID 3002030360, 5-10-2024) especially when designed, constructed, and maintained in accordance with the modern fire code and NFPA 855. Additionally, the Project Sponsor only procures small outdoor cabinet-type enclosure battery systems which drastically reduces or eliminates the need for firefighting intervention. Nevertheless, this impact is potentially significant and will be further evaluated in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

a.2. Would the project result in substantial adverse physical impacts associated with the provision of new or physically altered police protection facilities, or the need for new or physically altered police protection facilities, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios, response times or other performance objectives?

Police protection for the Project site is provided by the Kings County Sheriff's Office, with the nearest station located in the city of Avenal, approximately 18 miles northwest of the Project site. The County maintains a 20 minute or faster response time goal for police (Kings County 2010c).

As stated in the Project Description, fencing would be installed around the perimeter of most project facilities, including all areas requiring controlled access for safety and security purposes as needed. Locking gates would be installed at access roads entering the site. Limiting access to the site would be necessary both to ensure the safety of the public and to protect the equipment from potential theft and vandalism. Fencing would remain for the life of the Project. Final fence design will be developed in consultation with appropriate agencies.

Security lighting would also be provided at the O&M building. These security features would minimize the potential need for police protection services at the Project site. No increased demands for police services are anticipated to occur as a result of the Project, and the Project is not expected to result in the increased demand for police protection or new or expanded police facilities. The Project is not anticipated to increase response times to the Project site or surrounding area as operation of the Project would not require a substantial amount of employees. Therefore, the Project would not result in substantial adverse physical impacts associated with the provision of new or physically altered police protection facilities, or the need for new or physically altered police protection facilities, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios, response times or other performance objectives. Therefore, impacts would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

a.3. Would the project result in substantial adverse physical impacts associated with the provision of new or physically altered schools, or the need for new or physically altered schools, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios or other performance objectives?

The Project is a solar power generation and energy storage facility that would neither involve the construction of new or physically altered schools nor result in the need for new or physically altered schools, the construction of which could cause significant environmental impacts. Increases in demand for public services requiring new or physically altered public facilities typically are associated with substantial increases in population. The Project would not include new residences or development on a scale so large that it could not be provided with services through existing facilities. Therefore, no impacts to schools would occur.

NO IMPACT

a.4. Would the project result in substantial adverse physical impacts associated with the provision of new or physically altered parks, or the need for new or physically altered parks, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios or other performance objectives?

The Project is a solar power generation and energy storage facility that would neither involve the construction of new or physically altered parks nor result in the need for new or physically altered parks the construction of which could cause significant environmental impacts. Increases in demand for public services requiring new or physically altered public facilities typically are associated with substantial increases in population. The Project would not include new residences or development on a scale so large that it could not be provided with services through existing facilities. Therefore, no impacts to parks would occur.

NO IMPACT

a.5. Would the project result in substantial adverse physical impacts associated with the provision of other new or physically altered public facilities, or the need for other new or physically altered public facilities, the construction of which could cause significant environmental impacts, in order to maintain acceptable service ratios, response times or other performance objectives?

The Project is a solar power generation and energy storage facility that would neither involve the construction of new or physically altered public facilities nor result in the need for new or physically altered public facilities, the construction of which could cause significant environmental impacts. Increases in demand for public services requiring new or physically altered public facilities typically are associated with substantial increases in population. The Project would not include new residences or development on a scale so large that it could not be provided with services through existing facilities. Therefore, no impacts to public facilities would occur.

NO IMPACT

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16 Recreation

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
a. Would the project increase the use of existing neighborhood and regional parks or other recreational facilities such that substantial physical deterioration of the facility would occur or be accelerated?	☐	☐	☐	■
b. Does the project include recreational facilities or require the construction or expansion of recreational facilities which might have an adverse physical effect on the environment?	☐	☐	☐	■

a. Would the project increase the use of existing neighborhood and regional parks or other recreational facilities such that substantial physical deterioration of the facility would occur or be accelerated?

The Project involves the construction, operation and maintenance, and decommissioning of a solar power facility and energy storage facility. The Project does not propose to construct new or expand existing recreational facilities that might have an adverse physical effect on the environment. Similarly, the Project would not require the construction or expansion of recreational facilities. The Project does not include new residences and would not remove obstacles to growth or indirectly induce substantial population growth. Therefore, the Project would not increase the use of existing neighborhood or regional parks or other recreational facilities such that substantial physical deterioration of the facility would occur or be accelerated or require the construction of new or expanded recreational facilities. No impacts to recreational facilities would occur.

NO IMPACT

b. Does the project include recreational facilities or require the construction or expansion of recreational facilities which might have an adverse physical effect on the environment?

See threshold 16a above. The Project does not propose to construct new or expand existing recreational facilities that might have an adverse physical effect on the environment. Similarly, the Project would not require the construction or expansion of recreational facilities. No impact to recreational facilities would occur.

NO IMPACT

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17 Transportation

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Conflict with a program, plan, ordinance or policy addressing the circulation system, including transit, roadway, bicycle and pedestrian facilities?	■	□	□	□
b. Conflict or be inconsistent with CEQA Guidelines section 15064.3, subdivision (b)?	■	□	□	□
c. Substantially increase hazards due to a geometric design feature (e.g., sharp curves or dangerous intersections) or incompatible use (e.g., farm equipment)?	■	□	□	□
d. Result in inadequate emergency access?	■	□	□	□

a. Would the project conflict with a program, plan, ordinance or policy addressing the circulation system, including transit, roadway, bicycle and pedestrian facilities?

Regional and local plans and policies addressing the circulation system include the County's General Plan Circulation Element, the Kings County Association of Governments' Regional Transportation Plan, and the 2025 Regional Active Transportation Plan.

The Project includes the construction, operation and maintenance, and decommissioning of a solar power generation and energy storage facility. Project construction and decommissioning would generate short-term employment, and would utilize the local circulation system for vehicle trips by workers, material deliveries, and hauling to and from the Project site. The maximum daily construction workforce required for the Project is anticipated to be 660 personnel; however, the average daily construction workforce is expected to be approximately 470 personnel.

While construction traffic would be temporary and would not result in long-term or permanent changes to local roadway conditions, potential temporary impacts to intersections proposed to be utilized for site access could occur during peak hour construction commute periods. This impact would be potentially significant.

It is anticipated that approximately 10 to 20 permanent, full-time employees would be needed to maintain the facility which would result in minimal vehicle trips to and from the Project site. During Project operations, solar panel washing is expected to occur one time per year, requiring minimal additional vehicle trips to the Project site. The minimal level of additional trips generated as a result of the Project would have minimal potential to conflict with a program, plan, ordinance, or policy addressing the circulation system, including transit, roadway, bicycle and pedestrian facilities once the construction phase is complete. During construction, potential temporary impacts to

intersections proposed to be utilized for site access could occur which may conflict with an existing program, plan, or ordinance. These impacts would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project conflict or be inconsistent with CEQA Guidelines section 15064.3, subdivision (b)?

Vehicle miles traveled (VMT) primarily is a metric for assessing Project-related GHG emissions impacts. The analysis related to GHG emissions associated with Project-related construction traffic is analyzed elsewhere in this Initial Study and will be studied in an EIR. Overall, as the Project would provide clean renewable energy to the grid and would offset GHG emissions that would have otherwise resulted from producing energy from a non-renewable source, the Project will have a net beneficial impact in offsetting GHG emissions.

However, the construction and decommissioning phase of the Project would generate vehicle miles traveled associated with vehicle trips by workers, material deliveries, and hauling to and from the Project site. Construction and decommissioning would be temporary and would not result in permanent, long-term increases in regional vehicle miles traveled because workers and materials would be primarily sourced locally and would be temporary. Nonetheless, approximately 470 construction workers would commute to the site and generate additional VMTs during that time period.

Potential impacts associated with operational vehicle miles traveled are determined based on guidance from the Governor's Office of Land Use and Climate Innovation's *Technical Advisory on Evaluating Transportation Impacts in CEQA* (Governor's Office of Land Use and Climate Innovation 2018). Pursuant to the technical advisory, in general, projects that generate fewer than 110 trips per day may be assumed to cause a less than significant transportation impact (Governor's Office of Land Use and Climate Innovation 2018). Given that operation of the Project would result in between 10 to 20 full-time employees to maintain the facility, with minimal additional vehicle trips required for panel washing occurring once or twice per year, Project operation would not result in a long-term trip generation rate exceeding 110 trips per day. Therefore, operation of the Project would not conflict or be inconsistent with CEQA Guidelines Section 15064.3(b). However, a VMT analysis will be prepared for the Project and further evaluated in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project substantially increase hazards due to a geometric design feature (e.g., sharp curves or dangerous intersections) or incompatible use (e.g., farm equipment)?

Construction and decommissioning of the Project would require the delivery of heavy construction equipment and facility materials, some of which may require transport by oversize vehicles. The volume of traffic generated by the Project would be greatest during the construction and decommissioning phases. This would include regular deliveries of materials and equipment by large trucks. Slow moving trucks could result in temporary congestion near the Project entrance, and could pose a safety concern due to abrupt changes in the speed of traffic flow, or due to slow turning movements across on-coming lanes of traffic. Delivery truck traffic could also interact with the slow-moving farm equipment and vehicles utilizing the roadway. The use of oversized vehicles during construction and decommissioning can create a hazard to the public by limiting motorist views on roadways and by the obstruction of space. Construction-related oversize vehicle loads must comply with permit-related and other requirements of the California Vehicle Code and the

California Streets and Highway Code. California Highway Patrol escorts may be required at the discretion of Caltrans and the County and would be detailed in respective oversize load permits. Therefore, the Project has the potential to increase hazards due to a compatible use or result in inadequate emergency access during construction and decommissioning. These impacts are potentially significant.

Although existing roadways would be used to the greatest extent possible, new unpaved Project access roads would be constructed to provide internal access from the existing road network. Project access roads would be up to 30 feet wide and would be cleared and compacted for travel and access to the solar blocks. Access roads would be designed in accordance with Kings County Fire Department and County standards which would minimize the potential for hazardous transportation facilities due to a geometric design feature, and this impact would be less than significant.

Dirt roads that are located within the boundaries of the Project, and that are not the subject of a written easement, would be closed to third-party use for vehicular or other access. Any such third parties that may currently use these dirt roads would be rerouted to public access routes, which would remain open. The Project does not propose any modifications to the existing public access routes beyond the new ingress/egress points. During construction, the use of large trucks and vehicles could potentially cause a hazard and result in temporary inadequate emergency access. Therefore, these impacts would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

d. Would the project result in inadequate emergency access?

See threshold 17c above for a discussion of traffic disturbances related to construction and decommissioning activities. Access to the Project site would be provided via one primary and two additional ingress/egress points along the existing Utica Avenue exit from I-5. The Health and Safety Element of the 2035 Kings County General Plan designates evacuation routes to be relied upon for emergency or disaster responses. While no designated evacuation routes occur within the Project area, I-5 and Utica Avenue are designated primary and secondary emergency access routes, respectively, adjacent to the Project site (Kings County 2010). The Project would not block or inhibit emergency access to the site or nearby land uses via these designated routes during construction or operation. These routes would remain open throughout construction, and emergency access would not be limited by construction activities at the Project site. However, this impact will further be evaluated in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

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18 Tribal Cultural Resources

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project cause a substantial adverse change in the significance of a tribal cultural resource, defined in a Public Resources Code Section 21074 as either a site, feature, place, or cultural landscape that is geographically defined in terms of the size and scope of the landscape, sacred place, or object with cultural value to a California Native American tribe, and that is:				
a. Listed or eligible for listing in the California Register of Historical Resources, or in a local register of historical resources as defined in Public Resources Code Section 5020.1(k)?	■	□	□	□
b. A resource determined by the lead agency, in its discretion and supported by substantial evidence, to be significant pursuant to criteria set forth in subdivision (c) of Public Resources Code Section 5024.1? In applying the criteria set forth in subdivision (c) of Public Resources Code Section 5024.1, the lead agency shall consider the significance of the resource to a California Native American tribe.	■	□	□	□

a. Would the project cause a substantial adverse change in the significance of a tribal cultural resource as defined in Public Resources Code Section 21074 that is listed or eligible for listing in the California Register of Historical Resources, or in a local register of historical resources as defined in Public Resources Code Section 5020.1(k)?

The County is conducting formal consultation pursuant to Assembly Bill 52 in order to determine the potential for the Project to result in impacts to tribal cultural resources.

A consultation notice packet was sent to the Santa Rosa Rancheria Tachi Yokut Tribe on August 7, 2025. To this date, no requests for consultation have been received. The tribe will be included on all further public notices for the Project. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project cause a substantial adverse change in the significance of a tribal cultural resource as defined in Public Resources Code 21074 that is a resource determined by the lead agency, in its discretion and supported by substantial evidence, to be significant pursuant to criteria set forth in subdivision (c) of Public Resources Code Section 5024.1?

See threshold 18a above.

POTENTIALLY SIGNIFICANT IMPACT

19 Utilities and Service Systems

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Would the project:				
a. Require or result in the relocation or construction of new or expanded water, wastewater treatment or storm water drainage, electric power, natural gas, or telecommunications facilities, the construction or relocation of which could cause significant environmental effects?	■	□	□	□
b. Have sufficient water supplies available to serve the project and reasonably foreseeable future development during normal, dry and multiple dry years?	■	□	□	□
c. Result in a determination by the wastewater treatment provider which serves or may serve the project that it has adequate capacity to serve the project's projected demand in addition to the provider's existing commitments?	□	□	■	□
d. Generate solid waste in excess of State or local standards, or in excess of the capacity of local infrastructure, or otherwise impair the attainment of solid waste reduction goals?	□	□	■	□
e. Comply with federal, state, and local management and reduction statutes and regulations related to solid waste?	□	□	■	□

a. Would the project require or result in the relocation or construction of new or expanded water, wastewater treatment or storm water drainage, electric power, natural gas, or telecommunications facilities, the construction or relocation of which could cause significant environmental effects?

Water

Water would be required for the construction, operation and maintenance, and decommissioning of the Project. Construction water demand is anticipated to be approximately 1,000 acre-feet over an 18 to24 month period, primarily for dust suppression and soil compaction. Operational water demand is estimated at 86 acre-feet per year for PV panel washing and O&M activities. Water for the Project is anticipated to be purchased from local water purveyors or via existing or new onsite water

wells, as municipal water service is not available in the Project vicinity. This water is anticipated to be sourced from the San Joaquin Valley – Tulare Lake Groundwater Basin. Well water is not expected to require treatment and could be pulled directly from the well for construction and operational use. For these reasons, the proposed Project would not require or result in the construction of new or expanded water facilities, which could cause significant environmental effects. Potential impacts during construction, operation and maintenance, and decommissioning would be less than significant.

Wastewater

Because the site is in a rural location and outside of a municipal sphere of influence, no wastewater service is currently provided to the Project site. The Project would not require or result in the relocation or construction of new or expanded municipal wastewater facilities, and no connection to a public wastewater system is required or proposed.

During construction and decommissioning, portable restroom facilities would be provided and serviced by licensed providers.

During Project operations, permanent restroom facilities with septic tanks and/or portable toilets would be used for sanitary purposes. Wastewater would be discharged into a 1,500-gallon septic tank, if one is installed. The septic tank, if installed, would be designed and constructed consistent with applicable state and County requirements. Specifically, the Project would adhere to the Kings County Environmental Health Department septic system permit requirements, including the requirements of the King County Code of Ordinances Article VI Plumbing Code and the Kings County Local Area Management Program, which set out procedures for the design, installation, and operation of on-site wastewater treatment systems. As part of the requirements, applicants proposing to install septic systems must conduct test holes to identify subsurface conditions and conduct percolation tests to adequately demonstrate the range of percolation rates within the primary disposal area. Septic system design would be reviewed and approved by the Kings County Building Department prior to construction. These regulatory requirements and oversight would ensure the septic system is appropriately sized, meets applicable standards, and would not leach into groundwater. If no septic system is installed, the Project would utilize portable toilets serviced and maintained by an outside vendor. No off-site sewage or disposal connections to a municipal sewer system exist or are proposed. However, this impact would be potentially significant and will be further evaluated in the EIR.

Stormwater

There are no constructed stormwater drainage systems present on-site and stormwater on the Project site either percolates on-site or drains off-site by way of existing natural drainage features. Under current conditions, pooling is possible on site, especially within designated flood zones. While no off-site connections to a municipal stormwater facility exist or are proposed, the Project would increase impervious surfaces at the Project site that would require implementation of stormwater controls and on-site retention measures. This impact would be potentially significant.

Electric Power, Natural Gas, and Telecommunications

No natural gas is anticipated to be required for construction, operation and maintenance, and decommissioning of the Project. The Project would not require or result in the construction of new gas facilities which could cause significant environmental effects; thus, no impact would occur.

Power required for Project construction and decommissioning activities, such as for the temporary construction trailers, would be provided via portable generators or temporary local distribution. Power for operational and maintenance activities would be provided by the proposed Project's electrical generation or supplied by the local power provider. PG&E operates distribution circuits in the Project area. The Project Sponsor would coordinate with PG&E to obtain access to a nearby distribution circuit, if needed.

Telecommunications via fiber optic lines would not be required to support construction or decommissioning activities; however, would be required to support operation and maintenance facilities. The proposed Project would use local exchange carrier services for telecommunication to support remote monitoring requirements. It would connect to telecommunication fiberoptic lines owned and managed by a local provider. No off-site telecommunication connections are proposed. This impact would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Would the project have sufficient water supplies available to serve the project and reasonably foreseeable future development during normal, dry and multiple dry years?

The proposed Project would require an estimated 1,000-acre feet of water every 18 to 24 months of the construction period for common construction-related purposes, including, but not limited to, dust suppression, soil compaction, and grading and dust control for ingress and egress of on-site construction vehicle equipment traffic and for Project facilities construction. During operations, water would be required for PV panel washing, equipment washing, and other miscellaneous water uses including O&M building use, and a permanent water source in the form of trucked water, well water, or bottled water would be provided for staff. The Project would include a number of aboveground fire water tanks for operations, exposure cooling, and non-battery fire suppression. Annual water usage during operations is expected to be approximately 86-acre feet of water. This amount includes the water necessary for Project operations, fire suppression, and site maintenance. A water supply assessment will be prepared for the Project to evaluate potential impacts, which will be discussed further in an EIR.

POTENTIALLY SIGNIFICANT IMPACT

c. Would the project result in a determination by the wastewater treatment provider which serves or may serve the project that it has adequate capacity to serve the project's projected demand in addition to the provider's existing commitments?

Wastewater service is not currently provided to the Project site. During Project construction and decommissioning, portable restroom facilities would be provided and serviced by licensed providers. Wastewater from the Project's O&M building would be discharged into a septic tank, if one is installed, where most of the solids would be removed. The septic tank would be a maximum of 1,500 gallons and would be designed and constructed according to County requirements. If no septic system is installed, the Project would be served by portable toilets to be serviced and maintained by an outside vendor. Because the Project would not require the construction of new wastewater facilities, the construction of which could cause significant environmental effects, the resulting impact would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

d. Would the project generate solid waste in excess of State or local standards, or in excess of the capacity of local infrastructure, or otherwise impair the attainment of solid waste reduction goals?

The closest Class III municipal landfill is the Avenal Regional Landfill, located approximately 17 miles northwest of the northwestern corner of the Project site boundary at 201 North Hydril Road. The Avenal Regional Landfill has a remaining capacity of approximately 27,300,000 cubic yards and a projected cease operation date of 2056 (California Department of Resources, Recycling, and Recovery 2025).

It is not anticipated that the Project would generate substantial amounts of nonrecyclable waste during construction. Currently, the Project site is largely undeveloped and, therefore, there would be minimal demolition or removal of large debris. Materials brought to the Project site during construction would be used to construct facilities, and few residual materials are expected. Common construction waste may include paper, wood, glass, plastics from packing material, waste lumber, insulation, scrap metal and concrete, empty nonhazardous containers, and vegetation wastes. Any hazardous waste generated during construction would be disposed of at an approved location. Operations and maintenance would include approximately 20 permanent employees, and are not anticipated to produce a substantial amount of solid waste. Solar PV panels contain valuable materials that would likely be reused and recycled at the end of their useful life. Solar panel manufacturers have identified that approximately 90 percent of materials in solar panel modules can be recycled. Batteries will be discharged, hauled off-site, and recycled or disposed of at an appropriate location in accordance with all applicable hazardous waste requirements. Decommissioning would occur after the Avenal Regional Landfill reaches its permitted capacity; therefore, decommissioning waste would be hauled to the nearest active landfill. Potential impacts related to landfill capacity are anticipated to be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

e. Would the project comply with federal, state, and local management and reduction statutes and regulations related to solid waste?

See threshold 19d above. Spent Project materials and solid waste generated throughout the Project's useful lifetime would be recycled to the extent feasible. Remaining solid waste would be disposed of at licensed facilities and in compliance with federal, state, and local regulations. This impact would be less than significant.

LESS-THAN-SIGNIFICANT IMPACT

20 Wildfire

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
If located in or near state responsibility areas or lands classified as very high fire hazard severity zones, would the project:				
a. Substantially impair an adopted emergency response plan or emergency evacuation plan?	☐	☐	■	☐
b. Due to slope, prevailing winds, and other factors, exacerbate wildfire risks and thereby expose project occupants to pollutant concentrations from a wildfire or the uncontrolled spread of a wildfire?	☐	☐	■	☐
c. Require the installation or maintenance of associated infrastructure (such as roads, fuel breaks, emergency water sources, power lines or other utilities) that may exacerbate fire risk or that may result in temporary or ongoing impacts to the environment?	☐	☐	■	☐
d. Expose people or structures to significant risks, including downslopes or downstream flooding or landslides, as a result of runoff, post-fire slope instability, or drainage changes?	☐	☐	■	☐

a. If located in or near state responsibility areas or lands classified as very high fire hazard severity zones, would the project substantially impair an adopted emergency response plan or emergency evacuation plan?

The Project site is not located in a state responsibility area or very high fire severity hazard zone (California Department of Forestry and Fire Protection [CAL FIRE] 2025). The Project site is in a local responsibility area. According to the Fire Hazard Severity Zone map, the nearest Very High Fire Hazard Severity zone in a state responsibility area is located in the Diablo Range at the western edge of Kings County, approximately 16 miles west from the Project site.

The Health and Safety Element of the 2035 Kings County General Plan designates evacuation routes to be relied upon for emergency or disaster responses. Within the Project area, the primary evacuation route includes I-5, and the secondary evacuation route is Utica Avenue (Kings County 2010c).

As a large-scale electrical infrastructure Project with onsite batteries and high voltage electrical equipment, the Project could pose an increased fire risk compared to existing conditions. While batteries generally burn with difficulty, they can potentially catch fire, but the fires do not produce embers and thus do not contribute to wildfire risk with proper on-site vegetation management. Due to planned ongoing vegetation management, the Project is not anticipated to significantly increase the risk of wildfire. The Project would additionally be required to develop and implement a fire safety plan for all phases of the Project in coordination with the Kings County Fire Department. While it is anticipated this impact would be less than significant, potential impact as it relates to wildfire will be further discussed in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

b. If located in or near state responsibility areas or lands classified as very high fire hazard severity zones, would the project, due to slope, prevailing winds, and other factors, exacerbate wildfire risks and thereby expose project occupants to pollutant concentrations from a wildfire or the uncontrolled spread of a wildfire?

See threshold 20a above. Fire moves faster under hot, dry, and windy conditions. Wind may also blow embers ahead of a fire, causing its spread. Wildfire smoke produced from combustion of natural biomass contains thousands of individual compounds, including particulate matter, carbon dioxide, water vapor, carbon monoxide, hydrocarbons and other organic chemicals, nitrogen oxides, and trace minerals that can be carried in the wind. Slope can determine how quickly a fire spreads. The Project site and surrounding area are generally flat. The Project would not introduce or enhance any large slopes in the area. As such, the Project would not subject surrounding land uses to exacerbated fire risk due to any existing or altered topographic feature. While it is anticipated this impact would be less than significant, potential impact as it relates to wildfire will be further discussed in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

c. If located in or near state responsibility areas or lands classified as very high fire hazard severity zones, would the project require the installation or maintenance of associated infrastructure (such as roads, fuel breaks, emergency water sources, power lines or other utilities) that may exacerbate fire risk or that may result in temporary or ongoing impacts to the environment?

See threshold 20a above. The Project would include a gen-tie line within or immediately adjacent to the Project site, as well as other electrical components, such as transformers, inverters, and substations. It would also include on-site access roads. The installation and maintenance of associated infrastructure is not anticipated to exacerbate fire risk or result in temporary or ongoing impacts to the environment. While it is anticipated this impact would be less than significant, potential impact as it relates to wildfire will be further discussed in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

d. If located in or near state responsibility areas or lands classified as very high fire hazard severity zones, would the project expose people or structures to significant risks, including downslopes or downstream flooding or landslides, as a result of runoff, post-fire slope instability, or drainage changes?

See threshold 20a and 20b above. The Project site and surrounding area is generally flat and not subject to landslides. The Project would not create slopes that result in the potential for downslope flooding or landslides as a result of post-fire slope instability. Therefore, the Project would not expose people or structures to significant risks, including downslope or downstream flooding or landslides, as a result of runoff, post-fire slope instability, or drainage changes. While it is anticipated this impact would be less than significant, potential impact as it relates to wildfire will be further discussed in an EIR.

LESS-THAN-SIGNIFICANT IMPACT

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21 Mandatory Findings of Significance

	Potentially Significant Impact	Less than Significant with Mitigation Incorporated	Less-than - Significant Impact	No Impact
Does the project:				
a. Have the potential to substantially degrade the quality of the environment, substantially reduce the habitat of a fish or wildlife species, cause a fish or wildlife population to drop below self-sustaining levels, threaten to eliminate a plant or animal community, substantially reduce the number or restrict the range of a rare or endangered plant or animal or eliminate important examples of the major periods of California history or prehistory?	■	□	□	□
b. Have impacts that are individually limited, but cumulatively considerable? ("Cumulatively considerable" means that the incremental effects of a project are considerable when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probable future projects)?	■	□	□	□
c. Have environmental effects which will cause substantial adverse effects on human beings, either directly or indirectly?	■	□	□	□

a. Does the project have the potential to substantially degrade the quality of the environment, substantially reduce the habitat of a fish or wildlife species, cause a fish or wildlife population to drop below self-sustaining levels, threaten to eliminate a plant or animal community, substantially reduce the number or restrict the range of a rare or endangered plant or animal or eliminate important examples of the major periods of California history or prehistory?

As noted under Section 4, *Biological Resources*, and Section 5, *Cultural Resources*, implementation of the proposed Project would have potentially significant impacts on special status species and cultural and paleontological resources. This impact is potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

b. Does the project have impacts that are individually limited, but cumulatively considerable? ("Cumulatively considerable" means that the incremental effects of a project are considerable when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probable future projects)?

The Project would have no impact on subtopics within aesthetics (scenic vistas and scenic resources within a State Scenic Highway), forestry resources, air quality (emissions leading to odors adversely affecting a substantial number of people), biological resources (conflict with an adopted habitat conservation plan), cultural resources (significance of historical resources), geology and soils (landslides), hazards and hazardous materials (hazards within 0.25 mile of a school, location on a hazardous materials site, and hazards within two miles of an airport), mineral resources, noise (exposure within two miles of an airport), population and housing (displacement of people or housing), public services (schools, parks, and other public facilities), or recreation. Thus, the Project would not contribute to cumulative impacts for these resource topics.

Section 15355 of the *CEQA Guidelines* defines a cumulative impact as the condition under which "two or more individual effects which, when considered together, are considerable or which compound or increase other environmental impacts. The cumulative impact from several projects is the change in the environment which results from the incremental impact of the Project when added to other closely related past, present, and reasonably foreseeable probable future projects. Cumulative impacts can result from individually minor but collectively significant projects taking place over a period of time" (CCR Section 15355). Impacts of the Project in conjunction with other projects in the surrounding area may be cumulatively considerable relative to aesthetics, agriculture, air quality, biological resources, cultural resources, geology and soils, GHG emissions, hazards and hazardous materials, hydrology and water quality, land use, noise, public services, transportation, tribal cultural resources, utilities and service systems, and wildfire. Cumulative impacts of the Project would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

c. Does the project have environmental effects which will cause substantial adverse effects on human beings, either directly or indirectly?

In general, impacts to human beings are associated with air quality, hazards and hazardous materials, geology and soils, noise, and traffic safety. As detailed in the preceding responses, the Project would potentially have environmental effects that cause substantial adverse effects in the issue areas of air quality, geology and soils, hazards and hazardous materials, and traffic safety. Impacts to human beings would be potentially significant.

POTENTIALLY SIGNIFICANT IMPACT

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List of Preparers

Rincon Consultants prepared this Initial Study under contract to Kings County. Persons involved in data gathering analysis, project management, and quality control are listed below.

Rincon Consultants

Shauna Callery, Director of Environmental Planning
Nicole West, Senior Supervising Environmental Planner
Chris Schmuckal, Senior Environmental Planner
Ethan Knox, Environmental Planner

Appendix A

APN List

APN	Acres
46010012000	44.16
46010013000	2.5
46010014000	2.5
46010015000	2.5
46010016000	639.07
46010017000	601.22
46010018000	626.67
46010019000	620.53
46010020000	607.64
46010021000	616.43
46080001000	25.81
46080004000	627.88
46080005000	617.18
46080006000	608.83
46080007000	619.39
46080010000	632.87
46080011000	620.75
48030016000	160
48030017000	80
48030026000	315.26
48030027000	239.7
48030028000	85.04
48040007000	48
48040008000	13.76
48040009000	17.92
48040010000	40
48040011000	40
48040012000	40
48040013000	40.5
48040014000	40
48040015000	40
48040020000	100
48040021000	20
48040022000	5
48040023000	11
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48040053000	10
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48040055000	13.33
48040056000	113.33
48040057000	173.33
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48040061000	160
48190013000	160
48190005000	144.83

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48190014000	607.24
48190018000	225.99
48190019000	84.07
48190020000	327.6
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48200051000	200
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48260005000	480
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48270012000	526
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48280026000	320
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48280028000	522.33
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Appendix B

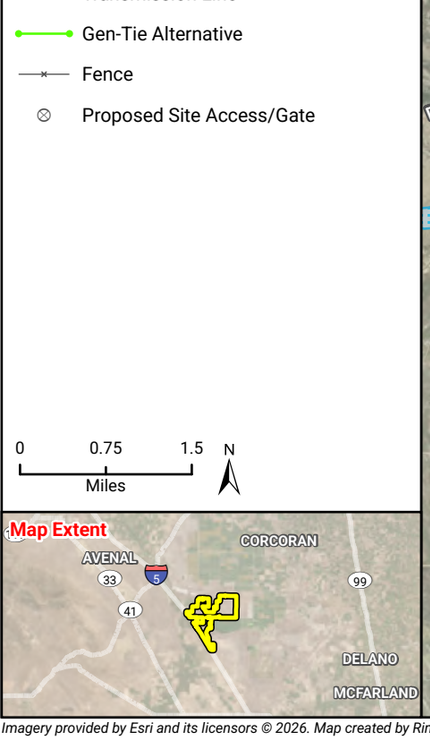
Preliminary Site Plan

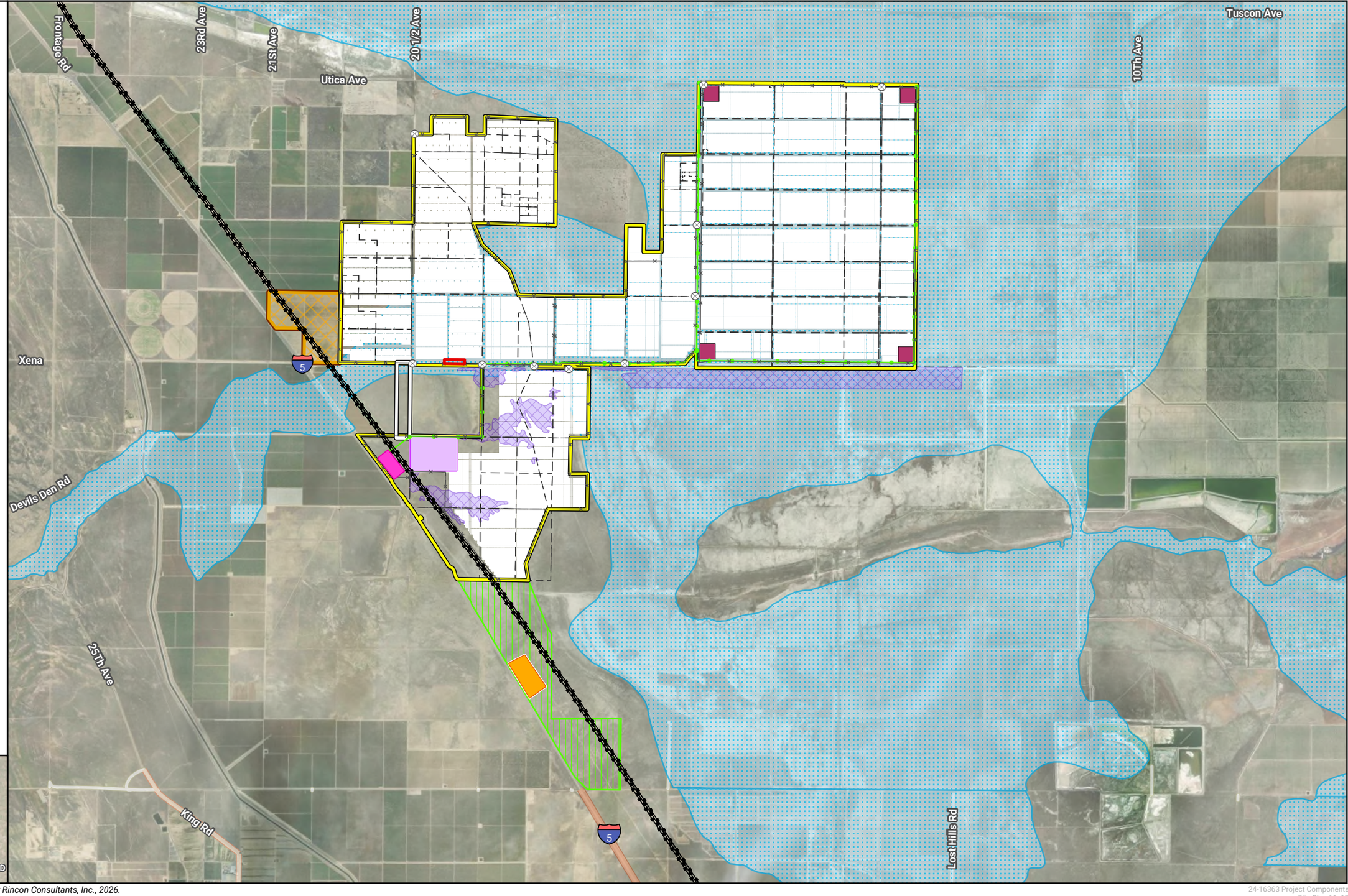
Terra-Gen Rex Solaire Project
Site Plan
Kings County, California

- Project Site
- Parcel Exclusion
- Switchyard Gen-Tie Area
- Substation
- MV Corridor-Alternate
- BESS Area
- Switchyard
- Optional BESS Substation
- Optional Sub Switchyard BESS-Approximate
- FEMA 1% Annual Flood Hazard Area
- NWI Waters
- Proposed Solar Array
- Parcel Boundary
- Transmission Line
- Gen-Tie Alternative
- Fence
- Proposed Site Access/Gate

0 0.75 1.5 Miles

N





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Appendix C

Geotechnical Engineering Report

Preliminary Geotechnical Engineering Report

Rex Solaire Solar Project
Kings County, California

Prepared for
Terra-Gen Development Company, LLC

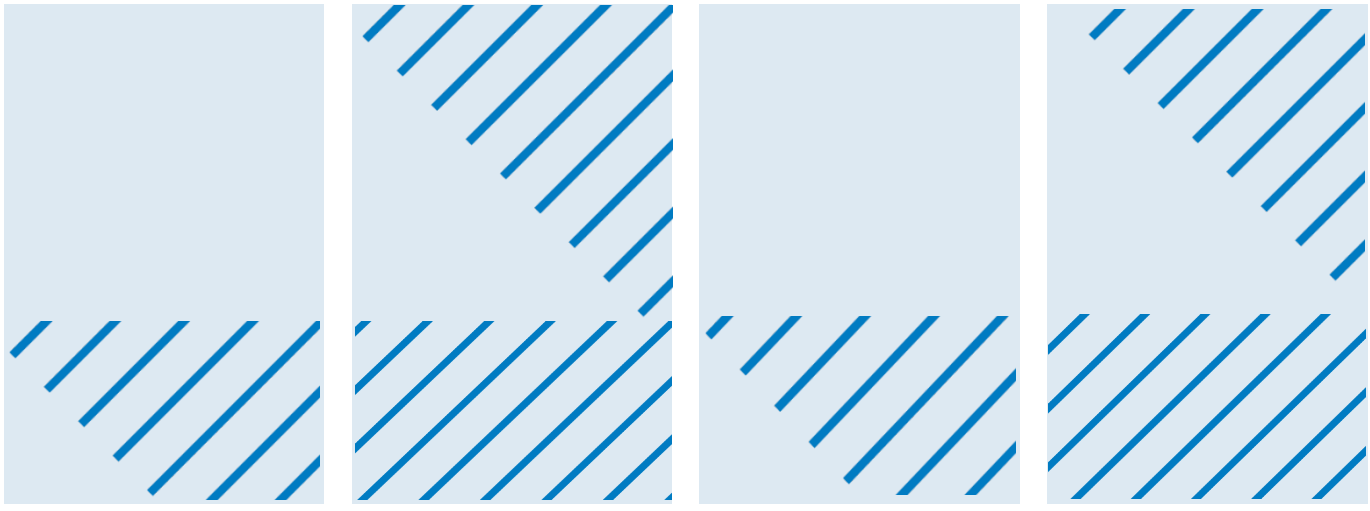
Prepared by
Barr Engineering Co.

October 2024

DRAFT

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Certification

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of California.

DRAFT FOR REVIEW (10-25-2024)
Jedediah D. Greenwood
PE #: 79417

October 25, 2024
Date

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Certified Engineering Geologist under the laws of the State of California.

DRAFT FOR REVIEW (10-25-2024)
Jedediah D. Greenwood
CEG #: 2558

October 25, 2024
Date

Preliminary Geotechnical Engineering Report

Rex Solaire Solar Project
Kings County, California

October 2024

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Executive Summary

Barr Engineering Co., under authorization and contract with Terra-Gen Development Company, LLC, completed a preliminary geotechnical investigation of the Rex Solaire Solar Project located in Kings County, California. The Rex Solaire Solar Project is a proposed solar development project consisting of a series of solar photovoltaic (PV) modules spread across approximately 18,500 acres and a project substation.

The preliminary geotechnical investigation consisted of cone penetration test soundings, test pit excavations, and soil sampling. Laboratory testing was conducted on selected soil samples from the test pits. Electrical resistivity testing and laboratory thermal resistivity testing were performed in support of the project electrical design (to be completed by others). This geotechnical investigation program was selected to develop preliminary design parameters of the soils at the project site and to provide a general discussion of construction considerations.

The typical stratigraphy, as determined from field data, consists of a layer of topsoil overing clay, silt, and sand layers of varying thicknesses. Based on the CPT-interpreted soil types, a majority of the materials were classified as silt mixtures, sand mixtures, and sands, with occasional layers of clay and gravelly sand. The results of the test pit excavations indicated varying subgrade conditions, including locations that encountered primarily cohesive soils and locations that encountered primary granular soils within the upper 10 feet below existing grade.

The project site is located in an active seismic area in relatively close proximity to the San Andreas and other significant faults. The geotechnical data collected as part of this investigation was used to complete a preliminary liquefaction susceptibility assessment. The results indicate that approximately 15 to 75 percent of the soils at individual investigation locations are prone to liquefaction over the depths of the CPT soundings when a magnitude 6.5 earthquake is applied. Liquefaction poses a risk to structures due to the potential for liquefaction-induced bearing failure, excessive settlement, and/or lateral spreading. The preliminary susceptibility analysis presented in this report is a simplified analysis based on available data, and a comprehensive liquefaction analysis is recommended during later phases of design. Please note that the preliminary recommendations for foundation design presented within this report exclude the potential impacts of liquefaction.

The primary findings of the preliminary geotechnical exploration and analysis are presented below.

- Results of the field investigation indicate groundwater levels ranging from approximately 4 to 22 feet below existing grade, with groundwater levels observed to be shallower than 10 feet at 10 of the 22 investigation locations. Based on these results, groundwater is anticipated to be a factor in foundation design and construction.

- Solar modules are commonly supported on steel piles as they are typically the most economical and, thus, the most desirable foundation system. Similarly, steel pile foundations may be used for inverters and other lightweight equipment. Barr presently assumes steel piles will be used for this project and should be contacted if alternate foundation systems are considered for foundation design.

    - The results of the geotechnical exploration indicate varying subsurface conditions throughout the project site. To account for this variability, Barr recommends three design profiles at the project site on a preliminary basis. The design profiles were determined

primarily based on the CPT-interpreted soil behavior type at each investigation location. Based on the geotechnical analysis for an axial tension loading case, an axial compression loading case, and a lateral loading case, the recommended minimum embedment depths are summarized below (which exclude other factors such as liquefaction and soil swell). The preliminary pile size and embedment is based on geotechnical analysis only (not structural capacity).

- Design Profile 1 (generally coincides with locations determined to have primarily CPT-interpreted cohesive soils in the upper 10 feet below existing grade):
  - Exterior Columns (W8x15): embedment depth of 10 feet
  - Interior Columns (W8x13): embedment depth of 9 feet
- Design Profile 2 (generally coincides with locations determined to have a combination of CPT-interpreted cohesive and granular soils in the upper 10 feet below existing grade):
  - Exterior Columns (W8x13): embedment depth of 8.5 feet
  - Interior Columns (W8x13): embedment depth of 8 feet
- Design Profile 3 (generally coincides with locations determined to have primarily CPT-interpreted granular soils in the upper 10 feet below existing grade):
  - Exterior Columns (W8x13): embedment depth of 8.5 feet
  - Interior Columns (W8x13): embedment depth of 7.5 feet

- Of concern for pile installation is the presence of driving obstructions such as dense and/or cemented soils, gravels, cobbles, and boulders at the project site. Results of the geotechnical investigation generally indicate that pile driving appears feasible with standard pile driving equipment. Each of the 22 CPT soundings performed during the preliminary geotechnical investigation reached the target depths ranging from 20 to 100 feet below grade. In addition, each test pit reached the target depth of 10 feet below existing grade with a rubber-tired backhoe.

- The presence of liquefiable soils results in seismic Site Class F, which requires that a site response analysis be conducted to determine the geotechnical design criteria per ASCE 7. The site response analysis estimates amplification or de-amplification of seismic waves as they propagate upward from the bedrock surface through the soil profile to the ground surface, which is determined based on subsurface stratigraphy, soil type, shear wave velocity profile, and input accelerations across the project site. A site response analysis will need be conducted during later phases of design.

- The surficial soils are susceptible to wind and water erosion. Long-term erosion control measures for the project site largely consist of revegetation for all areas of the project site where it can be performed following construction. Revegetation will help control erosion from both wind and water. In areas where revegetation is not possible, wind erosion should be anticipated. Control of water erosion will generally consist of the design of erosion control structures, berms, diversion devices, and other drainage control measures to limit drainage to confined zones away from the structures. The design of specific erosion control structures should be performed by a civil

engineer that specializes in this practice. A site-specific hydrology study is recommended prior to final design to understand the implications of scour.

- The results of thermal resistivity testing performed on bulk samples collected during the preliminary geotechnical investigation indicate the thermal resistivity of samples re-compacted to a density equivalent to approximately 85 percent of the standard Proctor density near their as-received moisture contents ranges from 85 to 120°C-cm/W at the as-received moisture content and from 245 to 389°C-cm/W in the fully dried condition. The thermal resistivity of the sample collected at substation investigation location SUB-Opt1-01 re-compacted to a density equivalent to approximately 95 percent of the standard Proctor density at the lowest moisture content necessary to reach the target compaction is 120°C-cm/W near the as-received moisture content and 209°C-cm/W in the fully dried condition.

- The results of the field and laboratory electrical resistivity testing indicate that the soils at the project site can generally be classified as severely corrosive to progressively less corrosive to steel when at typical in-situ moisture content.

- Based on the results of analytical testing, cement with a sulfate exposure category of S2 appears suitable for foundation design. The foundation engineer of record has final responsibility for selection or approval of an appropriate concrete mix design.

1 Introduction

Barr Engineering Co. (Barr), under authorization and contract with Terra-Gen Development Company, LLC (Terra-Gen), completed a preliminary geotechnical investigation of the Rex Solaire Solar Project located in Kings County, California. The Rex Solaire Solar Project is a proposed solar development project consisting of a series of solar photovoltaic (PV) modules spread across approximately 18,500 acres and a project substation.

This report describes the investigation and testing performed, presents the results of the work, and provides geotechnical analyses and preliminary recommendations for structure foundations, as well as general construction considerations. This report addresses geologic and geotechnical risks and summarizes the preliminary geotechnical investigation completed at selected locations spread throughout the proposed development.

1.1 Site Location

The proposed Rex Solaire Solar Project is located in southern Kings County, California, as shown in Figure 1. Figure 2 shows the project boundary and proposed substation location based on the project boundary provided by Terra-Gen on February 8, 2024 (Terra-Gen, 2024). The coordinates of the geotechnical test locations are provided in Appendix A.

1.2 Previous Investigations

No previous geotechnical investigations are known by Barr to have been completed for the Rex Solaire Solar Project.

1.3 Project Layout

This report incorporates the project area provided to Barr by Terra-Gen on February 8, 2024 (Terra-Gen, 2024).

2 Site Geology and Geohazards

2.1 Regional Physiography

The project site is located in the Great Valley geomorphic province of California, an alluvial plain roughly 50 miles wide and 400 miles long. The northern portion of the Great Valley geomorphic province is the Sacramento Valley drained by the Sacramento River, and the southern portion of the Great Valley, in which the project site is located, is the San Joaquin Valley drained by the San Joaquin River. The Great Valley is a depression in which sediments from the adjacent mountains have been deposited in since the Jurassic Period about 160 million years ago (CGS, 2002).

2.2 Topography

The site topography is indicated on Figure 3. In general, the project site has relatively low topographic relief. Elevations at the project site range from about to 185 feet about mean sea level in the eastern portion of the project site to about 245 feet in the southwestern portion of the site. In general, the western portion of the project area slopes gently downwards from west to east, and the eastern portion of the project site slopes gently downwards from south to north. Dudley Ridge extends into the northwestern portion of the site and is a local topographic high in the immediate area. A levee and canal system is present in the eastern portion of the site.

2.3 Geologic History

The San Joaquin Valley is an asymmetric structural trough in California's Central Valley. It features a broad, gently sloping east flank and a narrow western flank. The trough contains up to about 5 ½ miles of upper Mesozoic and Cenozoic age sediments overlying crystalline bedrock of the Sierran Block (Bartow, 1991).

The valley originated in the Mesozoic as a forearc basin in a convergent-margin setting. It evolved during the Cenozoic into the hybrid intermontane basin that exists today in a transform-margin setting. At the beginning of the Cenozoic, the area was occupied by a marine shelf and slope system at the western edge of North America. In the Paleocene, right slip along the proto-San Andreas fault and uplift of the Stockton arch began to influence sedimentation patterns within the basin. The northern and eastern portions of the basin were occupied by a marine shelf, while deeper marine slope and basinal facies were deposited in the southwestern portion. The uplifted Stockton arch formed a peninsula between the Sacramento basin to the north and the San Joaquin basin, likely creating a shallow or nearshore environment (Bartow, 1991).

By the end of the upper Paleocene-lower Eocene, a major sea level lowering restricted the marine basin, possibly leading to the formation of large delta systems near the basin margins. The middle Eocene saw maximum sea levels, with much of the present-day valley covered by deep marine waters. A large deep-sea fan developed in the southern portion, while the eastern valley likely consisted of fluvial and deltaic environments (Bartow, 1991).

While the northern Great Valley rose above sea level due to tectonic forces, the southern portion remained a marine environment with deep-water sedimentation through the Oligocene into the Miocene. In the early to middle Miocene, uplift in the southern basin led to the formation of alluvial fan and fan-delta depositional environments. By the late Miocene, northward movement of isolated high areas began to

close the western edge of the San Joaquin basin, removing the deep-marine outlet to the Pacific Ocean (Bartow, 1991).

Tectonic activity intensified during the Pliocene, leading to the closing of the basin by the late Pliocene. This resulted in the deposition of non-marine sediments from all sides. By the start of the Pleistocene, the San Joaquin basin was no longer a shallow sea. However, Corcoran Lake occupied nearly the whole valley, surrounded by the Sierra Nevada Mountains to the east and the Coast Ranges to the west (Bartow, 1991).

As the climate warmed and dried following the last glacial period, Corcoran Lake eventually dried up. However, its legacy remained in the form of Tulare Lake, which persisted into historical times. Tulare Lake was a large freshwater lake in the southern San Joaquin Valley, fed primarily by runoff from the Sierra Nevada Mountains. It was the largest freshwater lake west of the Mississippi River and the second-largest freshwater lake entirely in the United States (Preston, 1981).

Beginning in the 19th century, agricultural development in the San Joaquin Valley led to the diversion of the lake's tributary rivers for irrigation. This, combined with land reclamation projects, caused Tulare Lake to shrink dramatically. By the early 20th century, the lake had largely disappeared, transforming into fertile agricultural land. Today, the former lakebed only fills during years of exceptional rainfall, serving as a reminder of the region's dynamic geological and hydrological history (Preston, 1981).

2.4 Site Geology

The geology of the project area is dominated by Quaternary-age lacustrine, eolian, and alluvial deposits. Figure 4a is a geologic map presenting the Quaternary-age geology of the project site and surrounding area. A description of the primary geologic units within the project boundary is presented below (Smith, 1964):

- Recent Dune Sand, Qs (Recent): Windblown sand, including dune sand; mapped in northwestern portion of project site and in band running roughly north-south in western portion of project site.
- Recent Basin Deposits in the Great Valley, Qb (Recent): Sediments deposited during flood stages of major streams in the area between natural stream levees and fans; mapped in western and southwestern portions of project site.
- Quaternary Lake Deposits, Ql (Pleistocene-age to Recent): Clay, silt, and fine sand of lake beds in the San Joaquin Valley; mapped throughout majority of central and eastern portions of project site.
- Pleistocene Nonmarine Sedimentary Deposits, Qc (Pleistocene-age): Older alluvium consisting mainly of slightly consolidated and dissected fan deposits but may locally include Quaternary-age terrace deposits; mapped in western portion of project site.

Pliocene-age sedimentary deposits are mapped to the west of project site in the Kettleman Hills area. In the area of the Kettleman Hills, the upper Pliocene-age marine sedimentary rocks (denoted as symbol Pu on Figure 4a) consist of silty sandstone, silt, clay, blue sandstone, and conglomerate. The middle and/or lower Pliocene marine sedimentary rock unit (denoted as symbol Pml on Figure 4a) consist primarily of coarse-grained sandstone including blue sandstone, conglomerate, silty sand, and siltstone in the Kettleman Hills area (Smith, 1964).

Mapped unit contacts and faults are shown on Figure 4a, and Quaternary-age faults in the project region are shown on Figure 4b. Several faults in the Kettleman Hills shown on Figure 4a are not listed in the United States Geological Survey (USGS) Quaternary fault and fold database, which contains information on faults and associated folds in the United States that demonstrate geological evidence of coseismic surface deformation in large earthquakes in the past 1.6 million years (USGS, 2024a). These faults are listed as pre-Quaternary age in the Fault Activity Map of California prepared by the California Department of Natural Resources (California Department of Conservation, 2024a). Additional information regarding seismicity and faults in the project area is presented in Section 2.5.3.

Figures 5 through 7 present parent materials and classifications for the surficial soil based on mapping completed by the United States Department of Agriculture (USDA). Figure 5 presents the surficial soil parent materials, and indicates the surficial soils are primary alluvium and eolian deposits. Figure 6 shows the USDA textural classifications for the surficial soils in the general project area. The classifications include loamy sand, loamy fine sand, sandy loam, fine sandy loam, loam, clay loam, silty clay, and clay. Figure 6 shows the mapped surficial soil classifications according to the Unified Soil Classification System (USCS), and indicates primarily fat clay (CH), lean clay (CL), silty clay (CL-ML), and silty sand (SM).

Based on mapping completed by the USGS, the depth to crystalline basement rock in the project area is estimated to be approximately 20,000 to 29,500 feet (California Department of Natural Resources, 2024).

2.5 Geologic Hazards

The sections below provide a summary of geologic and geotechnical hazards for the site. Of the most common potential geologic hazards (seismicity, liquefaction, slope stability, flooding, soil collapse, shrink-swell, and karst), some are present at the project site. Additional information for select geologic and geotechnical hazards is provided the following sections. Design considerations for selected risks are provided in Sections 6 and 7.

2.5.1 Shallow Groundwater

A review of publicly available water well data indicates that shallow groundwater (less than 15 feet below ground surface) is present in the general project area. In most cases, static groundwater levels noted on publicly available water well logs ranged from approximately 10 to 50 feet below ground surface, though static water levels shallower than 5 feet were observed within several wells in the project area (California Department of Natural Resources, 2024). A discussion of groundwater conditions observed during the preliminary geotechnical investigation is presented in Section 4.2.

2.5.2 Flooding and Scour

Poorly draining soils are common throughout the project site and isolated flooding may occur following heavy or prolonged precipitation events in flatter areas where adequate surface drainage is not present. Federal Emergency Management Agency (FEMA) flood hazard mapping is presented in Figure 8. The eastern and central portions of the site, as well as areas in the western portion of the site, are mapped within Flood Zone A, defined as having a 1 percent annual chance of being equaled or exceeded in a given year, also known as the 100-year flood. The remainder of the site is mapped as Flood Zone X, defined as having a minimal flood hazard.

The project site is located in an alluvial basin, which is generally an accumulative environment. However, scour may occur along washes and flow paths during significant precipitation events. A site-specific hydrology study is recommended to understand the implications of scour.

2.5.3 Seismicity, Faults, and Liquefaction

2.5.3.1 Seismicity

The project area is located in an area of moderate seismic activity. Numerous earthquakes with magnitudes in the range of 2.5 to 4.0 and several earthquakes with magnitudes greater than 4.0 have epicenters identified within 10 miles of the projects site in the past 50 years (USGS, 2024b). The seismicity of the project site, with respect to ground motion parameters and stability of the proposed structures, is addressed in Section 6.8.

2.5.3.2 Faults

The United States Geological Survey (USGS) Earthquake Hazards Program database was reviewed to determine the extent of fault activity in the area. The primary faults/structures in the project region (within approximately 30 miles of the project site) with noted Quaternary activity are classified as Class A. Class A faults are defined as the following (USGS, 2024a):

> *Class A: Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether the fault is exposed by mapping or inferred from liquefaction or other deformational features.*

Quaternary-age faults within the general project area are shown in Figure 4b, and a summary of the regional Quaternary faults is presented in Table 1.

There are no mapped Quaternary-age faults within the project boundary. The nearest Quaternary-age fault to the project site, located approximately 8 miles to the northwest of the project site, is the Great Valley thrust fault system. The Great Valley thrust fault system can be described as a complex system of east-vergent, shallow-dipping blind thrust faults and associated west-vergent shallow to moderately dipping backthrust faults. Significant seismic events, ranging from magnitude 5.5 to 6.5, are noted to have occurred in 1982, 1982, and 1985. The estimate dip-slip rate is approximately 3.0 mm/yr (USGS, 2024a).

The Poso Creek fault, located approximately 18 miles to the southeast of the project site, has limited information available. The most recent prehistoric deformation is noted to be undifferentiated Quaternary (less than 1.6 million years), and the slip rate is unspecified (USGS, 2024a).

The Cholame-Carrizo section of San Andreas fault is located about 25 miles west-southwest of the project site. The San Andreas fault zone is the primary component of a network of dextral strike-slip faults that constitute the San Andreas fault system that collectively accommodates the majority of relative north-south motion between the Pacific and North American plates. The Cholame-Carrizo section consists of about 125 miles of the San Andreas fault system, which has a total length of roughly 675 miles. The movement is right lateral, and the slip rate is greater than 5 mm/year. The estimated reoccurrence interval is 100 to 450 years, with the Fort Tejon earthquake of 1857 noted to be the most significant earthquake in historic times (USGS, 2024a). The magnitude 7.9 Fort Tejon earthquake is estimated to have occurred about 25 miles west-southwest of the site and is estimated to have a surface rupture of about 225 miles, from near Parkfield, California, to near Wrightwood, California (USGS,2024c).

The San Jaun fault zone, located approximately 30 miles to the west of the project site, has limited information available. The most recent prehistoric deformation is noted to be latest Quaternary (less than 15 thousand years), and the slip rate is unspecified (USGS, 2024a).

2.5.3.3 Liquefaction

Liquefaction is defined as “the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore water pressures” (Youd, 1973). As seismic waves pass through saturated granular sediments, pore water pressures build and effective stresses decrease, thus reducing the shear strength. If the pore water pressure increases to a point that it equals the total stress of the overlying soils, thus resulting in an effective stress of zero, the sediment behaves as a fluid. This can cause ground failure, lateral spreading, differential settlement, landslides, ground cracking, and ground warping.

Grain-to-grain bonds and the relative density of granular materials are the primary physical controls on liquefaction susceptibility. Very loose to medium dense sands with a low fines content are at higher risk. In general, younger deposits are at higher risk than older deposits, as older deposits tend to develop grain-to-grain bonds (Obermeier, 1988).

The risk for liquefaction-induced bearing failure, excessive settlement, and/or lateral spreading is present at this site due to the presence of relatively young, loose to medium dense granular soils and the presence of shallow groundwater. A preliminary evaluation of the liquefaction susceptibility at each investigation location is discussed in Section 5. The risk of liquefaction at the site should be further evaluated during later phases of design.

2.5.4 Subsidence

There are several mechanisms that can lead to ground subsidence: dissolution of rock beneath the site (Section 2.5.6), underground mining, withdrawal of fluids such as oil, gas, and water, and soft soils with high consolidation potential. Some mechanisms can lead to sudden, catastrophic subsidence, while some are long term and generally not threatening to development.

Subsidence related to groundwater withdrawal in the San Joaquin Valley began in the mid-1920s and accelerated in the 1940s due to an increase in groundwater pumping to support agriculture. Subsidence rates in the valley eventually exceeded 1 foot per year in some areas (USGS, 2024d). Between 1926 and 1981, subsidence reached up to about 28 feet in portions of the valley. The subsidence is noted to have caused damage to structures such as aqueducts, roads, bridges, and well casings. The importation of surface water for irrigation in the 1950s through the Delta-Mendota Canal and in the 1970s through the California Aqueduct led to decreased groundwater pumping, rising groundwater levels, and a reduction in subsidence. However, drought conditions and regulatory reductions have led to increased pumping, lowering of groundwater levels, and additional subsidence (Sneed et al., 2013).

The California Department of Natural Resources has estimated that between 0 and 5 feet of vertical displacement occurred in the general project area between 1949 and 2005. This estimate was made by comparing elevation contours from 1950s quadrangle maps to a digital elevation model (DEM) from a 2005 NextMap Synthetic-aperture radar (SAR) acquisition. A Continuous Global Positioning System (CGPS) that recorded vertical displacement between 2006 and 2021 is located about 4 miles northwest of the site. The data indicates minimal subsidence over this time period. Vertical displacement estimates derived from Interferometric Synthetic Aperture Radar (InSAR) data in the project area from 2015 to 2024 indicate a slight upward movement of the ground surface during this time period (California Department of Natural Resources, 2024). Based on this information, the risk of significant subsidence for the project area is generally considered low. Future extractions of groundwater or changes in geologic conditions cannot be predicted and were not considered for this assessment.

The oil and gas wells within the project boundary and surrounding area are generally listed as plugged, dry, canceled, or idle (California Department of Conservation, 2024b). Therefore, the risk of subsidence due to oil and gas withdrawal is considered low at the project site. Future extractions of oil and gas or changes in geologic conditions cannot be predicted and were not considered for this assessment.

There are no documented active, inactive, or abandoned underground mines present within the project area based on a review of the USGS Mineral Resources Data System and information from the California Department of Conservation (USGS, 2024e; California Department of Conservation, 2024c). Therefore, based on the available data, the risk of subsidence due to oil and gas withdrawal and underground mining is considered low for the project site.

2.5.5 Slope Stability and Landslides

Topography at the project site is presented in Figure 3. A majority of the site has low topographic relief, though moderate slopes are present along Dudley Ridge in the northwestern portion of the project site as well as in the immediate vicinity of drainage paths, canals, and levees present throughout the project site. There are no mapped landslide deposits within the project site (California Department of Conservation, 2024d). A review of the California Geological Survey publication *Susceptibility to Deep-Seated Landslides in California* indicates isolated areas in the northwestern portion of the project site along Dudley Ridge are mapped as Landslide Susceptibility Class VII (moderate susceptibility) due to the presence of unconsolidated surficial units and slopes ranging from 15 to 20 percent (Figure 9; Wills et al., 2011). In general, the risk of slope instability is considered low for a majority of the project site, however, the risk of slope instability in isolated areas of the project site with moderate to steep slopes should be further evaluated during later phases of design.

2.5.6 Karst

Karst can be defined as a terrain, generally underlain by carbonate or evaporite rock, in which the topography is primarily formed by the dissolving of rock, and which is commonly characterized by karren (a distinct type of channel caused by solution on bare rock surfaces), closed depressions, subterranean drainage, and caves (Monroe, 1970). Caves and related solution features pose a risk to structures.

The United States Geological Survey (USGS) has completed mapping that delineates areas of the United States that have karst or the potential for development of karst and pseudokarst. The results of this mapping do not indicate the presence of carbonate or evaporite rocks within the project boundary (Weary et al., 2014). Based on available information and an understanding of the site geology, the risk of karst is considered low.

2.5.7 Soil Collapse

Collapsible soils are defined as unsaturated soils that undergo a drastic rearrangement of particles and subsequent decrease in volume upon wetting, additional loading, or a combination of the two. Alluvial and eolian deposits, such as those present in portions of the project site, may be prone to collapse due to their deposition, structure, and characteristics. In arid climates, it is common for flash floods to mobilize coarse unsorted sediments in debris flows, relatively low moisture-high solids mixes that move very quickly due to the slope of the terrain. These debris flows come to rest and dry out quickly. If there is a binder of plastic clays, these debris flow deposits can retain a soil matrix that has the potential to be collapsible.

Further discussion of the risk of soil collapse is presented in Sections 6.6 and 7.10.

2.5.8 Soil Shrink-Swell

Lean clay and fat clay are mapped throughout much of the project site (Figure 7). Moderate to high plasticity clays may swell when wetted, which can cause damage to shallow foundations and upward heave of shallow driven pile foundations. Clays at the site are described as having low to high shrink-swell potential (USDA, 2024). Further discussion of the risk of soil shrink-swell is presented in Sections 6.5 and 7.9.

2.5.9 Wind Erosion

The USDA soil mapping indicates that the surficial soils generally have moderate to high susceptibility to wind erosion (USDA, 2024). Long-term erosion mitigation typically consists of revegetation for areas of the project site where it can be performed following construction. Revegetation will help control erosion. In areas where revegetation is not possible, some erosion should be anticipated.

2.5.10 Soil Corrosion

Corrosion of buried concrete and metals can be a significant geologic hazard if soluble sulfate salts or soluble chloride salts are present at high levels. Soluble sulfate and chloride salts are most prevalent and of concern in clayey soils although some granular soils can also be of concern. Results of analytical testing are presented in Section 4.5 and the results of field electrical resistivity testing are presented in Section 4.6. A discussion of the soil corrosivity at the site is presented in Sections 6.11 and 7.12.

3 Geotechnical Investigation Methods

The preliminary geotechnical investigation consisted of cone penetration test (CPT) soundings, geophysical testing, test pit excavations, and soil sampling. Laboratory testing was conducted on selected soil samples from the test pits. Electrical resistivity testing and laboratory thermal resistivity testing was also performed in support of the project electrical design (to be completed by others).

Figures 10 through 15 show the plan location of all CPT soundings, test pits, analytical testing, and resistivity testing, which was conducted in September of 2024.

Prior to the start of field work, the geotechnical investigation locations were proposed by Barr and either approved or revised by Terra-Gen. The test locations were staked in the field by Barr with a handheld GPS capable of accuracy on the order of 15 feet, which is considered acceptable by Barr for the purposes of this geotechnical investigation.

A summary of the geotechnical test locations is included in Appendix A.

3.1 Subsurface Exploration

3.1.1 CPT Soundings

Twenty-two CPT soundings were completed during the preliminary geotechnical investigation (Figure 10). Twenty-one CPT soundings were performed within the proposed solar array areas, each to a target depth of 20 feet below existing grade. One CPT sounding was performed within the proposed substation area to a target depth of 100 feet below existing grade. Refusal was not encountered prior to reaching the target depth of the soundings.

All CPT work was performed in accordance with ASTM D5778, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils”. CPT sounding logs can be found in Appendix B.

The CPT soundings were conducted by ConeTec, Inc., of Richmond, California (ConeTec). The CPT testing was performed with a 30-ton track mounted rig with an enclosed workspace. For the CPT test, a cylindrical cone is pushed vertically into the ground at a constant rate of penetration of 20 mm/sec. During penetration, measurements are made of the cone tip resistance (q_c), the side friction on the cylindrical shaft (f_s) just above the tip, and dynamic pore water pressure generated by cone penetration (u_2).

The cones used in the investigation have a 15 cm^2 base area and a 60-degree apex angle. The sleeve area of the cones is 225 cm^2. The fluid used for saturation of the filter was glycerin and/or silicone. ConeTec provided Barr with complete records of tip resistance, sleeve friction, pore pressure, and friction ratio of all CPT soundings. These records were analyzed by Barr and graphical variations of all readings with depth, including an interpreted soil classification were developed and can be found in Appendix B.

The cone was also equipped with a geophone that measured the arrival time of shear waves generated at the ground surface. Geophysical testing was performed at one location within the proposed solar array (GEO-16) and one location within the proposed substation area (SUB-Opt1-01). The shear waves were generated at the ground surface by the CPT rig. Arrival times were measured at 1-meter depth intervals (approximately every 3 feet) at the proposed substation location and 1.5-meter intervals (approximately

every 5 feet) at investigation location GEO-16 to determine the interval shear wave velocities. The results of geophysical testing can be found in Appendix C.

Pore pressure dissipation tests were performed at selected locations where the presence of groundwater was observed. Tests conducted until the water levels reached equilibrium were used to estimate the groundwater conditions at the site. The results of pore pressure dissipation tests are summarized in Table 2 with complete results found in Appendix D.

3.1.2 Test Pits

Test pits were completed at six investigation locations within the footprint of the proposed solar development area and one investigation location within the proposed substation area (Figure 11). Test pits were performed by Brown's Construction Service, Inc. of Bakersfield, California, using a rubber-tired Case 580N backhoe. The target depth of 10 feet below existing grade was reached at each test pit location.

Test pit logs are provided as Appendix E. Photos of the test pits are provided in Appendix F.

3.1.3 Bulk Soil Sampling

Bulk soil samples were collected at five investigation locations from a depth of 3 to 5 feet below ground surface in support of laboratory thermal resistivity testing (Figure 12). At each of these locations, grab samples were also collected at 1-foot intervals from a depth of 0.5 to 4 feet for moisture content testing.

Two bulk soil samples were collected from a depth of 0.5 to 1.5 feet below the ground surface in support of California Bearing Ratio (CBR) testing to evaluate the strength of the road subgrade (Figure 13).

3.2 Laboratory Testing

3.2.1 General Laboratory Testing

The following tests were performed by SET:

- Moisture content in accordance with ASTM D2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"
- Atterberg limits in accordance with ASTM D4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
- Sieve and hydrometer in accordance with ASTM D422, "Standard Test Method for Particle-Size Analysis of Soils"
- Standard Proctor density in accordance with ASTM D698, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))"
- California Bearing Ratio in accordance with ASTM D1883, "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils"

All general laboratory test results are provided in Appendix G.

3.2.2 Analytical Testing

Analytical testing was performed on samples collected at eight investigation locations (Figure 14). The samples selected for testing were typically collected in the upper 4 feet below existing grade. The following laboratory tests were performed on each sample:

- Soil pH in accordance with ASTM D4972 “Standard Test Method for pH of Soils”
- Soluble chloride and soluble sulfate of soils in accordance with EPA Method 9056 “Determination of Inorganic Anions by Ion Chromatography”
- Oxidation-reduction potential (ORP) in accordance with ASTM G200, “Standard Test Method for Measurement of Oxidation-Reduction Potential (ORP) of Soil”
- Miller Box Electrical Resistivity in accordance with ASTM G57, “Standard Test Method for Measurement of Soil Resistivity Using the Wenner Four-Electrode Method”

All laboratory analytical test results are provided in Appendix H.

3.3 Field Electrical Resistivity Testing

Field electrical resistivity testing was completed at investigation location GEO-20 within the proposed solar array area and investigation location SUB-Opt1-01 within the proposed substation area as part of the geotechnical investigation (Figure 15). The testing was performed in September of 2024.

Field electrical resistivity testing was completed by Barr personnel in accordance with ASTM G57 “Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method” (equivalent to IEEE Std. 81). At each of the locations, measurements were taken to determine average soil resistivity in two perpendicular arrays. Electrode “a”-spacings of 2, 5, 10, 20, 50, 100, and 200 feet were used at investigation location GEO-20 within the proposed solar array area. Electrode “a”-spacings of 0.5, 1, 1.5, 2, 3, 5, 7, 10, 15, 20, 30, 45, 80, 150, and 200 feet were used at investigation location SUB-Opt1-01 within the proposed substation area. Measurements were attempted at an electrode “a”-spacing of 300 feet at investigation location SUB-Opt1-01, however, an accurate resistivity measurement was unable to be obtained.

The equipment used to collect the data consisted of a resistivity meter, four metal electrodes, and connecting wire. A Mini-Res Ultra resistivity meter manufactured by L&R Instruments, Inc. was used to collect the data. The resistivity meter read in resistance (R) directly and did not require the conversion of electrical potential (V) and inductance (I) to calculate resistance (V/I in Ω). Before and after each array was completed, the resistivity meter was connected to a resistor of known resistance and the resulting values were compared to the known resistance value for quality assurance and quality control purposes. The meter was properly calibrated for all test locations and no instrument adjustments had to be made.

Co-linear arrays of four electrodes were placed in the ground for each measurement. In order to check the accuracy of the single resistivity array, a perpendicular array was set up at each test location for all electrode spacings. Electrical current was input to the ground through the two outer electrodes of the array. The voltage drop produced by the resulting electrical field was measured across the two inner electrodes. The “a” spacing was increased with each measurement, expanding the array about a common center. Increasing the electrode separation increases the depth of investigation and indicates vertical variation in resistivity.

The soil electrical resistivity results are discussed in Section 4.6 and included as Appendix I.

3.4 Thermal Resistivity Testing

Bulk soil samples were collected at five investigation locations by Barr personnel in support of laboratory thermal resistivity testing (Figure 12). Sample locations included four within the proposed solar development and one within the proposed substation area. The samples were obtained from test pits at a depth of approximately 3 to 5 feet below the surface. The samples were transported to SET for laboratory testing. Thermal resistivity tests were performed in accordance with ASTM D5334, "Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure". Laboratory tests included measurement of the soil's moisture content, maximum dry density, and thermal dryout characteristics (a function of moisture content). SET performed the thermal resistivity tests in October of 2024.

The bulk sample specimens collected within the proposed solar array area were re-compacted to approximately 85 percent of standard Proctor dry density near the as-received moisture content. The bulk sample collected within the proposed substation footprint was re-compacted to approximately 85 percent of standard Proctor dry density near the as-received moisture content and 95 percent of standard Proctor dry density at the lowest moisture content necessary to reach the target compaction. The soil specimens were compacted in four layers in test molds (75 mm in diameter by 150 mm high) to minimize contact resistance at the soil/probe interface and to provide a uniform density.

The results of the soil thermal resistivity testing are discussed in Section 4.7 and included in Appendix J.

3.5 Field Reconnaissance

In addition to a review of available maps and published data, Barr staff performed a limited site reconnaissance of the project site area for potential geotechnical and geological concerns while on site in September of 2024. This work was performed at or near the accessible preliminary phase geotechnical investigation locations. The site reconnaissance was focused primarily on surficial features such as indications of slope instability, the presence of cobbles, boulders, or bedrock outcrops, and low areas or drainage channels that may have the potential to flood or scour.

Site vegetation through much of the site generally consists of tall grass and sparse bushes. Thicker vegetation, consisting predominantly of bushes, was present in some areas of the site, primarily near existing levees, drainage paths, and topographic depressions. The surficial soils at the site generally consist of silty sand, clayey sand, sandy lean clay, or fat clay. Cobbles, boulders, and rock outcrops were not observed at the ground surface during the site reconnaissance.

A levee and canal system is present in the eastern portion of the site. Standing water was observed in some of the canals at the time of the investigation. Standing water was also observed in a low topographic depression approximately 750 to 800 feet west of investigation location GEO-08A.

Based on field observations and a desktop review of site slopes, no issues related to slope instability were identified. The site is generally flat, with low relief, moderate to steep slopes generally only present along existing levees and canals.

General site photographs taken during the geotechnical investigation are provided as Appendix K.

4 Results

Section 3 describes the field and laboratory investigation procedures. Section 4 presents the data from testing and investigation and provides further analysis of these results.

4.1 Subsurface Stratigraphy

The results of the CPT soundings (Appendix B through D), test pits (Appendix E and F), and laboratory tests (Appendix G) were compiled to obtain an understanding of the subsurface stratigraphy of the study areas.

4.1.1 Cone Penetration Test Data Interpretation

The following describes the procedures used to interpret the CPT data and the lithology inferred from the CPT soil behavior type.

The CPT data interpretation was performed using an in-house program designed by Barr specifically for use on renewable projects. The in-house program has been cross-checked with CPTINT version 5.2 for quality assurance and has been found to be compliant. The program uses the soil behavior type classification system from CPT data proposed by Robertson et al. (1990). The classification system is based on the corrected tip resistance, q_t, friction ratio, R_f, and pore-water pressure parameters, B_q; the classification includes a total of nine soil behavior types. These cone parameters are defined as follows:

$$q_t = q_c + (1 - a)u_2 \quad \text{(Lunne et al., 1997)}$$

$$R_f = \left({}^{f_s}/_{q_t}\right) 100 \quad \text{(Lunne et al., 1997)}$$

$$B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \quad \text{(Lunne et al., 1997)}$$

where:

q_c = tip resistance measured by the cone, load per area

a = the area ratio of the cone

u_2 = measured pore-water pressure during cone penetration, load per area

f_s = unit sleeve friction resistance, load per area

σ_{vo} = total overburden stress, load per area

u_o = in-situ pore water pressure, load per area

As presented in Robertson et al. (2010), materials classified as Soil Behavior Type (SBT) 5 are traditionally considered to have granular properties. During the review of the CPT analysis, the materials in many soundings at various depths were typically found to be SBT 5 material. However, based on a review of the stratigraphy encountered within the test pits, the CPT analysis was augmented such that cohesive strength properties were determined and assigned to materials classified as SBT 5. The results

of the CPT data interpretation are presented in Appendix B. A statistical summary of the CPT inferred soil behavior type (SBT) for soundings is provided in Table 3.

4.1.2 Typical Stratigraphy

The typical stratigraphy, as determined from field data, consists of a layer of topsoil overlying clay, silt, and sand layers of varying thicknesses. Based on the CPT-interpreted soil types, a majority of the materials were classified as silt mixtures, sand mixtures, and sands, with occasional layers of clay and gravelly sand. The results of the test pit excavations indicated varying subgrade conditions, including locations that encountered primarily cohesive soils and locations that encountered primary granular soils within the upper 10 feet below existing grade. Sections 4.1.2.1 through 4.1.2.4 present additional details about the primary material types encountered during the investigation.

4.1.2.1 Topsoil

Based on the results of the geotechnical investigation, the typical thickness of the topsoil at the project site was found to range from approximately 6 to 12 inches. The topsoil across the site generally consisted of silty sand, clayey sand, sandy lean clay, or fat clay, and was commonly tan, brown, or gray in color. Localized zones of thicker topsoil may be present, particularly in and adjacent to drainage swales, ephemeral ponds, and ditches.

4.1.2.2 Clays

Clays were encountered within six of the seven test pits completed for the investigation. At investigation location GEO-05, sandy lean clay was encountered underlying the topsoil and extending to the termination depth of 10 feet below existing grade. At the remaining test pit locations where clays were encountered, the stratigraphy within the upper 10 feet below existing grade consisted of clays, sands, and/or silts, and the clay layers ranged in thickness from about 1 to 7 feet. Based on visual observations and the results of laboratory testing, the clays were classified as fat clay, lean clay, or sandy lean clay with varying amounts of sand and trace amounts of gravel in some instances. The clays were commonly observed to be tan, brown, or gray in color. The CPT-interpreted soil behavior type for the clays was often noted as clays, silt mixtures, and sand mixtures, with isolated layers interpreted as very stiff fine-grained material on the CPT logs (Appendix B; Table 3).

4.1.2.3 Silt

Sandy silt was encountered within test pit GEO-16 from a depth of 2 feet below existing grade to the termination depth of the test pit at approximately 10 feet below existing grade. The sandy silt was tan to gray in color. The CPT-interpreted soil behavior types over this depth interval within sounding GEO-16 were sand and sand mixtures.

4.1.2.4 Sand

Sands were encountered within five of the seven test pits completed for the investigation. At investigation location GEO-10, layers of clayey sand and silty sand were encountered underlying the topsoil and extending to the termination depth of 10 feet below existing grade. At the remaining test pit locations where sands were encountered, the stratigraphy within the upper 10 feet below existing grade consisted of both sands and clays, and the sand layers ranged in thickness from about 2 to 6 feet. Based on visual observations and the results of laboratory testing, the sands were classified as clayey sand or silty sand with varying fines contents. The sands were commonly observed to be tan, brown, or gray in color. The

CPT-interpreted soil behavior type for the sands was often noted as sands, with isolated layers interpreted as gravelly sand on the CPT logs (Appendix B; Table 3).

4.2 Groundwater Conditions

Groundwater was observed at select investigation locations over the course of geotechnical field investigation. As a result, groundwater may be a factor in the design and construction of foundations.

Pore pressure dissipation tests were performed in selected layers during the CPT investigation. Pore pressure dissipation tests were performed by stopping the CPT test at a specific depth and monitoring the pore water pressure reading as it varied with time. Tests performed to a point at which equilibrium pore pressure was reached were used to estimate groundwater levels. The results of the pore pressure dissipation tests indicated groundwater levels ranging from 4.2 to 22.4 feet below ground surface. Table 2 includes a summary of the groundwater levels calculated from the pore pressure dissipation tests that reached equilibrium. Results of the pore pressure dissipation testing are included as Appendix D.

Groundwater levels observed during and immediately after excavation of the test pits ranged from approximately 5 to 10 feet below the ground surface (where encountered). A summary of the groundwater levels observed within the test pit excavations is presented in Table 2. Due to the presence of clay, groundwater observations in the test pits may not provide a reliable estimate of the groundwater table.

Many factors contribute to water level fluctuations, such as heavy rainfall events, dry periods, etc., and the measurements noted during this investigation may not represent the long-term groundwater levels present at the site.

4.3 General Soil Laboratory Testing

All samples submitted for laboratory testing were sealed immediately after collection to preserve moisture. Grab samples collected within the test pit excavations were placed in one-quart plastic bags with a double seal. These samples were transported from the project site to SET in 5-gallon buckets.

All general laboratory test results performed on grab samples from the test pit excavations are summarized in Table 4a and the laboratory test results from bulk samples (thermal resistivity and CBR samples) are summarized in Table 4b. All general laboratory test results from the geotechnical investigations are included in Appendix G.

4.3.1 Moisture Content

Moisture content tests were conducted on 28 grab samples (Table 4a) and seven bulk samples (Table 4b) collected from the test pits during the geotechnical investigation. The samples tested included fat clay, lean clay, sandy lean clay, sandy silt, clayey sand, silty sand, silty clayey sand, and poorly graded sand with silt.

The results of 22 tests performed on cohesive soil samples (clays) indicate the moisture contents ranging from approximately 4 to 25 percent, with an average of 14 percent, indicating that the clays were generally in a moist condition.

The results of 13 tests performed on granular soil samples (silts and sands) indicate the granular soils had moisture contents ranging from 3 to 24 percent, with an average moisture content of 12 percent, indicating that the granular soils were generally in a moist condition.

4.3.2 Atterberg Limits

Atterberg limits testing was performed on three grab samples from the test pits and the results are summarized in Table 4a. An additional seven Atterberg limits tests were performed on bulk soil samples collected in the upper 5 feet below existing grade in support of thermal resistivity and CBR testing. These results are summarized in Table 4b.

Results of testing performed on eight cohesive soil (clay) samples indicated liquid limits ranging from 29 to 54 percent, plastic limits ranging from 14 to 21 percent, and plasticity indices ranging from 12 to 37 percent. These results classify the materials tested as fat clay (CH), lean clay (CL), and sandy lean clay (CL).

Atterberg limits testing was performed on two granular (sand) samples collected at investigation locations GEO-10 and GEO-20 in support of thermal resistivity testing. The sample collected at investigation location GEO-10 had a liquid limit of 21 percent and was determined to be nonplastic and, therefore, the plastic limit could not be determined. The sample collected at investigation location GEO-20 had a liquid limit of 37 percent, a plastic limit of 14 percent, and a resulting plasticity index of 23 percent. Based on the results of the Atterberg limits testing and grain size analyses performed on these samples, the samples were classified as silty sand (SM) and clayey sand (SC).

4.3.3 Grain Size Analysis

Grain size analyses (hydrometer and/or sieve analyses) were performed on seven grab samples selected from the test pits and the results are summarized in Table 4a. An additional seven sieve analyses were performed on bulk soil samples collected in support of thermal resistivity and CBR testing, and the results are summarized in Table 4b.

Based on the results of the 14 grain size analyses, the samples tested were classified fat clay (CH), lean clay (CL), sandy lean clay (CL), sandy silt (ML), clayey sand (SC), and silty sand (SM). The percent fines (percent by weight passing the number 200 sieve) of the samples tested ranged from approximately 26 to 82 percent, while the percent gravel (percent by weight retained on the number 4 sieve) ranged from 0 to 4 percent. The results of three hydrometer tests indicated the clay-size fraction (defined as the material smaller than 2 μm) of the samples tested ranged from 30 to 45 percent.

4.3.4 Standard Proctor Testing

Seven standard Proctor tests were conducted on bulk soil samples collected at shallow depths. Results of the standard Proctor density testing indicated the native soil's maximum dry density ranges from approximately 99 to 115 pcf. The corresponding optimum moisture content ranged from approximately 14 to 24 percent.

Standard Proctor density test results are summarized in Table 5 and included in Appendix G.

4.4 Shear Strength

As noted in Section 4.2, the project site consists of both cohesive and granular soils. The following sections discuss, in detail, the shear strength of the soils tested during this investigation.

4.4.1 Undrained Shear Strength

The undrained shear strength of the clays was estimated based on correlations from CPT testing and field pocket penetrometer measurements.

The undrained shear strength of the soil at various depths was calculated based on CPT data using the following equation:

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \quad \text{(Lunne et al, 1997)}$$

where:

s_u = undrained shear strength

N_{kt} = empirical cone factor

σ_{vo} = total in-situ vertical stress = varies (from CPT data)

q_t = corrected cone tip resistance = varies (from CPT data)

The empirical cone factor, N_{kt}, is used to estimate in-situ soil shear strength from CPT data, which is significantly more reliable than estimating strength from geotechnical borings through SPT results and more efficient than performing a large program of laboratory strength tests on undisturbed thin-wall samples. An N_{kt} value of 20 was selected for preliminary design considerations based on Barr's experience in the general project region.

Table 6 summarizes the average undrained shear strength calculated from CPT data for depth intervals of 0.5 to 10 feet below existing grade and 10 to 20 feet below existing grade, determined from CPT soundings with significant cohesive layers present (based on Barr's interpretation of CPT soil behavior type), for each CPT sounding location. The average undrained shear strength from a depth of 0.5 to 10 feet below existing grade varies from approximately 1,160 to 4,000 psf. The average undrained shear strength from a depth of 10 to 20 feet below existing grade varies from approximately 840 to 2,840 psf. Appendix B illustrates the undrained shear strengths plotted versus depth for each CPT sounding.

Four pocket penetrometer tests were conducted at select depth intervals within the test pit excavations. The pocket penetrometer results indicate an estimated unconfined compressive strength ranging from 1.5 to 3.5 tons per square foot (tsf), which corresponds to an undrained shear strength of 0.75 to 1.75 tsf (1,500 to 3,500 psf).

4.4.2 Drained Shear Strength

The drained friction angle of granular soil was estimated through correlations with CPT data using the expression derived by Mayne (2006):

$$\varphi' = 17.6 + (11)\, log\left(\frac{q_c}{\sqrt{\sigma'_{vo}}}\right) \quad \text{(Mayne, 2006)}$$

where:

ϕ' = drained friction angle

q_c = cone tip resistance = varies (from CPT data)

σ'_{vo} = effective in-situ vertical stress = varies (from CPT data)

Table 7 summarizes the average friction angle calculated from CPT data for depth intervals of 0.5 to 10 feet below existing grade and 10 to 20 feet below existing grade, determined from CPT soundings with more than 25 percent granular soil present (based on Barr's interpretation of CPT soil behavior type), for each CPT sounding location. The average friction angle from a depth of 0.5 to 10 feet below existing grade varies from approximately 40 to 43 degrees. The average friction angle from a depth of 10 to 20 feet below existing grade varies from approximately 39 to 43 degrees. Note that several CPT sounding locations were determined to have less than 25 percent granular soil present (based on Barr's interpretation of CPT soil behavior type), and therefore, associated friction angles are not presented in Table 7. Appendix B illustrates the friction angles plotted versus depth for each of the CPT soundings.

The drained friction angle of the clay present across the project site was not directly measured. However, it is estimated at 27 degrees based on correlations to the plasticity index from Terzaghi et al. (1996).

4.5 Soil Analytical Testing

Results of the Miller box testing indicate that the electrical resistivity of the soils at the as-received moisture content ranges from 114 to 98,000 Ohm-centimeters (Ω-cm) and at the saturated condition ranges from 98 to 5,800 Ω-cm. The pH results indicate that the soils have a pH ranging from 7.5 to 9.0. The soluble chloride (dry weight) of the soil ranges from 7.1 to 3,590 mg/kg. The soluble sulfate (dry weight) ranges from 25.5 to 10,300 mg/kg. The results of the oxidation-reduction potential tests indicate that the soils ranged from 157 to 265 millivolts (mV).

Analytical test results are summarized in Table 8 and included in Appendix H.

4.6 Field Electrical Resistivity Testing

Field electrical resistivity testing was completed at two investigation locations as discussed in Section 3.3. Apparent resistivity (ρ_a) was calculated for each measurement and corresponding electrode spacing (a) using the resistance measurement (Ω) and the geometric factor (K) as follows:

$$\rho_a = K\left(V/I\right) \quad \text{where:} \quad K = 2\pi a$$

The apparent resistivity measurements for test performed within the proposed solar array area (GEO-20) ranged from approximately 269 to 740 Ω-cm, with an average of 435 Ω-cm.

The apparent resistivity measurements for the test performed within the proposed substation area (SUB-Opt1-01) ranged from approximately 185 to 27,776 Ω-cm, with an average of 3,658 Ω-cm.

The results of the field electrical resistivity testing are included in Appendix I.

4.7 Thermal Resistivity Testing

Laboratory thermal resistivity testing was completed on samples collected at five investigation locations as discussed in Section 3.4. Laboratory thermal resistivity results are summarized in Table 9 and included in Appendix J.

The thermal resistivity of samples re-compacted to a density equivalent to approximately 85 percent of the standard Proctor density near their as-received moisture contents ranges from 85 to 120°C-cm/W at the as-received moisture content and from 245 to 389°C-cm/W in the fully dried condition. The thermal resistivity of the sample collected at substation investigation location SUB-Opt1-01 re-compacted to a density equivalent to approximately 95 percent of the standard Proctor density at the lowest moisture content necessary to reach the target compaction is 120°C-cm/W near the as-received moisture content and 209°C-cm/W in the fully dried condition.

Soil temperature measurements were obtained at 1-foot increments from the ground surface to a depth of 5 feet within each test pit. The results of the temperature measurements are included in Table 10. A soil sample was also taken from the location and depth of each temperature measurement in the upper 4 feet to determine the corresponding moisture content profile and these results are included in Table 11.

4.8 Geophysical Test Results

The results of the geophysical testing (shear wave velocity testing) are included in Appendix C and summarized in Table 12. The interval wave velocities (V_s) were used to compute the average velocities from the ground surface to the termination depth of the soundings. Based on the results of the shear wave testing and analysis, the weighted average interval shear wave velocities (V_s) varied from approximately 565 to 817 ft/sec over the depth intervals tested.

The shear wave velocity testing within CPT sounding SUB-Opt1-01 was performed to a depth of approximately 100 feet below existing grade and was used to evaluate the average shear wave velocity, $\overline{v}_s$, according to ASCE 7-16, Chapter 20.4 using the following equation (ASCE, 2017):

$$\overline{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

d_i = the thickness of any layer between 0 and 100 feet

v_{si} = the shear wave velocity in ft/s

$\sum_{i=1}^{n} d_i$ = 100 feet

Using this formula, the average shear wave velocity, $\overline{v}_s$, is 787 ft/sec. The seismicity of the project site, with respect to ground motion parameters and stability of the proposed structures, is addressed in Section 6.8.

4.9 California Bearing Ratio Testing

The CBR samples were prepared to relative compaction levels of 95 percent of standard Proctor maximum density near the optimum moisture content in the laboratory prior to testing. Results of the CBR tests indicate that the CBR values under a surcharge load of 50 psf ranged from 5.4 to 7.6 percent. Note that these are “soaked” CBR values. Results of the CBR testing are summarized in Table 13 and included in Appendix G.

5 Preliminary Liquefaction Susceptibility Assessment

The project site is located in an active seismic area in relatively close proximity to the San Andreas other significant faults. Seismic ground motions have the potential to cause a sudden drop of shear strength under undrained conditions from the yield strength to the substantially lower liquefied strength, known as liquefaction. Liquefaction can be triggered by the dynamic application of a single large increment of shear stress or by repeated application of smaller shear stress increments from seismic sources associated with earthquakes or explosives.

The geotechnical data collected as part of this investigation was used to complete a preliminary liquefaction susceptibility assessment. Based on guidance provided by Youd and Idriss (Youd et al., 2001), the simplified procedure for evaluating liquefaction susceptibility is performed by calculating two parameters: (1) cyclic stress ratio (CSR), or the seismic demand on the soil, and (2) cyclic resistance ratio (CRR), or the soil's resistance to liquefaction.

The CSR can be calculated using the following equation developed by Seed and Idriss (1971):

$$CSR = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where:

a_{max} = peak horizontal acceleration at the ground surface (see commentary below, assumed to be equivalent to the PGA for the underlying "bedrock" at the site for this preliminary analysis)
g = acceleration due to gravity
r_d = stress reduction coefficient
σ_{vo} = initial vertical total stress
σ'_{vo} = initial vertical effective stress

The presence of liquefiable soils results in a seismic Site Class F, which requires a site response analysis be conducted to determine geotechnical and structural design criteria per ASCE 7-16 (ASCE, 2017). A site response analysis estimates amplification or de-amplification of seismic waves as they propagate upward through the soil profile. A site response analysis was not included in the scope of this preliminary geotechnical investigation and will need to be performed during later phases of design. Therefore, the ASCE hazard tool website (ASCE, 2024) was used to determine seismic parameters for the underlying "bedrock" (assumed Site Class B, or a shear wave velocity of about 2,500 ft/s). The PGA value for the underlying "bedrock" at the site was determined to be 0.488g in the western portion of the project site near the proposed substation location.

Based on Barr's previous experience, the results of a site response analysis may indicate both amplification and de-amplification of the ground motion within foundation soils depending on the soil parameters (shear wave velocity, density, stiffness reduction, damping, etc.), depth within the profile, and earthquake input record. Due to the varying results and lack of a site response analysis, a value of 0.488g, equal to the PGA value determined from the ASCE hazard tool website (ASCE, 2024) and associated with ASCE 7-16, was selected for use in the preliminary liquefaction assessment herein without any assumed amplification or de-amplification.

The CRR curve was approximated for a magnitude 7.5 earthquake using the following equation developed by Robertson and Wride (1998):

$CRR_{7.5} = 0.833*[(q_{c1N})_{cs}/1{,}000]+0.05$ for $(q_{c1N})_{cs} < 50$

and

$CRR_{7.5} = 93*[(q_{c1N})_{cs}/1{,}000]^3+0.08$ for $50 \leq (q_{c1N})_{cs} < 160$

where:

$(q_{c1N})_{cs}$ = clean-sand cone penetration resistance normalized to approximately 100 kPa

The $CRR_{7.5}$ curve determined using the formulas above applies only to magnitude 7.5 earthquakes and must be adjusted for the anticipated earthquake magnitude at the project site. A review of the earthquake deaggregation plot for the project area indicated earthquakes around magnitude 6.5 contribute the most to the seismic hazard (USGS, 2024f), and therefore, the preliminary liquefaction analysis presented herein is based on liquefaction susceptibility during a magnitude 6.5 seismic event. To scale the $CRR_{7.5}$ curve to a magnitude 6.5 event, the following equation was used (Youd et al., 2001):

$$MSF = {10^{2.24}}/{M_w^{2.56}}$$

where:

MSF = magnitude scaling factor

M_w = earthquake magnitude

The resulting factor of safety against liquefaction was determined using the following equation (Youd et al., 2001):

$$FS_L = \left({CRR_{7.5}}/{CSR}\right) MSF$$

where:

FS_L = factor of safety against liquefaction

$CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes

CSR = calculated cyclic stress ratio

MSF = magnitude scaling factor

Liquefaction can be expected to occur within a soil layer when the FS_L is less than 1.0. Based on the preliminary liquefaction susceptibility analysis, approximately 15 to 75 percent of the soils at individual investigation locations are prone to liquefaction over the depths of the CPT soundings when a magnitude 6.5 earthquake is applied. The results of the liquefaction susceptibility analysis based on the CPT data are provided in Appendix L.

6 Preliminary Geotechnical Recommendations for Solar Module and Inverter Foundation Design

Results of the field and laboratory investigation have been presented in Section 4. Based on these results, Section 6 provides analysis, conclusions, and recommendations for the preliminary design and construction of solar module and inverter foundations, as well as general construction considerations.

6.1 General Site Grading

The following sub-sections present general recommendations for site clearing, grading, and compaction for the project site.

6.1.1 Clearing and Grubbing

The project site consists of predominantly undeveloped and agricultural land and clearing and grubbing will generally consist of removal of brush, crop debris, grass, and topsoil. Based upon information collected during the field investigation, the topsoil thickness at the site is approximately 6 to 12 inches, though localized zones of thicker topsoil should be expected, particularly in and adjacent to perennially flooded areas and ditches or swales.

Any topsoil or organic material should not be used for structural fill and should be placed separately to avoid contamination with other excavated soils. This material could be used in grading non-structural fill such as fields or service areas in which compressibility of the material does not have an impact on structures.

6.1.2 Site Excavations and Grading

Results of the preliminary geotechnical investigation indicate that cut and fill of the subsoil in the area can be achieved with conventional machinery.

The soils at the site can be classified as Type A, B, and C from OSHA soil classifications (29 CFR 1926 Subpart P-Excavations). It is the responsibility of the competent field personnel to verify the in-situ soil classification at each excavation and ensure that the benching or slopes are adequate during construction.

6.1.2.1 Grading in Fill

If large fill areas are required, all lifts should be placed as close to horizontal as possible, with lift thickness not to exceed 12 inches in a loose condition, and the material is placed at a minimum of 95 percent of the standard Proctor maximum dry density. Fills on the order of 3 feet or greater above existing grade should be evaluated with regards to site settlement and foundation design recommendations.

It is assumed that all foundations will bear on or in native soil, and the use of compacted fill for foundation support should not be required, except where excavation of unsuitable material is performed. Any fill below shallow foundations should consist of engineered fill in accordance with Section 6.1.4.

6.1.3 Subgrade Preparation for Shallow Foundations

Upon excavation to the proposed bearing surface, the excavation base should be inspected by the construction phase geotechnical engineer of record or authorized representative. If the excavation base surface is deemed unsuitable for construction in the opinion of the geotechnical engineer of record or authorized representative, the conditions shall be noted and communicated to the foundation and geotechnical engineers for appropriate foundation modifications.

If, in the course of excavating the foundation, the base of the excavation becomes rutted, damaged or is otherwise determined to be of inadequate character, the following actions should be performed:

- If the subgrade soils consist of silts or clays, the excavation should be subcut a minimum of 6 inches beyond the depth of the rutted soils and backfilled with engineered fill. Granular fill placed for subgrade stabilization below foundations should comply with Section 6.1.4.
- If the subgrade soils consist of primarily granular soils, the excavation base surface should be leveled and surface compacted. Compaction of this material is required to achieve at least 98 percent of the laboratory maximum dry density measured according to the standard Proctor method.

6.1.4 Engineered Fill

Any fill placed below foundations should consist of approved engineered fill. Engineered fill should consist of a well graded granular material, such as California Department of Transportation (CALTRANS), AS-Class 2 or 3 Aggregate Base Course material. Alternative engineered fill materials may be used depending upon availability and approval by the foundation engineer. Loose lifts should not exceed 9 inches. The engineered fill should be compacted to at least 98 percent of the standard Proctor maximum dry density.

The excavations and subsequent placement of engineered fill should be oversized by 1 foot on all sides for each foot of excavation below the foundation embedment depth. For example, a 1-foot excavation below the foundation depth will require a bottom of foundation excavation width (and length) 2 feet wider than a standard foundation excavation width (1 foot on each side of the footing). The base of every excavation should be inspected by a representative of the geotechnical engineer to evaluate the soil conditions at the base of the over excavation prior to fill placement.

6.1.5 Backfill for Shallow Foundations

The native site soils can be used as backfill for the foundation if it meets the following conditions:

- Material should be placed in loose lifts not exceeding 12 inches in thickness
- The material should be placed at a minimum of 95 percent of the standard Proctor dry density.
- The backfill surface is graded such that water is directed away from the foundation to prevent moisture infiltration (minimum slope of ¼ inch per foot).
- Density is checked in the field periodically to verify adequate compaction (typically one test per 2,500 square feet of fill and no less than one test for each lift).

On-site backfill may consist of native soil, although is it recommended the liquid limit of these materials should not exceed 45 and the plastic index should not exceed 30 when used in a non-structural application. Most of the on-site material is anticipated to meet this requirement based on laboratory testing completed. It is recommended that all material in consideration for use as fill be tested and compared with this recommendation prior to use. Fine-grained fill soils (silt or clay) may be particularly difficult to compact if wet or allowed to become wet, or if spread and compacted over wet or marginally stable subgrades. The backfill material should be debris-free and free of any organic matter.

6.2 Recommended Geotechnical Parameters for Preliminary Foundation Design

Axial and lateral parameters recommended for use in preliminary pile foundation design based on the results of the geotechnical investigation are summarized in Table 14 and Table 15, respectively, and discussed in the following sections.

The results of the geotechnical exploration indicate varying subsurface conditions throughout the project site. To account for this variability, Barr recommends three design profiles at the project site on a preliminary basis. The design profiles were determined primarily based on the CPT-interpreted soil behavior type at each investigation location. Design Profile 1 generally coincides with locations determined to have primarily CPT-interpreted cohesive soils in the upper 10 feet below existing grade, Design Profile 2 generally coincides with locations determined to have a combination of CPT-interpreted cohesive and granular soils in the upper 10 feet below existing grade, and Design Profile 3 generally coincides with locations determined to have primarily CPT-interpreted granular soils in the upper 10 feet below existing grade. The associated design profile for each investigation location is shown on Figure 16. Note that these locations are not grouped spatially, but rather were selected to represent the varying subgrade conditions in support of the preliminary pile analysis presented within this report.

6.2.1 Low Strength Layers

Based on the analyses, some lower density and lower strength soils are present at the site (lower than the preliminary Design Strength Profiles included in Table 14 and Table 15). While the analysis includes general consideration for the frequency of lower strength soils, natural variations in the subsurface soil profile, and the lower frequency of testing associated with the preliminary phase indicates it is likely that lower strength soils may be present in areas of the site.

6.2.2 Liquefaction

Based on the results of the preliminary liquefaction risk assessment presented in Section 5, some of the soils at the site are susceptible to liquefaction. The preliminary liquefaction risk assessment was based on available information and did not consider the impacts of amplification or de-amplification of seismic waves through the soils at the site.

The axial and lateral parameters recommended for use in preliminary pile foundation design presented in Tables 14 and 15, as well as the preliminary pile foundation design recommendations presented within this report, do not consider the risk of liquefaction. The design considerations related to liquefaction should be further evaluated during later phases of design.

6.3 Preliminary Pile Foundation Design

Solar modules are commonly supported on steel piles as they are typically the most economical and, thus, the most desirable foundation system. Similarly, steel pile foundations may be used for inverters and other lightweight equipment.

Other deep foundation options such as helical piers, drilled piers, and shallow spread footings have been used but are much less common. Barr presently assumes steel piles will be used for this project and should be contacted if alternate foundation systems are considered for foundation design.

6.3.1 Preliminary Design and Construction Considerations for Pile Foundations

The results of this preliminary geotechnical investigation indicate that driven pile foundations are a feasible foundation system for support of the solar modules and inverters. Special design and constructability considerations for driven pile systems include:

- Soils susceptible to liquefaction (Sections 5 and 6.2.2)
- Soil corrosivity (Section 6.11)
- Low density and/or low strength soils (Section 6.2.1)
- Uplift due to swelling soils (Section 6.5)
- Soil Collapse (Section 6.6)
- Pile driving (Section 6.3.1.1)
- Wind erosion and scour (Sections 2.5.2 and 2.5.9)

The preliminary geotechnical investigation was performed at selected locations spread across the project site. While the results of the preliminary geotechnical investigation are considered representative for the project area based on an understanding of the site geologic depositional history, the potential exists for low strength soils or otherwise undesirable soil conditions to be present in other areas of the project site.

6.3.1.1 Pile Installation Considerations

Of concern for pile installation is the presence of driving obstructions such as dense and/or cemented soils, gravels, cobbles, and boulders at the project site. Results of the geotechnical investigation generally indicate that pile driving appears feasible with standard pile driving equipment. Each of the 22 CPT soundings performed during the preliminary geotechnical investigation reached the target depths ranging from 20 to 100 feet below grade. In addition, each test pit reached the target depth of 10 feet below existing grade with a rubber-tired backhoe.

While the results of the preliminary geotechnical investigation are considered representative for the project area based on an understanding of the site geologic depositional history, the potential exists for hard pile driving conditions to be present in other areas of the project site. Suitability of pile driving and other installation methods (if applicable) should be further evaluated during the design phase geotechnical investigation.

6.3.2 Load Capacity – Pile Foundations

Assuming a pile foundation system is used for support of the solar modules, the pile will develop axial compression capacity through a combination of end bearing and skin friction, and uplift capacity from skin friction only. Lateral capacity should be evaluated by the structural designer with LPile or MFAD. The following sections describe preliminary recommendations for the design of pile foundation systems. The design values established in Tables 14 and 15 are recommended for use in preliminary design of pile foundation systems.

6.3.2.1 Axial Capacity

The axial capacity of pile foundation systems in tension is based on the skin friction that the soil provides. The axial capacity of pile foundation systems in compression is based on the contribution of the skin friction and the end bearing that the soil provides.

Calculation of the allowable skin friction was performed based on the results of the CPT soundings through the use of the following equation (Eslami and Fellenius Method, FHWA 2016):

$$f_{s(all)} = \frac{C_s q_E}{FS} \qquad \text{(FHWA, 2016)}$$

where:

$f_{s(all)}$ = allowable unit skin friction resistance

C_s = shaft correlation coefficient, based on CPT-interpreted soil type (FHWA, 2016 Table 7-11)

q_E = cone point resistance after correction for pore pressure on the cone shoulder

FS = factor of safety

Calculation of the allowable end bearing capacity was performed based on the results of the CPT soundings through the use of the following equation (Eslami and Fellenius Method, FHWA 2016):

$$q_{p(all)} = \frac{C_p q_{Eg}}{FS} \qquad \text{(FHWA, 2016)}$$

where:

$q_{p(all)}$ = allowable unit toe resistance

C_p = toe correction coefficient, equal to 1.0 (FHWA, 2016)

q_{Eg} = geometric average of the cone tip resistance over the influence zone after correction for pore pressure on shoulder and adjustment to effective stress.

FS = factor of safety

Based on these formulas and the results of CPT testing, estimates of ultimate axial capacity per square foot of end bearing and per square foot of side friction for each Design Profile are presented in Table 14. The preliminary design values provided in Table 14 should be reduced by an appropriate factor of safety

by the foundation designer. A factor of safety of 2 is recommended for use in preliminary design for a wind loading scenario.

Nominal axial resistance within the upper 6 inches below grade is recommended for preliminary design considerations. It is recommended that a hydrology study be performed to determine if the analysis should incorporate a scour depth. When available, the pile analysis should be revised to account for the effects of scour.

The preliminary design values provided in Table 14 are based on correlations to CPT data. If a pile foundation system is selected for use at the site, a pile load testing program is recommended during later phases of design. It should be noted that this pile load testing is critical to the final foundation design, as load testing provides information regarding actual pile performance. Pile load testing should include testing at a sufficient number of diverse locations within the project site.

6.3.2.1.1 Axial Tension Loading

Preliminary axial tension analyses were performed for W8x13 and W8x15 section piles with the following assumed loading conditions and the preliminary axial design parameters outlined in Table 14:

- Exterior Columns: 600 lbs axial tension
- Interior Columns: 400 lbs axial tension

Preliminary axial tension capacity plots that incorporate a factor of safety of 2 are included in Appendix M.

6.3.2.1.2 Axial Compression Loading

Preliminary axial compression analyses were performed for W8x13 and W8x15 section piles with the following assumed loading conditions and the preliminary axial design parameters outlined in Table 14:

- Exterior Columns: 1,800 lbs axial compression
- Interior Columns: 1,600 lbs axial compression

Preliminary axial compression capacity plots that incorporate a factor of safety of 2 are included in Appendix M.

6.3.2.2 Lateral Capacity

Based on the results of this preliminary geotechnical investigation, geotechnical parameters recommended for use in preliminary lateral pile capacity calculations are presented in Table 15. Preliminary analyses were performed for W8x13 and W8x15 section piles using the software LPile 2022 created by Ensoft Inc. The following two assumed loading cases were used:

- Exterior Columns: 2,500 lbs shear, 34,200 in.-lb moment, 2,000 lbs axial compression
- Interior Columns: 2,200 lbs shear, 36,300 in.-lb moment, 1,500 lbs axial compression

The loads were applied at the pile head, which was assumed to be 5 feet above existing grade, in the strong-axis direction during the analyses. The analyses assumed a maximum allowable lateral deflection of 0.6 inches measured at the ground surface.

Nominal lateral resistance within the upper 6 inches below grade is recommended for preliminary design considerations as noted in Table 15. Lateral deflection values from the preliminary analysis are summarized in Table 16 and the LPile output files are provided in Appendix N.

6.3.3 Settlement

The foundation type, dimensions, and depths are not known or not provided, as such, estimated settlements cannot be computed at this time. Estimates for settlement of driven piles or other deep foundations should be computed prior to final structural design.

6.3.4 Preliminary Pile Analysis Summary

Based on the geotechnical analyses for an axial tension loading case (Section 6.3.2.1.1), an axial compression loading case (Section 6.3.2.1.2), and a lateral loading case (Section 6.3.2.2) the recommended minimum embedment depths for driven piles are summarized below:

- Design Profile 1:
    - Exterior Columns (W8x15): embedment depth of 10 feet
    - Interior Columns (W8x13): embedment depth of 9 feet
- Design Profile 2:
    - Exterior Columns (W8x13): embedment depth of 8.5 feet
    - Interior Columns (W8x13): embedment depth of 8 feet
- Design Profile 3:
    - Exterior Columns (W8x13): embedment depth of 8.5 feet
    - Interior Columns (W8x13): embedment depth of 7.5 feet

The preliminary pile size and embedment depth are based on geotechnical analyses only (not structural capacity) and exclude evaluation of the impacts of liquefaction. The structural engineer should verify that the final pile design develops sufficient capacity for all applicable LRFD or ASD load combinations once final design and loading information is available. Additionally, the structural engineer should verify the structural capacity including accounting for any design section loss due to corrosion.

6.4 Shallow Foundations

For shallow foundations, it is anticipated that strip or spread footings would be used. A minimum embedment depth of 6 inches is recommended for foundation stability and protection against frost (Section 6.7).

6.4.1 Allowable Bearing Capacity of Shallow Foundations

The ultimate bearing capacity of the soil supporting a spread footing can be determined using the Terzaghi-Meyerhoff equation as follows:

$$q_{ult} = {}^{1}/_{2}\,\gamma B N_{\gamma} + q N_{q} + s_{u} N_{c} \qquad \text{(Das, 2007)}$$

where:

γ '= effective soil unit weight

B = foundation width

N_γ = bearing capacity factor

q = effective stress at foundation embedment depth

N_q = bearing capacity factor

s_u = undrained shear strength

N_c = bearing capacity factor

The first term of the above equation is associated with granular soils which typically exhibit drained modes of failure (except under earthquake loading) and where excess pore pressures do not build up in the soil when loaded.

The third term of the equation is associated with fine-grained/clayey soils which typically exhibit an undrained mode of failure and where excess pore pressures can build up in the soil when loaded.

To calculate the net ultimate undrained bearing capacity, the second term drops off and only the third term remains in the equation. As a result, the following equation representing the net ultimate undrained bearing capacity is as follows:

$$q_{ult(net)} = s_u N_c$$

where:

s_u = undrained shear strength

N_c = bearing capacity factor = 5.14 (Das, 2007)

For foundations supported by primarily granular soils, the third term of the bearing capacity equation drops off, as the cohesion for these types of soils is typically taken as zero. The first and second terms remain, along with an additional term to calculate the net ultimate bearing capacity, resulting in the following equation:

$$q_{ult(net)} = {}^{1}/_{2}\,\gamma' B N_\gamma + q N_q - q$$

where:

γ = soil unit weight

B = foundation width

q = effective stress at the foundation embedment depth

N_γ = bearing capacity factor = $2(N_q - 1)\tan\phi$ (Das, 2007)

N_q = bearing capacity factor = $tan^2\left(45 + {}^{\phi}/_{2}\right)e^{\pi\tan\phi}$ (Das, 2007)

Calculation of the bearing capacity is a function of the soil shear strength parameters and the foundation dimensions, so final estimates of the bearing capacity cannot be explicitly calculated at this time.

As the subsurface conditions consist of predominantly clays in some areas of the site, the undrained condition will be the limiting bearing capacity case. Based on a representative undrained shear strength of 600 psf correlated from CPT data within cohesive soils in the upper 10 feet (Section 4.5.1), a foundation width of 5 feet, and a minimum foundation embedment depth of 6 inches below grade, a net allowable bearing capacity of approximately 1,000 psf is recommended for preliminary design purposes, incorporating a factor or safety of 3.

6.4.2 Sliding Friction

The friction coefficient between the site soils and shallow foundation concrete should be taken as 0.40 in accordance with recommendations provided by Potyondy (1961), assuming cast-in-place concrete.

6.4.3 Settlement

Settlement is an important factor in the design of foundations. Due to the fact that loads and specific details regarding these foundation systems are not available at this time, it is recommended that the foundation designer evaluate the settlement of the soils under foundation loading at some point in the future when these details are known. The allowable bearing capacity provided in Section 6.4.1 incorporated a preliminary settlement analysis that assumed a maximum foundation width of 10 feet and a maximum allowable settlement of 1 inch.

6.4.4 Liquefaction

Based on the results of the preliminary liquefaction risk assessment presented in Section 5, some of the soils at the site are susceptible to liquefaction. The preliminary liquefaction risk assessment was based on available information and did not consider the impacts of amplification or de-amplification of seismic waves through the soils at the site.

The recommended net allowable bearing capacity for use in preliminary spread footing foundation design presented in Section 6.4.1 does not consider the risk of liquefaction. The design considerations related to liquefaction should be further evaluated during later phases of design.

6.5 Shrink-Swell Potential

Clays exhibiting moderate to high levels of plasticity can be an indicator of the potential for soil swell. Atterberg limits and grain size analysis testing on samples collected from the test pits generally indicate low to moderate swell potential, though the sample collected at a depth of 2.5 to 3 feet within test pit GEO-15 indicating high swell potential (Section 4.4; Das, 2007). In general, while the results of the preliminary geotechnical investigation indicate a relatively low risk of significant shrink-swell throughout a majority of the project site, some areas of the site appear to be at risk. The risk of soil shrink-swell and associated design considerations should be further evaluated during the design phase geotechnical investigation.

6.6 Soil Collapse

As noted in Section 2.5.7, eolian and alluvial deposits, such as those present in portions of the project site, may be prone to collapse due to their deposition, structure, and characteristics. A review of the relative density inferred from the CPT data collected in the upper 10 feet indicates some soils may be susceptible to collapse.

Soil collapse is generally triggered by two factors: increased moisture levels and the application of load. The proposed solar development is assumed to not introduce increased moisture to the general project area; however, periodic precipitation or flooding events may be a source of increased moisture. Sections 6.6.1 and 6.6.2 present preliminary recommendations related to mitigation of soil collapse. The risk of soil collapse and associated design considerations should be further evaluated during later phases of design.

6.6.1 Pile Foundations

Pile foundations are anticipated to be used for support of the solar modules. The installation methods, including driving and vibration, will likely break down adhesion in the soil surrounding the pile, which will reduce the potential for future collapse. Additionally, the proposed solar structures impose a relatively minor load on the soils.

On a preliminary basis, the following is recommended for design of the foundations to support the solar modules:

- Design the foundation/soil interface at the ground surface to shed precipitation or surface runoff and limit the potential for gapping and infiltration to occur.
- The potential for gaps to form along the side of a pile due to collapse of the shallow soils under lateral loading may necessitate a more robust maintenance program to replumb structures and fill the gaps to prevent water infiltration to greater depth. The designer should consider the depth of embedment for piles to balance the risk of collapse potential with the tolerance for maintenance activities.

6.6.2 Shallow Foundations

Based on the results of the preliminary geotechnical investigation, the soils underlying shallow foundations may be at some risk of soil collapse in areas of the project site. Wetted collapse of the soils will tend to break or dissolve the soil bonds and potentially result in relatively high magnitude, rapid, or differential settlement. If shallow foundation systems will be utilized, subsurface remediation to mitigate the risk of soil collapse is recommended on a preliminary basis, though the risk of soil collapse and potential risk mitigation strategies should be further evaluated during future phases of geotechnical investigation and design.

To mitigate the risk of soil collapse below shallow foundations at this project site, it is recommended that surface compaction be completed at the foundation bearing grade. Surface compaction should be performed by using either a smooth-drum vibratory compactor with a minimum of four passes (two passes in each perpendicular direction) and should be compacted to a minimum of 95 percent of standard Proctor maximum dry density.

6.7 Frost Adfreeze

The estimated extreme frost penetration depth for this project location is approximately 6 inches (UFC, 2022). Due to the relatively shallow extreme frost penetration depth at the project site, the risk of significant tangential frost heave forces (adfreeze forces) along steel piles is considered low. In addition, based on the recommended steel pile embedment depths presented in Section 6.3.4, the axial tension capacity will exceed the tangential frost heave forces.

6.8 Seismicity Considerations

As discussed in Section 5, the presence of liquefiable soils results in seismic Site Class F, which requires that a site response analysis be conducted to determine the geotechnical design criteria per ASCE 7. The site response analysis estimates amplification or de-amplification of seismic waves as they propagate upward from the bedrock surface through the soil profile to the ground surface, which is determined based on subsurface stratigraphy, soil type, shear wave velocity profile, and input accelerations across the project site. The ASCE hazard tool website (ASCE, 2024) was used to determine seismic parameters for the underlying "bedrock" (assumed Site Class B, or a shear wave velocity of about 2,500 ft/s) based on the 2021 International Building Code. The parameters are presented below.

Representative Coordinates: 35.86050°N, 119.83762°W

S_S = 1.137g

S_1 = 0.407g

Due to the presence of liquefiable soils, a site response analysis will need be conducted during later phases of design.

6.9 Dewatering

Based on the discussion presented in Section 4.2, groundwater is present at depths ranging from approximately 4 to 10 feet below existing grade throughout much of the project site. Dewatering may be required for excavations that extend deeper than the static groundwater level or are allowed to remain open for extended periods of time. In the event of heavy rainfall, water outflow from excavations may be limited by lower permeability soils. It is recommended that the contractor remove infiltrating and surface water in each foundation excavation at all times during construction (i.e., on a continuous basis). At no time should an excavation be allowed to fill with water as this practice may lead to damage or weakening of the subgrade soils. Standard sump and pit methods may be suitable for some areas where excavations are not anticipated to extend below the groundwater level. For excavations extending slightly below the piezometric water level, or shallow excavations where water cannot be controlled (removed) by these methods, the contractor should be prepared to slope the excavation bottoms to a low point and construct a sump for groundwater removal. The groundwater may not be suitable for direct discharge.

6.10 Erosion Control

Measures should be taken to prevent runoff from depositing sediment in any excavations in the case of rain. Erosion of the roadways should be anticipated where they cross existing swales or act as preferred drainage paths; ongoing maintenance will be required following most rain events and appropriate equipment should be available on site or nearby on a permanent basis throughout operation of the facility. Disturbance from construction activities may result in greater potential for wind or water erosion.

Long term erosion control measures for the project site largely consist of revegetation for all areas of the project site where it can be performed following construction. Revegetation will help control erosion from both wind and water. In areas where revegetation is not possible, wind erosion should be anticipated. Control of water erosion will generally consist of the design of erosion control structures, berms, diversion devices, and other drainage control measures to limit drainage to confined zones away from the structures. The design of specific erosion control structures should be performed by a civil engineer that specializes in this practice. It is recommended that a hydrology study be performed to determine if scour should be incorporated into the design of the project.

6.11 Soil Corrosivity

Soil electrical resistivity, pH, and soluble chloride ion and sulfates are some of the primary factors in evaluating the rate and amount of corrosion of buried structures. A laboratory and field testing program was conducted to evaluate these factors (Section 3 and Section 4). It should be noted that soil corrosivity is also influenced by other variables including the amount of moisture, drainage, soil particle size/oxygen content, and organic content.

Soil electrical resistivity indicates the relative capability of the soil to conduct electrical current and is generally recognized as one of the most significant soil characteristics with regard to corrosion potential. In general, soils with low electrical resistivity exhibit high corrosion potential. The electrical resistivity testing program for this project included both laboratory electrical resistivity testing (Section 4.6) and field electrical resistivity testing (Section 4.7). Based on the results of the testing, the soils can be classified as severely corrosive to progressively less corrosive to steel when at typical in-situ moisture contents (Ellis, 1978).

The degree of acidity and alkalinity of a soil is expressed as the pH value, where 7 represents neutral conditions, lower numbers indicate increasing acidity and higher numbers represent increasing alkalinity. Soils with low pH are considered acidic and have a high corrosivity. Soils with high pH have high alkalinity which lowers soil electrical resistivity and increases corrosivity. High acidity soil can also react with concrete. Laboratory test results (Table 8) indicate the soils are slightly alkaline (as high as 9.0 pH), and considerations for corrosion potential with regards to soil pH should be reviewed.

Chloride ion contents in the soil can result in accelerated corrosion of steel reinforcement in concrete and steel structures by breaking down the normally present protective layer of oxides on the steel surface, as well as decreasing the soil electrical resistivity. Laboratory test results (Table 8) indicate the soils have chloride ion contents that range 7.1 to 3,590 mg/kg (ppm), and consideration for corrosion potential with regards to chloride ion content should be reviewed.

Sulfate ions, similar to chloride ions, may result in accelerated corrosion of steel. High sulfate contents are particularly deleterious to concrete by reacting with the lime in the concrete. A sulfate concentration of 1,000 ppm or greater is a generally accepted indication of corrosive conditions. Laboratory test results (Table 8) indicate the soils have sulfate ion concentrations range from 25.5 to 10,300 mg/kg (ppm), and consideration for corrosion potential with regards to sulfate ion content should be reviewed.

In summary, based on the field electrical resistivity testing, pH testing, and sulfate ion testing, there is a potential for high corrosivity. The corrosivity of the soils at the site and associated design considerations should be further reviewed during later phases of design.

6.11.1 Cement Type

Based on the results of soil analytical testing presented in Section 4.5 and Table 8, cement with an S2 exposure class appears necessary for use in foundation design (ACI, 2019). The foundation engineer of record has final responsibility for selection of the appropriate cement mixture based on the results provided in this report.

7 Preliminary Substation Recommendations

Section 7 presents the results of the preliminary geotechnical investigation and provides analysis, conclusions, and preliminary recommendations for the design and construction of proposed substation structure foundations.

7.1 Soil Stratigraphy

The location of the CPT sounding and test pit completed at the proposed substation location are shown in Figures 10 and 11. CPT sounding logs are included as Appendix B, test pit logs are included as Appendix E, and test pit photographs are included as Appendix F.

Test pit SUB-Opt1-01 indicated the presence of approximately 12 inches of clayey sand topsoil underlain by sandy lean clay extending to a depth of approximately 4 feet below existing grade. Silty sand was encountered below the sandy lean clay and extended to the termination depth of the test pit at approximately 10 feet below existing grade.

The CPT-interpreted soil behavior type for sounding SUB-Opt1-01 indicated primarily sands from the ground surface to a depth of approximately 7.5 feet below existing grade. Silt mixtures, sand mixtures, and clays were the primarily CPT-interpreted soil behavior types from a depth of approximately 7.5 to 50 feet below existing grade and were underlain by primarily sands extending to a depth of approximately 56 feet below existing grade. CPT-interpreted clays were the predominant material between approximately 56 feet and the termination depth of the sounding at approximately 100 feet below existing grade, though relatively thin layers of silt mixtures, sand mixtures, and sands were also encountered over this depth interval.

7.2 Groundwater and Dewatering

Groundwater was not encountered within the test pit excavation at investigation location SUB-Opt1-01, which extended to a depth of approximately 10 feet below existing grade. However, due to the presence of clay, groundwater observations in the test pit excavation may not provide a reliable estimate of the groundwater table. Results of pore pressure dissipation testing performed within CPT sounding SUB-Opt1-01 indicated a groundwater level of approximately 22 feet below existing grade. Many factors contribute to water level fluctuations, such as heavy rainfall events, dry periods, etc., and the measurements noted during this investigation may not represent the long-term groundwater levels present at the site. Appendix O provides the recommended groundwater level to be used for preliminary substation foundation design.

Dewatering may be required for excavations that extend deeper than the static groundwater level or are allowed to remain open for extended periods of time. In the event of heavy rainfall, water outflow from the excavation may be limited by lower permeability clays, and typical dewatering can be achieved by use of a sump pit and small pump. Following rain, excavations in soil should be allowed to return to the in-situ moisture content before continuing with construction. The contractor should be prepared to remove water in all excavations at all times during construction (i.e., on a continuous basis). At no time should an excavation be allowed to fill with water as this practice may lead to damage or weakening of the subgrade soils.

7.3 Laboratory Testing Results

Five moisture content tests were performed on soil samples collected from test pit SUB-Opt1-01, including four tests performed on grab samples at depths ranging from approximately 0.5 to 4 feet below existing grade and one test performed on the bulk sample collected at a depth interval of approximately 3 to 5 feet below existing grade in support of thermal resistivity testing. The results indicate moisture contents ranging from 4 to 13 percent, with an average of 9 percent, generally indicating dry to moist conditions.

One Atterberg limits test was performed on the bulk sample collected at a depth interval of approximately 3 to 5 feet below existing grade at location SUB-Opt1-01 in support of thermal resistivity testing. The results indicate a liquid limit of 29 percent, a plastic limit of 17 percent, and a plasticity index of 12 percent, classifying the sample as lean clay (CL) in accordance with the USCS.

Grain size analyses (sieve analyses) were performed on two soil samples from test pit SUB-Opt1-01, including one grab sample collected at a depth of approximately 1.5 to 2 feet below existing grade and one bulk sample collected from a depth of approximately 3 to 5 feet below existing grade. The percent fines of the samples tested ranged from approximately 55 to 75 percent, while the percent gravel was 0 percent for both samples tested.

7.4 Recommended Preliminary Substation Foundation Design Parameters

Recommended geotechnical parameters for use in preliminary substation foundation, including density and strength, are included in Appendix O.

Based on the results of the preliminary liquefaction risk assessment presented in Section 5, some of the soils encountered within CPT sounding SUB-Opt1-01 are susceptible to liquefaction. The preliminary liquefaction risk assessment was based on available information and did not consider the impacts of amplification or de-amplification of seismic waves through the soils at the site. Therefore, the recommended geotechnical parameters for use in preliminary substation foundation included in Appendix O do not consider the risk of liquefaction. The design considerations related to liquefaction should be further evaluated during later phases of design.

7.5 Foundation Type

It is assumed that spread footings or drilled piers will be utilized for support of critical substation structures. Other lightweight equipment associated with the substation is assumed to be supported on slabs-on-grade. Sections 7.6 and 7.7 provide a discussion on allowable load capacity of the anticipated foundation systems based on the available geotechnical data.

7.6 Shallow Foundations

7.6.1 Bearing Capacity

Refer to Section 6.4.1 for a discussion on the determination of the bearing capacity recommendations for shallow foundations. A summary of the recommended soil strength parameters and net allowable bearing capacities for preliminary substation spread footing foundation design is provided in Appendix O. Note that the net allowable bearing capacities provided are in the absence of a structure layouts, including

specific foundation locations and parameters. Net allowable bearing capacities are provided in Appendix O for the following assumed foundation dimensions:

- 5-foot-wide by 5-foot-long spread footing foundation
- 15-foot-wide by 25-foot-long spread footing foundation

The net allowable bearing capacities provided in the tables incorporate a factor of safety of 3. Spread footing foundation bearing capacity and settlement calculations are dependent on the foundation dimensions. Once final foundation design information is available, Barr should be contacted to determine if the recommendations provided are still applicable.

7.6.2 Settlement

Settlement is an important factor in the design of foundations. Due to the fact that loads and specific details regarding the planned foundation systems are not available at this time, it is recommended that the foundation designers evaluate the settlement of the soils under foundation loading when these details are known. Values of the allowable bearing capacity provided in Appendix O assume that a maximum settlement of 1 inch is acceptable for foundation design.

7.6.3 Sliding Friction

For shallow foundations, the friction coefficient between the site soils and construction materials should be taken as 0.40 assuming cast-in-place concrete (Potyondy, 1961).

7.7 Drilled Shaft Foundation Systems

7.7.1 Axial Capacity

In the absence of foundation design information, specific recommendations for bearing capacity and lateral load capacity cannot be calculated. The foundation designer should calculate the design capacity once final design information is available. The design values established in Appendix O are recommended for use in preliminary design.

Calculation of the allowable end bearing capacity during drained loading conditions may be performed through the use of the following equation.

$$q_{p(all)} = \frac{q'(N_q - 1)}{FS} < \frac{0.5 p_a N_q^* \tan\phi}{FS} \qquad \text{(Das, 2007)}$$

where:

q' = vertical effective stress at drilled shaft bearing elevation

N_q = bearing capacity factor $= tan^2\left(45 + {\phi'}/{2}\right) e^{\pi tan\phi}$

ϕ' = soil friction angle

p_a = atmospheric pressure

N_q^* = bearing capacity factor (Das, 2007)

FS = factor of safety = 3

Calculation of the skin friction during drained loading conditions may be performed through the use of the following equation.

$$q_{s(all)} = \frac{\pi d(1-sin\,\varphi')\int_0^{L_i}\sigma'_o\,tan\,\delta' dz}{FS} \qquad \text{(Das, 2007)}$$

where:

d = pier diameter

σ_o' = vertical effective stress

ϕ' = soil friction angle

δ' = soil-drilled shaft friction angle = ¾*ϕ' (NAVFAC, 1986)

L_i = soil layer thickness

FS = factor of safety = 3

Calculation of the allowable end bearing capacity during undrained loading conditions may be performed through the use of the following equation.

$$q_{p(all)} = \frac{s_u N_c^*}{FS} \qquad \text{(Das, 2007)}$$

where:

s_u = undrained shear strength

N_c^* = bearing capacity factor = 9 (Das, 2007)

FS = factor of safety = 3

Calculation of the skin friction during undrained loading conditions may be performed through the use of the following equation.

$$q_{s(all)} = \frac{\pi d(\alpha s_u)}{FS} \qquad \text{(Das, 2007)}$$

where:

d = pier diameter

α = factor correlating soil cohesion to adhesion = 0.4 (Das, 2007)

s_u = undrained shear strength

FS = factor of safety = 3

Final axial design capacity will depend on the depth of the shaft, diameter of the shaft, and installation methods. If the depth/diameter ratio is less than 4, a reduction in the end bearing capacity factor (N_{cs}) will be required, as the foundation will act more as a column footing than a deep foundation.

A summary of the recommended skin friction and end bearing values for preliminary drilled shaft foundation design is provided in Appendix O. Please note that these values are presented as the ultimate skin friction and ultimate end bearing values and an appropriate factor of safety must be applied for design purposes.

Axial capacity values are provided for both drained and undrained loading conditions. The granular layers (sands) are subject to drained loading conditions and are not subject to undrained loading conditions. The cohesive layers (clays) are subject to drained and undrained loading conditions. For transient, short-term loading, the design profile should consist of undrained parameters for the cohesive layers and drained parameters for the granular layers. For static, long-term loading, the design profile should consist of drained parameters for both cohesive and granular layers. Additional recommendations can be provided based on specific foundation design information.

7.7.2 Resistance to Uplift

Design of drilled piers for uplift should be based on a combination of the skin friction and the weight of the pier/pile. As is standard, skin friction should not be assumed to contribute to uplift resistance throughout the frost zone and the weight of the pier/pile should be taken as the buoyant weight at depths below the water table. Skin friction contributing to uplift resistance in cohesive and cohesionless soils should be taken as the skin friction used in compression less a 10 percent reduction (FHWA, 2018) applied to account for the potential loss of lateral earth pressure in uplift.

7.7.3 Lateral Resistance

It is assumed that the computer program LPile or MFAD will be used for the final detail design of drilled shaft (by others) to evaluate the lateral capacity of the soil system supporting the foundations. A summary of the recommended values for lateral resistance related to preliminary foundation design, including soil density, soil shear strength, and LPile and MFAD design parameters is provided in Appendix O.

7.7.4 Settlement

Drilled shaft spacing, dimensions, and embedment depths are unavailable at the time of preparation of this report. As such, estimated settlements cannot be computed at this time. Estimates for settlement of drilled shaft foundations should be computed prior to final design.

7.8 Earth Pressures

Rankine's active and passive pressure coefficients can be determined for both drained and undrained conditions. Rankine's theory assumes vertical walls and that backfill slopes and wall friction angles are equal to zero. The coefficients are defined as follows:

$$K_a = tan^2\left(45 - {}^{\phi}/_{2}\right) \qquad K_p = tan^2\left(45 + {}^{\phi}/_{2}\right)$$ (Das, 2007)

where:

ϕ = angle of friction for material

K_a = coefficient of Rankine's Active Earth Pressure

K_p = coefficient of Rankine's Passive Earth Pressure

For clays with loading resulting in an undrained state (such as initial loading during or following construction or loading in response to a transient event such as wind, snow, or seismic events), the angle of friction is assumed to be zero and the earth pressure coefficients are 1.0. The active and passive pressure coefficients for clays (undrained and drained) and granular soils (drained) are presented in Appendix O.

7.9 Shrink-Swell Potential

Soils exhibiting moderate to high levels of plasticity can be an indicator of the potential for soil swell. The clays encountered at relatively shallow depths within the test pit performed at the proposed substation location were generally of low plasticity and typically contained more than 20 percent sand. These soils are not considered susceptible to significant shrink-swell. Therefore, based on the results of the preliminary investigation, the risk of significant soil shrink-swell at the proposed substation location is considered low.

To prevent soil swell, it is critical to carefully control moisture levels in the exposed subgrades following initial excavation. Water should not be allowed to pond in the base of excavations and subgrades should not be allowed to remain open for extended periods of time as this may lead to drying and, as a result, an increased risk of soil swell. Utility trenches should be constructed in a manner that prevents additional moisture from migrating down into the subgrade soil. Final site grading should be designed such that surface runoff is directed away from the foundations, with consideration for slightly steeper slopes than typical to account for any long-term settlement of the surface soils surrounding the slabs and foundations.

The risk of soil shrink-swell at the proposed substation location should be further evaluated during the design phase geotechnical investigation.

7.10 Soil Collapse

As noted in Section 2.5.7, alluvial deposits, such as those present at the proposed substation location (Figure 5), may be prone to collapse due to their deposition, structure, and characteristics.

Soil collapse is generally triggered by two factors: increased moisture levels and the application of load. The proposed substation development is assumed to not introduce increased moisture to the general project area, however periodic precipitation events may be a source of increased moisture. Sections 7.10.1 and 7.10.2 present preliminary recommendations related to mitigation of soil collapse. The risk of soil collapse should be further evaluated during later phases of design.

7.10.1 Drilled Shaft Foundations

On a preliminary basis, the following is recommended for design of the drilled shaft foundations at the proposed substation location:

- Design the foundation/soil interface at the ground surface to shed precipitation or surface runoff and limit the potential for gapping and infiltration to occur.

- The potential for gaps to form at the interface of the foundation and the soils due to lateral loading may necessitate a more robust maintenance program to fill the gaps to prevent water infiltration to greater depth. The designer should consider the depth of embedment to balance the risk of collapse potential with the tolerance for maintenance activities.

7.10.2 Shallow Foundations

For shallow foundations utilized for support of substation structures, subsurface remediation to mitigate the risk of soil collapse is recommended on a preliminary basis, though the risk of soil collapse and potential risk mitigation strategies should be further evaluated during future phases of geotechnical investigation and design.

To mitigate the risk of soil collapse below shallow foundations at this project site, it is recommended that surface compaction be completed at the foundation bearing grade. Surface compaction should be performed by using either a smooth-drum vibratory compactor with a minimum of four passes (two passes in each perpendicular direction) and should be compacted to a minimum of 95 percent of standard Proctor maximum dry density.

7.10.3 Frost Depth

The estimated extreme frost penetration depth for this project location is approximately 6 inches (UFC, 2022). A minimum embedment depth of 6 inches is recommended for foundation stability and protection against frost.

7.11 Excavations

7.11.1 Clearing and Grubbing

The proposed substation area consists of undeveloped land and clearing and grubbing will generally consist of removal of brush, tall grass, and topsoil. Based on observations made during the geotechnical field investigation, the thickness of the topsoil and/or root zone at the proposed substation was observed to be about 12 inches.

Any topsoil or organic material should not be used for structural fill and should be placed separately to avoid contamination with other excavated soils. This material could be used in grading non-structural fill such as fields or service areas in which compressibility of the material does not have an impact on structures.

7.11.2 Subgrade Preparation

Upon excavation to the proposed bearing surface, the excavation base should be inspected by the construction phase geotechnical engineer of record or authorized representative. If the excavation base surface is deemed unsuitable for construction in the opinion of the geotechnical engineer of record or authorized representative, the conditions shall be noted and communicated to the foundation and geotechnical engineers for appropriate foundation modifications.

If, in the course of excavating the foundation, the base of the excavation becomes rutted, damaged or is otherwise determined to be of inadequate character, the following actions should be performed:

- If the subgrade soils consist of silts or clays, the excavation should be subcut a minimum of 6 inches beyond the depth of the rutted soils and backfilled with engineered fill. Granular fill placed for subgrade stabilization below foundations should comply with Section 6.1.4.

- If the subgrade soils consist of primarily granular soils, the excavation base surface should be leveled and surface compacted. Compaction of this material is required to achieve at least 98 percent of the laboratory maximum dry density measured according to the standard Proctor method.

7.11.3 Shallow Excavations

Results of the preliminary geotechnical investigation indicate that cut and fill of the subsoil in the area can be achieved with conventional machinery.

Based on the results of CPT sounding SUB-Opt1-01, the soils at the proposed substation location can generally be classified as Type C from OSHA soil classifications (29 CFR 1926 Subpart P-Excavations). It is the responsibility of the competent field personnel to verify the in-situ soil classification at each excavation and ensure that the benching or slopes are adequate during construction.

7.11.4 Fill Below Foundations

Any fill placed below foundations should consist of approved engineered fill. Engineered fill should comply with the recommendations presented in Section 6.1.4.

The excavations and subsequent placement of engineered fill should be oversized by 1 foot on all sides for each foot of excavation below the foundation embedment depth. For example, a 1-foot excavation below the foundation depth will require a bottom of foundation excavation width (and length) 2 feet wider than a standard foundation excavation width (1 foot on each side of the footing). The base of every excavation should be inspected by a representative of the geotechnical engineer to evaluate the soil conditions at the base of the over excavation prior to fill placement.

7.11.5 Backfill Materials

The native site soils can be used as backfill for the foundation if it meets the following conditions:

- Material should be placed in loose lifts not exceeding 12 inches in thickness.

- The material is placed at a minimum of 95 percent of the standard Proctor dry density.

- The backfill surface is graded such that water is directed away from the foundation to prevent moisture infiltration (minimum slope of ¼ inch per foot).

- Density is checked in the field periodically to verify adequate compaction (typically one test per 2,500 square feet of fill and no less than one test for each lift).

On-site backfill may consist of native soil, although is it recommended the liquid limit of these materials should not exceed 45 and the plastic index should not exceed 30 when used in a non-structural application. Most of the on-site material will meet this requirement based on laboratory testing completed. However, it is recommended that all material in consideration for use as fill be tested and compared with this recommendation prior to use. More plastic clays may be used but may be difficult to place and compact if wet. Note that existing and native fine-grained soils (silt or clay) may be particularly difficult to

compact if wet or allowed to become wet, or if spread and compacted over wet or marginally stable subgrades. The backfill material should be debris-free and free of any organic matter.

7.11.6 Drilled Shaft Foundation Excavations

The base of the drilled shaft excavations should be firm and free of loose or disturbed soils. If necessary, any loose or disturbed soils should be removed from the excavation with a clean-out bucket. Rebar cages should be installed immediately prior to concrete placement to facilitate cleaning of the bottom of the excavation.

Granular soils may be prone to sidewall collapse and temporary casing and/or drilling fluid will likely be required to maintain stability of the excavation at some locations. All excavations should be adequately supported to prevent sloughing of the soil into the hole. Sloughing soil can lead to a loss of strength in the soils surrounding the foundation system, which would require a redesign of the foundation to properly account for the strength reduction. All temporary casing should be removed from the excavation during concrete placement. If temporary casing will remain in-place the foundation designer should be consulted to determine if additional foundation capacity is required.

The presence of groundwater may be encountered during the construction of the drilled shafts or caissons. Concrete should be placed using a tremie pipe to properly displace the water accumulated in the excavation. The tremie pipe should be placed to the bottom of the excavation to minimize mixing of the concrete and water during placement.

7.12 Cement Type

Analytical testing was performed on samples collected from eight test pit locations throughout the site, including seven samples collected within the proposed solar array areas and one sample collected at the proposed substation location. A summary of the results is presented in Table 8.

Depending on the sulfate content, the American Concrete Institute recommends specific types of cement for use in design to prevent against sulfate attack. Based on the testing completed, cement with an S2 exposure class appears necessary for use in substation foundation design (ACI, 2019). The foundation engineer of record has final responsibility for selection of the appropriate cement mixture based on the results provided in this report.

8 Preliminary Road Section and Design

Section 8 provides analysis, conclusions and recommendations for the design and construction of roads and laydown areas.

For roads the primary factors addressed include gravel thickness and subgrade preparation.

8.1 Road Base Design Considerations

The preliminary design thickness of placed granular fill was determined using CBR values. Based on the results presented in Section 4.9, a preliminary design CBR value of 5 percent is recommended for road section design at a subgrade compaction value of 95 percent of the standard Proctor maximum density. Typically, roadways for solar power developments are reinforced by chemical stabilization of the subgrade, geogrids, or geotextiles. The following section describes the required aggregate thicknesses for various reinforcement options.

8.1.1 Road Section Design

The use of aggregate base over the subgrade for roadways and laydown areas will provide for a stronger roadway, less rutting, and better overall performance in various weather conditions. The recommended aggregate thickness is based on subgrade compaction of a minimum of 95 percent of the soil's maximum dry density (determined through standard Proctor testing), and the preliminary design CBR value of 5 percent.

Three traffic conditions were evaluated and analyzed for use of the access roads: (1) conditions during construction of the project, (2) emergency vehicle access during operation of the facility, and (3) long term maintenance traffic consisting of light duty trucks during operation. Assumed design parameters for each traffic condition are presented below. The aggregate thickness required to support the assumed design loads was determined utilizing the Giroud-Han iterative equation (Giroud and Han, 2004). Revised aggregate thickness calculations can be performed by Barr with project specific design parameters upon request.

The construction condition assumes the parameters noted below. The recommended preliminary aggregate thickness for the construction condition is shown in Table 17.

- Subgrade CBR value of 5.0 percent
- Aggregate base CBR value of approximately 25 percent
- Maximum axle load of 10 kips
- Tire pressure of 100 psi
- 3,000 axle passes
- Maximum allowable rut depth of 1.5 and 3.0 inches

The emergency vehicle access condition assumes the parameters noted below. The recommended preliminary aggregate thickness for the construction condition is shown in Table 17.

- Subgrade CBR value of 5.0 percent
- Aggregate base CBR value of approximately 25 percent
- Maximum axle load of 33.5 kips
- Tire pressure of 100 psi
- 250 axle passes
- Maximum allowable rut depth of 1.5 and 3.0 inches

The maintenance condition assumes the parameters noted below. The recommended preliminary aggregate thickness for the construction condition is shown in Table 17.

- Subgrade CBR value of 5.0 percent
- Aggregate base CBR value of approximately 25 percent
- Maximum axle load of 2.5 kips
- Tire pressure of 65 psi
- 50,000 axle passes
- Maximum allowable rut depth of 1.5 inches.

While it is not required that the roads be designed for loads imparted by construction vehicles, the values provided are recommendations to avoid complications with roads during construction, and particularly in times of heavy precipitation.

At a minimum, the final road section should be of sufficient thickness to compensate for topsoil/root zone stripping and promote positive drainage away from the road. Please note that axle loads and/or axle passes in excess of the preliminary design values noted above may decrease the overall life of the road because of premature road deterioration. Following construction of the project, it is recommended that the access roadways be re-graded to reestablish the road surface. Additionally, in the event of heavy traffic or significant rain leading to excessive rutting or surface deterioration during the course of construction, it is recommended that 1 to 2 inches of gravel be added and re-graded to reestablish the road surface. Drainage and potential scour should be considered by the Civil Engineer during roadway and grading design.

If localized areas require significant cuts during the subgrade preparation phase, greater aggregate base thicknesses may be needed to allow for proper drainage of the road surface.

8.1.2 Aggregate Recommendation

It is recommended that the roadways be constructed of aggregate base material, combined with geosynthetic reinforcement (optional). It is recommended that the material comply with the requirements of Caltrans Specification 26-1.02 materials, with ¾-inch maximum and fewer than 10 percent fines or similar. If geogrid reinforcement is used, modifications to the gradation requirements may be necessary in order to comply with the geogrid manufacturer's recommendations. Alternative road surface materials may be used depending upon availability and approval.

As mentioned above, a smooth drum vibratory compactor should be used to compact the gravel roadway. This material should be compacted in loose lifts not to exceed 9 inches in thickness. The gravel roadway should be compacted to 98 percent of maximum standard proctor dry density, determined by ASTM D698. Testing should be performed on the order of one test per 2,500 to 5,000 square feet of roadway per lift.

Alternative road surface materials, such as crushed caliche, may be considered based upon availability and further review (including aggregate durability testing); however, a thicker base section may be required. Re-grading and re-compaction of unsurfaced roadways should be anticipated throughout construction and operation of the project.

9 Underground Utility Trenches

Preliminary geotechnical investigation results indicate that excavations for underground utility trenches will likely be confined to clays, silts, and sands throughout the project site and conventional excavation machinery should appears suitable for excavations. The presence of shallow groundwater within cable trenches should be anticipated in portions of the project site. The near surface soils may be sensitive to moisture changes and may be difficult to work and/or compact when wet.

10 Design Phase Geotechnical Investigation Recommendations

The test results and recommendations provided herein are for preliminary design purposes and should not be solely relied upon for final design. A thorough study of the subsurface conditions of the project site is necessary to determine the soil characteristics for the final design of the project. It is recommended that the design phase geotechnical investigation consist of the following scope:

- Detailed site reconnaissance to identify geologic or geotechnical hazards at the project site. The site reconnaissance should be performed by personnel with a background in geology or geotechnical engineering and solar power development.
- A geotechnical field investigation program of the proposed solar array areas consisting of:
    - CPT soundings at an approximate frequency of one per 30 acres of solar development. It is recommended that the CPT soundings extend to the top of competent rock, weathered rock, or very stiff/dense soils (shear wave velocity of about 2,500 ft/s) to support the site-specific seismic response analysis and evaluation of the risk of liquefaction. The CPT soundings should be supplemented with geotechnical borings at an approximate frequency of one per 200 acres of solar development to collect soil samples for laboratory testing. It is recommended that the geotechnical boring depth be determined based on the results of the CPT soundings, as it will be imperative to extend the borings to sufficient depth to gain additional geotechnical information necessary to evaluate the risk of liquefaction.
    - Seismic tests, consisting of shear and compression wave velocity measurements, concurrent with the CPT soundings. Consideration should be given to performing surface-based geophysical surveys, consisting of Multi-channel Analysis of Surface Waves (MASW), to further evaluate the shear wave velocities at the project site.
    - Pore water pressure dissipation (PPD) testing concurrent with the CPT soundings at varying depths.
    - Test pit excavations at an approximate frequency of one per 100 to 200 acres of development to a depth of 10 feet or refusal.
    - Piezometer installation and monitoring at select locations to evaluate groundwater conditions across the project site.
    - If a pile foundation system is selected for use, a pile load testing program is recommended during the design phase investigation. Pile load testing should include testing at a sufficient number of diverse locations within the project site. The recommended frequency of pile load testing is on the order of one axial tension test per 50 to 60 acres of development, one lateral load test per 50 to 60 acres of development, and one axial compression load test per 100 acres of development. It should be noted that this scope item is critical to the final foundation design, as load testing provides information regarding actual pile performance driving criteria. It is important that a preliminary or planned foundation system is known prior to pile load testing, such that test pile sections and loading criteria are properly selected.

- A geotechnical field investigation program of the proposed ancillary structure locations consisting of:
    - CPT soundings and/or geotechnical borings at proposed ancillary structure locations, such as proposed project substations, O&M buildings, transmission line structures, or other facilities planned for the project. It is recommended that the CPT soundings and/or geotechnical borings extend to the top of competent rock, weathered rock, or very stiff/dense soils (shear wave velocity of about 2,500 ft/s) to support the site-specific seismic response analysis and evaluation of the risk of liquefaction. The CPT soundings should be supplemented with geotechnical borings to collect soil samples for laboratory testing.
    - Seismic tests concurrent with the CPT soundings, consisting of shear and compression wave velocity measurements. Consideration should be given to performing surface-based geophysical surveys, consisting of Multi-channel Analysis of Surface Waves (MASW), to further evaluate the shear wave velocities at the project site.
    - Pore water pressure dissipation (PPD) testing concurrent with the CPT soundings at varying depths.
    - Piezometer installation and monitoring to evaluate groundwater conditions.
- Complete a laboratory testing program on soil samples collected during the investigation to include some or all of the following:
    - General laboratory testing to evaluate soil index properties, soil strength, and soil compressibility.
    - Laboratory testing to evaluate the risk of soil collapse and soil shrink-swell.
    - Standard Proctor density testing.
    - California Bearing Ratio (CBR) testing to evaluate subgrade strength related to access roads.
- Soil Corrosion Testing and Evaluation:
    - Soluble sulfate, soluble chloride, pH, reduction-oxidation potential, and laboratory electrical resistivity testing on select samples collected at the site.
    - Site specific soil and atmospheric corrosivity evaluation.
- Resistivity Testing:
    - Field electrical resistivity testing in support of electrical design and soil corrosion evaluation.
    - Laboratory thermal resistivity testing in support of electrical design.
- Complete a seismic site response analysis at representative locations throughout the proposed solar array area and at critical site infrastructure.

- Evaluate the risk of liquefaction throughout the proposed solar array area and site ancillary structure locations.

- Summarize final geotechnical investigation with a report detailing the field and laboratory results, an evaluation of geologic and geotechnical hazards, and a summary of geotechnical recommendations for foundation design of the solar development and other project infrastructure.

The final testing program for each discipline (geotechnical, structural, and electrical) should be determined or reviewed by their respective design engineers.

11 Limitations of Analysis

The analysis and conclusions provided are based on the results of fieldwork from recent investigations. Using generally accepted engineering methods and practices, the investigations performed have made every reasonable effort to characterize the site. However, the likelihood that conditions may vary from any specific location tested is still possible, and careful attention to soil conditions should be undertaken during the time of construction by qualified personnel. This investigation included a limited number of locations across the project site for preliminary design purposes and should not be relied upon for final design. A comprehensive design phase geotechnical investigation is required to support final design and construction.

This report is for the exclusive use of Terra-Gen Development Company, LLC. Without written approval by Barr Engineering Co., no responsibility to other parties is assumed regarding this report. The evaluation, analyses and recommendations may not be appropriate for other parties or projects.

Barr Engineering Co.’s services for this project were performed in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area under similar budget and time restraints. No warranty, expressed or implied, is made.

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Tables

Table 1

Summary of Regional Quaternary Faults

Fault Name (Fault ID)[1]	Relative Distance to Project Area / Trend Direction	Approximate Length	Age[2]	Fault Type[3]	Movement
Great Valley thrust fault system, Kettleman Hills-North Dome (GV 14) Section (28n)	8 miles northwest	19 miles	Middle and Late Quaternary <750 ka	A	Between 1.0 and 5.0 mm/yr
Poso Creek fault (381)	18 miles southeast	49 miles	Undifferentiated Quaternary <1.6 Ma	A	Unspecified
San Andreas fault Zone, Cholame-Charrizo section (1g)	25 miles west-southwest	125 miles	Latest Quaternary <15 ka	A	Greater than 5.0 mm/yr
San Juan fault zone (77)	30 miles southwest	59 miles	Latest Quaternary <15 ka	A	Unspecified

1) Quaternary faults within an approximate 30 mile radius of the project site were included in the summary and shown on Figure 4b.

2) ka = thousand years; ma = million years

3) Fault Type:

Class A: Geologic evidence demonstrates the existence of a Quaternary fault of tectonic origin, whether the fault is exposed by mapping or inferred from liquefaction or other deformational features.

Table 2

Summary of Groundwater Level Measurements

Investigation ID	Depth to Water[1,2,3] [ft]		
	Test Pits	Pore Pressure Dissipation Testing	
		Test Depth [ft]	Depth to Water [ft]
GEO-01	--	20.5	14.3
GEO-02	--	--	--
GEO-03	--	--	--
GEO-04A	--	--	--
GEO-05	10.0	--	--
GEO-06A	--	--	--
GEO-07A	--	12.3	5.1
GEO-08A	--	11.2	5.7
GEO-09	--	13.8	8.3
GEO-10A	5.0	10.3	4.2
GEO-11	--	11.3	5.4
GEO-12	--	18.0	4.6
GEO-13	NE	--	--
GEO-14	--	--	--
GEO-15	9.0	--	--
GEO-16	9.5	10.9	7.3
GEO-17	--	--	--
GEO-18	--	16.0	6.8
GEO-19	--	12.5	5.8
GEO-20	NE	--	--
GEO-21	--	12.6	11.3
SUB-Opt1-01	NE	57.5	22.4

1) Depth measured below existing grade.

2) NE indicates that groundwater was not encountered in the reading event.

3) '--' Indicates that a test pit or pore pressure dissipation test was not completed at the corresponding location.

Table 3

Summary of Soil Behavior Type from CPT Data

SBT	Description	Percent Encountered[1]				
		0-5 feet	5-10 feet	10-15 feet	15-20 feet	0-20 feet
1	Sensitive, fine-grained	0%	0%	0%	0%	0%
2	Organic soils-peats	0%	0%	0%	0%	0%
3	Clays-clay to silty clay	0%	0%	3%	12%	4%
4	Silt mixtures-clayey silt to silty clay	1%	13%	34%	58%	27%
5	Sand mixtures-silty sand to sandy silt	30%	64%	41%	21%	39%
6	Sands-clean sand to silty sand	29%	22%	20%	8%	20%
7	Gravelly sand to sand	33%	1%	2%	0%	9%
8	Very stiff sand to clayey sand	8%	0%	0%	0%	2%
9	Very stiff fine-grained	0%	0%	0%	0%	0%

1) Values rounded to the nearest whole percent.

Table 4a

Summary of General Laboratory Test Results from Test Pit Samples

Sample Location				Atterberg Limits			Grain Size Analysis		
Investigation ID	Depth [ft]	USCS Approx. Soil Type[1]	Moisture Content [%]	Liquid Limit [%]	Plastic Limit [%]	Plasticity Index [%]	Percent Gravel [%]	Percent Sand [%]	Percent Fines [%]
GEO-05	0.5-1	CL	5.5						
	1.5-2	CL	15.8	39	14	25	0.0	38.5	61.5
	2.5-3	CL	19.6						
	3.5-4	CL	20.3						
GEO-10A	0.5-1	SC-SM/SC	10.7						
	1.5-2	SC-SM/SC	13.7						
	2.5-3	SM/SC-SM	15.3				4.0	69.0	27.0
	3.5-4	SC	23.8						
GEO-13	0.5-1	SM/SP-SM	2.8						
	1.5-2	SM	9.5						
	2.5-3	SM	11.9				0.0	54.7	45.3
	3.5-4	CL/CH	21.6						
GEO-15	0.5-1	CL	10.4						
	1.5-2	CH	24.7						
	2.5-3	CH	21.5	54	17	37	0.0	18.5	81.5
	3.5-4	CH	23.5						
GEO-16	0.5-1	CL	12.0						
	1.5-2	CL	15.1						
	2.5-3	ML	9.7						
	3.5-4	ML	16.5				0.0	29.6	70.4
GEO-20	0.5-1	CL	12.3	45	16	29	0.0	24.4	75.6
	1.5-2	CL	16.9						
	2.5-3	SP/SP-SM	3.5						
	3.5-4	SP-SM	4.3						
SUB-Opt1-01	0.5-1	CL-ML/CL	4.4						
	1.5-2	CL	6.2				0.0	45.3	54.7
	2.5-3	CL	13.0						
	3.5-4	CL	13.2						
		Number of Tests	28	3	3	3	7	7	7
		Minimum	2.8	39	14	25	0.0	18.5	27.0
		Maximum	24.7	54	17	37	4.0	69.0	81.5
		Average	13.5	46	16	30	0.6	40.0	59.4
		Standard Deviation	6.4	6	1	5	1.4	16.5	17.5

1) Approximate Soil Types - see boring logs for full description.

Table 4b

Summary of Laboratory Test Results from Thermal Resistivity and CBR Bulk Samples

Sample Location				Atterberg Limits[1]			Grain Size Analysis		
Investigation ID	Depth	USCS Classification	Moisture Content [%]	Liquid Limit [%]	Plastic Limit [%]	Plasticity Index [%]	Percent Gravel [%]	Percent Sand [%]	Percent Fines [%]
GEO-06A	0.5-1.5	CL	7.3	31	15	16	0.0	49.2	50.8
GEO-10A	3-5	SM	13.6	21	N/A	NP	0.3	74.1	25.6
GEO-15	3-5	CH	20.7	50	16	34	0.0	21.3	78.7
GEO-16	3-5	CL	13.9	34	21	13	0.0	33.6	66.4
GEO-18	0.5-1.5	CL	5.0	40	16	24	0.1	25.9	74.0
GEO-20	3-5	SC	22.0	37	14	23	0.0	54.0	46.0
SUB-Opt1-01	3-5	CL	10.2	29	17	12	0.0	24.3	75.7
		Minimum	5.0	21	14	12	0.0	21.3	25.6
		Maximum	22.0	50	21	34	0.3	74.1	78.7
		Average	13.2	35	17	20	0.1	40.3	59.6
		Standard Deviation	5.9	8	2	8	0.1	18.0	18.1

1) The sample selected for testing from investigation location GEO-10A was determined to be nonplastic, denoted as NP, and therefore the plastic limit could not be determined.

Table 5

Summary of Standard Proctor Density Test Results

Investigation ID	Depth [ft]	USCS Classification	Maximum Dry Density [pcf]	Optimum Moisture Content [%]
GEO-06A	0.5-1.5	CL	112.6	16.1
GEO-10A	3-5	SM	114.7	13.9
GEO-15	3-5	CH	104.2	21.0
GEO-16	3-5	CL	103.7	18.0
GEO-18	0.5-1.5	CL	108.2	17.2
GEO-20	3-5	SC	98.6	23.5
SUB-Opt1-01	3-5	CL	109.6	16.8
		Average	107.4	18.1
		Standard Deviation	5.6	3.2
		Minimum	98.6	13.9
		Maximum	114.7	23.5

Table 6

Summary of Average Undrained Shear Strength Correlated from CPT Data

Investigation ID	Average Undrained Shear Strength [psf] (depth interval below grade)[1]	
	0.5 to 10 feet	10 to 20 feet
GEO-01	2,071	1,348
GEO-02	2,942	2,467
GEO-03	4,004	2,019
GEO-04A	1,159	1,902
GEO-05	1,773	843
GEO-06A	1,315	1,640
GEO-07A	1,986	1,721
GEO-08A	1,866	1,058
GEO-09	2,496	1,396
GEO-10A	*	1,424
GEO-11	2,419	1,319
GEO-12	2,381	2,325
GEO-13	2,536	1,195
GEO-14	*	2,301
GEO-15	1,859	1,448
GEO-16	*	*
GEO-17	2,389	2,661
GEO-18	1,967	2,717
GEO-19	1,743	2,120
GEO-20	1,326	2,042
GEO-21	1,683	2,842
SUB-Opt1-01	3,299	1,708

1) Summary neglects the upper 0.5 feet of loose material near the ground surface.

2) * Indicates less than 25% cohesive data was correlated from the CPT sounding over the summary interval.

Table 7

Summary of Average Drained Friction Angle Correlated from CPT Data

Investigation ID	Average Drained Friction Angle [degrees] (depth interval below grade)[1,2]	
	0.5 to 10 feet	10 to 20 feet
GEO-01	41.3	*
GEO-02	40.5	*
GEO-03	41.4	40.4
GEO-04A	*	*
GEO-05	*	*
GEO-06A	*	*
GEO-07A	41.9	39.9
GEO-08A	40.0	38.8
GEO-09	42.6	*
GEO-10A	39.7	*
GEO-11	*	42.7
GEO-12	*	*
GEO-13	42.4	*
GEO-14	40.8	*
GEO-15	41.1	*
GEO-16	41.8	42.3
GEO-17	41.8	*
GEO-18	40.3	*
GEO-19	*	*
GEO-20	*	39.3
GEO-21	41.7	*
Sub-Opt1-01	41.6	*

1) Summary neglects the upper 0.5 feet of loose material near the ground surface.

2) * Indicates less than 25% granular data was correlated from the CPT sounding over the summary interval.

Table 8

Summary of Analytical Test Results

Investigation ID	Sample Depth [ft]	USCS Classification	Laboratory Electrical Resistivity			Field Electrical Resistivity[1] [ohm-cm]	Laboratory Chemical Testing			
			As-Received Moisture Content [%]	As-Received Resistivity [ohm-cm]	Saturated Condition Resistivity [ohm-cm]		pH	Chloride[2] [mg/kg]	Sulfate[2] [mg/kg]	Oxidation-Reduction Potential [mV]
GEO-05	1-4	CL/CH	21.6	114	98	--	8.9	3,060	3,890	211
GEO-06A	0.5-1.5	CL	7.3	9,350	835	--	7.5	113	581	259
GEO-10A	1-4	SM/SC-SM	17.1	177	134	--	8.0	3,590	10,300	265
GEO-13	1-4	SM/SC-SM	9.7	2,010	980	--	9.0	221	332	186
GEO-15	1-4	CL/CH	22.9	131	114	--	8.1	2,980	4,870	238
GEO-16	1-4	SM	8.2	10,100	1,270	--	8.9	48.8	184	157
GEO-20	1-4	SM	3.6	98,000	5,800	365	8.7	7.1	25.5	188
SUB-Opt1-01	3-5	CL	11.8	1,300	330	751	8.1	839	2,360	252
		Average	12.8	15,148	1,195	558	8.4	1,357	2,818	220
		Standard Deviation	7.0	33,729	1,914	--	0.5	1,566	3,535	40
		Minimum	3.6	114	98	365	7.5	7.1	25.5	157
		Maximum	22.9	98,000	5,800	751	9.0	3,590	10,300	265

1) Electrical resistivity measurement from field Wenner array tests. Minimum measurement from "a"-spacings less than 20-feet.

Table 9

Summary of Thermal Resistivity Test Results

Investigation ID[1]	Depth [ft]	USCS Classification	Sample As Received Moisture Content [%]	Proctor Test[1]		Thermal Resistivity Test			
				Maximum Dry Density [pcf]	Optimum Moisture Content [%]	Sample Compaction[2] [%]	Sample Initial Moisture Content [%]	Initial Condition Thermal Resistivity [°C-cm/W]	Dry Condition Thermal Resistivity [°C-cm/W]
GEO-10A	3-5	SM	13.6	114.7	13.9	85	13.6	85	254
GEO-15	3-5	CH	20.7	104.2	21	85	20.9	98	245
GEO-16	3-5	CL	13.9	103.7	18.0	85	13.7	111	369
GEO-20	3-5	SC	22.0	98.6	23.5	85	22.3	108	389
SUB-Opt1-01	3-5	CL	10.2	109.6	16.8	85	10.1	120	303
						95	12.0	84	209
		Average[2]	16.1	106.2	18.6	--	16.1	104	312
		Standard Deviation[2]	5.0	6.2	3.7	--	5.2	13	65
		Minimum[2]	10.2	98.6	13.9	--	10.1	85	245
		Maximum[2]	22.0	114.7	23.5	--	22.3	120	389

1) From standard Proctor moisture density curve, ASTM D698.

2) Statistical summary corresponds to samples that were reconstituted to approximately 85% of maximum standard Proctor density at as-received moisture content.

Table 10

Summary of Soil Temperature Measurements from Test Pits

Investigation ID	Soil Temperature (Measurement Depth) [°F]					
	Ambient	1 ft	2 ft	3 ft	4 ft	5 ft
GEO-05	107.9	98.1	89.4	88.2	86.2	85.1
GEO-10A	106.0	91.8	90.0	88.0	86.7	85.1
GEO-13	108.0	90.9	89.2	87.4	86.2	84.7
GEO-15	110.0	87./8	85.1	83.3	82.2	80.4
GEO-16	99.9	89.1	86.9	86.0	83.1	82.0
GEO-20	102.0	88.0	85.5	82.4	81.0	79.7
SUB-Opt1-01	108.0	85.6	85.1	83.5	82.8	81.1
Average		90.6	87.3	85.5	84.0	82.6
Standard Deviation		4.3	2.2	2.4	2.3	2.3
Minimum		85.6	85.1	82.4	81.0	79.7
Maximum		98.1	90.0	88.2	86.7	85.1

Table 11

Summary of Soil Moisture Content Results from Test Pits

Investigation ID	Sample Depth[1] [ft]	USCS Classification	Moisture Content [%]
GEO-05	0.5-1	CL	5.5
	1.5-2	CL	15.8
	2.5-3	CL	19.6
	3.5-4	CL	20.3
GEO-10A	0.5-1	SC-SM/SC	10.7
	1.5-2	SC-SM/SC	13.7
	2.5-3	SM/SC-SM	15.3
	3.5-4	SC	23.8
GEO-13	0.5-1	SM/SP-SM	2.8
	1.5-2	SM	9.5
	2.5-3	SM	11.9
	3.5-4	CL/CH	21.6
GEO-15	0.5-1	CL	10.4
	1.5-2	CH	24.7
	2.5-3	CH	21.5
	3.5-4	CH	23.5
GEO-16	0.5-1	CL	12.0
	1.5-2	CL	15.1
	2.5-3	ML	9.7
	3.5-4	ML	16.5
GEO-20	0.5-1	CL	12.3
	1.5-2	CL	16.9
	2.5-3	SP/SP-SM	3.5
	3.5-4	SP-SM	4.3
SUB-Opt1-01	0.5-1	CL-ML/CL	4.4
	1.5-2	CL	6.2
	2.5-3	CL	13.0
	3.5-4	CL	13.2

Average	13.5
Standard Deviation	6.5
Minimum	2.8
Maximum	24.7

1) Grab sample collected from each test pit at the corresponding depth intervals

Table 12

Summary of Geophysical Test Results from Seismic CPT

Geotechnical ID	Weighted Average Shear Wave Velocity [ft/sec]	Average Shear Wave Velocity per ASCE 7-16, Vs_{100} [ft/sec]
GEO-16	565	--
Sub-Opt1-01	817	787

Table 13
Summary of California Bearing Ratio Test Results

Investigation ID	Depth [ft]	USCS Classification	Proctor Test[1]		CBR Value[2,3] (Optimum Moisture Content)
			Maximum Dry Density [pcf]	Optimum Moisture Content [%]	
GEO-06A	0.5-1.5	CL	112.6	16.1	7.6
GEO-18	0.5-1.5	CL	108.2	17.3	5.4

1) From standard Proctor moisture density curve (ASTM D698).

2) Test specimens were soaked for a period of 4 days before CBR tests were performed.

3) Test samples were recompacted to 95 percent of standard Proctor maximum dry density at optimum moisture content.

Table 14
Summary of Axial Parameters for Preliminary Pile Design

Design Profile	Depth to Top of Layer [ft]	Depth to Bottom of Layer [ft]	Ultimate Skin Friction[1,2,3] [lb/ft^2]	Ultimate End Bearing[2,3] [lb/ft^2]
Design Profile 1	0	1	100	N/A
	1	4	150	N/A
	4	7	350	20,000
	7	20	750	20,000
Design Profile 2	0	0.5	100	N/A
	0.5	4	400	N/A
	4	20	750	30,000
Design Profile 3	0	0.5	100	N/A
	0.5	4	400	N/A
	4	7	750	30,000
	7	20	750	40,000

1) Soil resistance should be neglected in the design frost depth (Section 5.7) and scour zone/depth. For scour refer to the project hydrology report to be completed by others.

2) A factor of safety of 2.0 is recommended for use in design for a wind loading condition. See Section 6.3.

3) For preliminary pile design only. Based on the results of CPT soundings and test pits.

Table 15
Summary of Lateral Parameters for Preliminary Pile Design

Design Profile	Depth to Top of Layer[1] [ft]	Depth to Bottom of Layer[1] [ft]	LPile Soil Type	Effective Unit Weight [pcf]	Friction Angle [deg.]	Cohesion [psf]	p-y Modulus[2], *k* [lb/in^3]	Strain Factor, e50
Design Profile 1	0.0	0.5	Sand (Reese)	100	30	--	10	--
	0.5	2.0	Sand (Reese)	100	36	--	200	--
	2.0	5.0	Stiff Clay w/o Free Water	100	--	1500	--	0.007
	5.0	20.0	Stiff Clay w/o Free Water	42.6	--	1500	--	0.007
Design Profile 2	0.0	0.5	Sand (Reese)	100	30	--	10	--
	0.5	5.0	Sand (Reese)	100	36	--	200	--
	5.0	20.0	Stiff Clay w/o Free Water	42.6	--	2000	--	0.006
Design Profile 3	0.0	0.5	Sand (Reese)	100	30	--	10	--
	0.5	5.0	Sand (Reese)	100	36	--	200	--
	5.0	13.5	Sand (Reese)	42.6	36	--	125	--
	13.5	20.0	Stiff Clay w/o Free Water	42.6	--	1500	--	0.007

1) Soil resistance should be neglected in the scour zone/depth. Refer to the project hydrology report to be completed by others.

2) Nominal lateral soil resistance should be provided in the frost zone/depth. See Section 5.3.2.2.

3) Preliminary design groundwater level of 5 feet below existing grade.

4) For preliminary pile design only. Based on CPT sounding and test pit results.

Table 16

Summary of LPile Deflection Results

Design Profile	Loading Condition[1,2]				Pile Type	Embedment Depth [ft]	Lateral Deflection [in.]	
	Column	Shear Load [lb]	Moment [in.-lb]	Axial Load [lb]			Pile Head	Ground Surface
Design Profile 1	Interior	2,200	36,300	1,500	W8x13	9.0	1.7	0.6
	Exterior	2,500	34,200	2,000	W8x15	10.0	1.5	0.6
Design Profile 2	Interior	2,200	36,300	1,500	W8x13	8.0	1.6	0.5
	Exterior	2,500	34,200	2,000	W8x13	8.5	1.7	0.6
Design Profile 3	Interior	2,200	36,300	1,500	W8x13	7.5	1.6	0.5
	Exterior	2,500	34,200	2,000	W8x13	8.5	1.7	0.6

1) A 5-ft reveal height was used in the LPile analyses.

2) The analyses was performed with loading in the strong-axis direction.

Table 17

Recommended Aggregate Base Thickness for Preliminary Road Section Design

Traffic Condition	Max Rut Depth [in.]	Percent Compaction [%][1]	Recommended Aggregate Thickness [in.][2]		
			Unreinforced	Geotextile	Geogrid[3]
Maintenance[4]	1.5	95	7	6	6
Emergency Vehicle Access[5]	1.5	95	21	12	6
	3.0		7	6	6
Construction[6]	1.5	95	13	9	6
	3.0		7	6	6

*Note that road cross-section should be of adequate thickness to compensate for topsoil stripping and promote positive drainage away from the road. This table is intended to provide the recommended values for use in the preliminary road section design. It is the responsibility of the civil designer to determine final road cross-section.

1) Percent compaction of standard Proctor maximum dry density (ASTM D698).

2) A minimum of 6 inches is recommended for road section design.

3) Biaxial geogrid or equivalent (assumed aperture stability modulus = 0.65 m-N/degree).

4) Maintenance traffic condition assumes a maximum axle load of 2.5 kips, a tire pressure of 65 psi, and 50,000 axle passes.

5) Emergency Vehicle traffic condition assumes a maximum axle load of 33.5 kips, a tire pressure of 100 psi, and 250 axle passes.

6) Construction traffic condition assumes a maximum axle load of 10 kips, a tire pressure of 100 psi, and 3,000 axle passes.

Figures

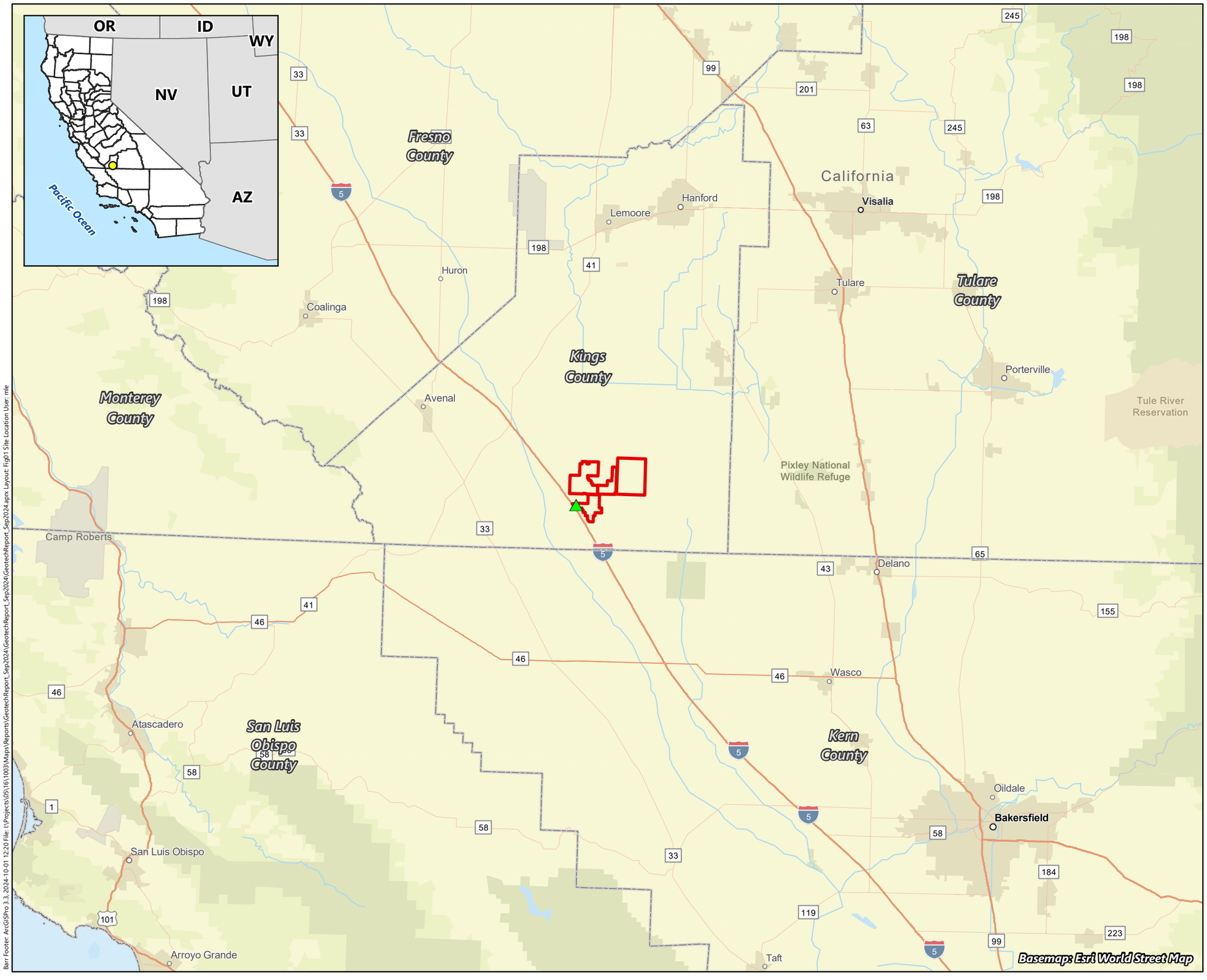
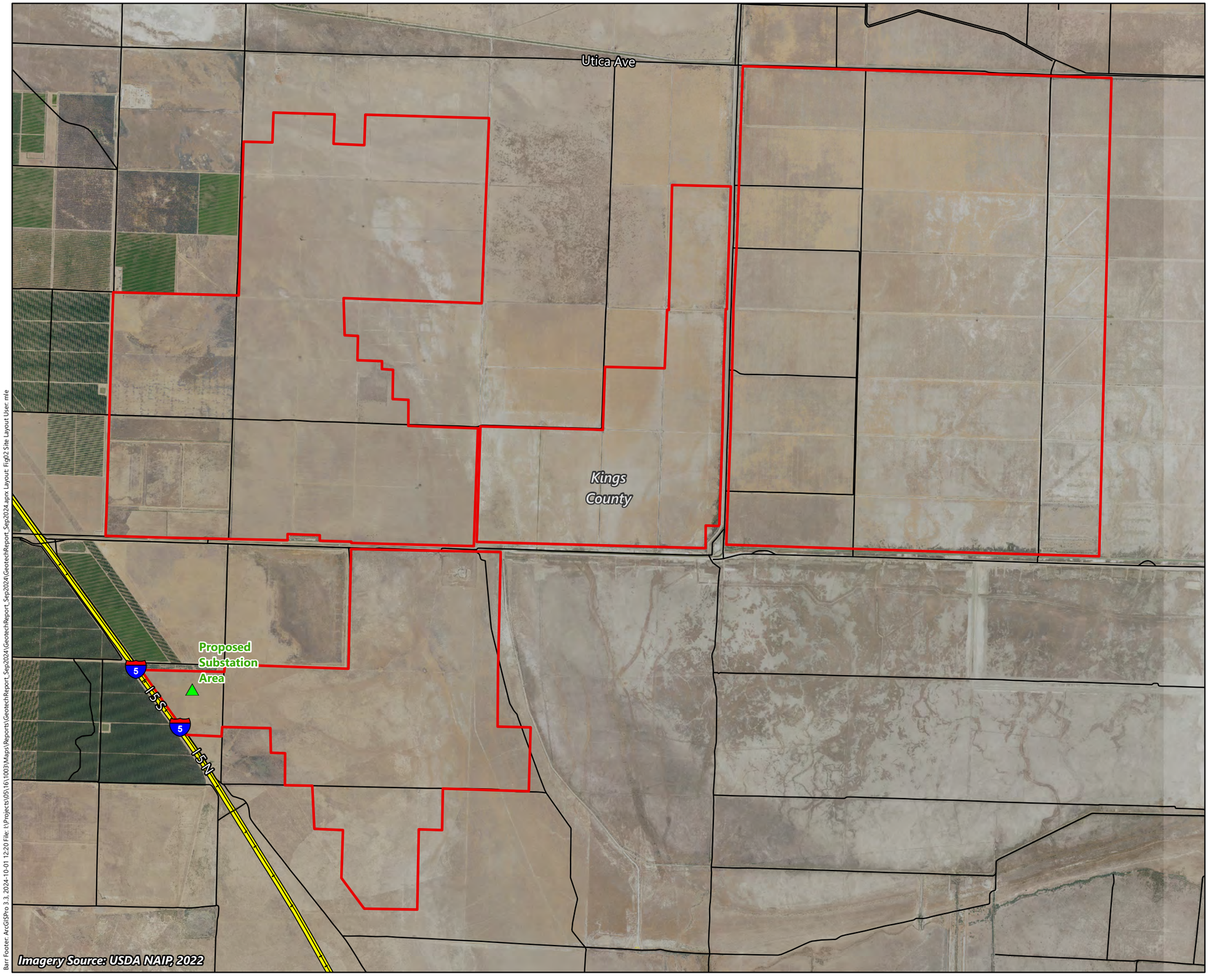


Figure 1

SITE LOCATION
Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer. ArcGISPro 3.3. 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig01 Site Location User: mle



Proposed Substation Location

Project Boundaries

Interstate

Public Road

N

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 2

SITE LAYOUT

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig02 Site Layout User: mle

Proposed Substation Location

Project Boundaries

Kings County

Proposed Substation Area

Topography Source: USGS 24K DRG

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 3

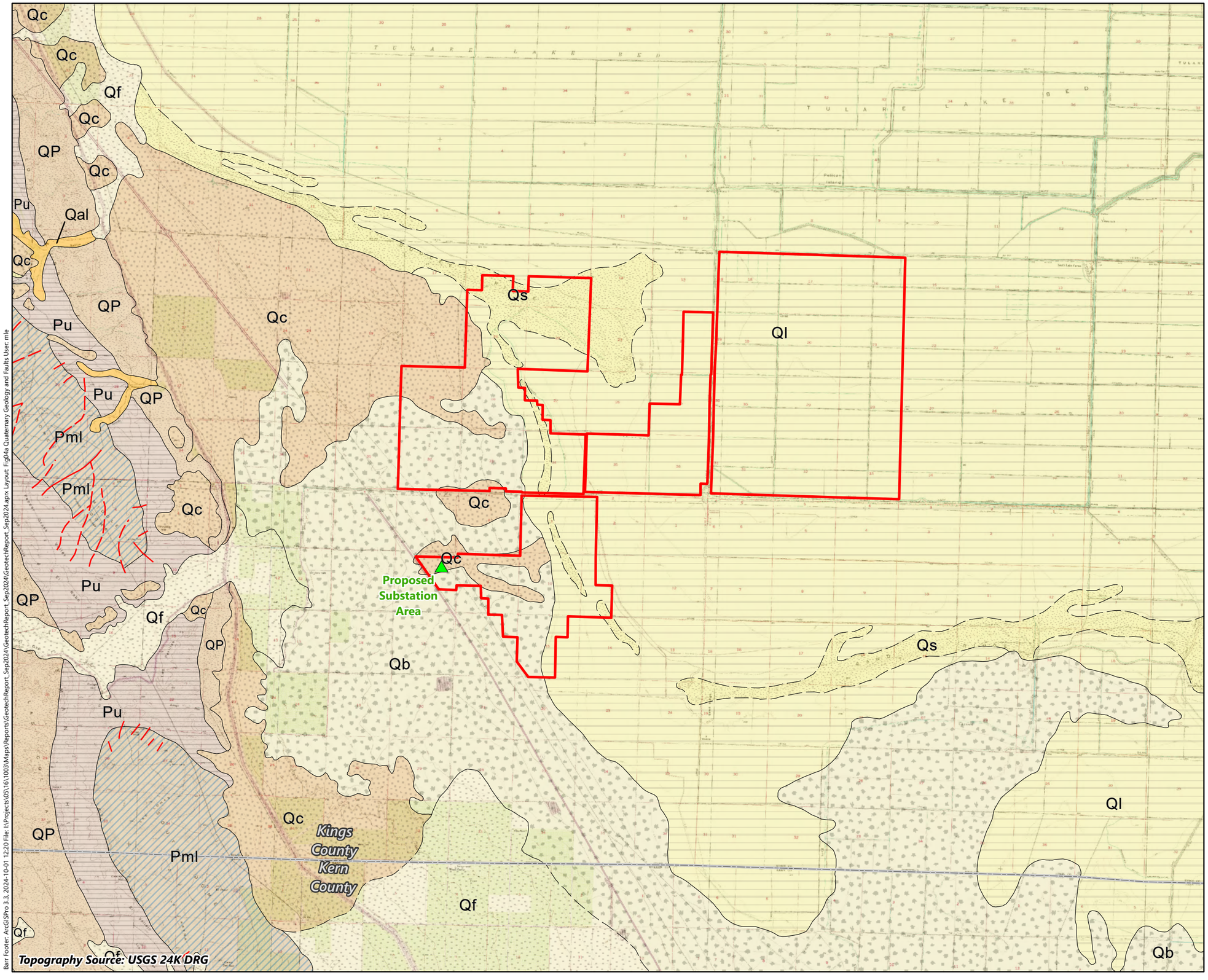
SITE TOPOGRAPHY

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig03 Site Topography User: mle



Barr Footer. ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig04a Quaternary Geology and Faults User: mle

Proposed Substation Location

Project Boundaries

Contacts And Faults

contact, approx. located

contact, well located, certain

fault, aprox. located

fault, well located, certain

Geologic Map Units

Qs - Recent dune sand

Qal - Recent alluvium

Qb - Great Valley basin deposits

Ql - Quaternary lake deposits

Qf - Quaternary nonmarine terrace deposits

Qc - Pleistocene nonmarine

QP - Pliocene-Pleistocene nonmarine sedimentary deposits

Pu - Upper pliocene marine sedimentary rocks

Pml - Middle and/or lower Pliocene marine sedimentary rocks

Jenkins and Smith. Geologic Map of California, 1:250,000. Bakersfield Sheet. California Geologic Survey. 1964.

Miles

1.5 0 1.5

Kilometers

3 0 3

Figure 4a

QUATERNARY GEOLOGY AND FAULTS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Proposed Substation Location

Project Boundaries

Quaternary Fault

Quaternary Fault and Fold Database of the United States. USGS Earthquake Hazards Program. Accessed 9/27/2024. Available online at: https://www.usgs.gov/programs/earthquake-hazards/faults

Miles

6 0 6

Kilometers

10 0 10

Figure 4b

REGIONAL QUATERNARY FAULTS

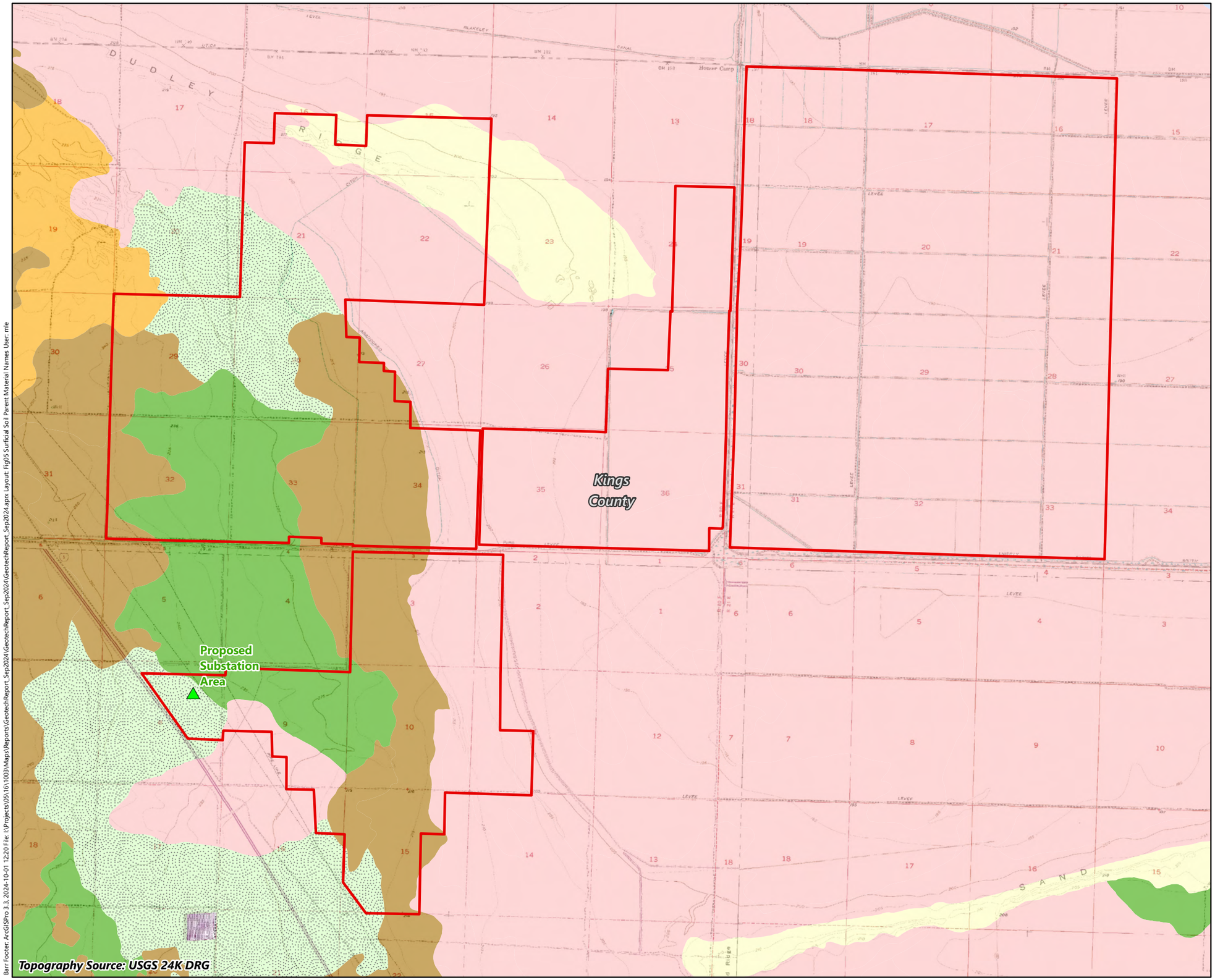
Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Basemap: Esri World Topographic Map and Esri World Hillshade

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig04b Regional Quaternary Faults User: mle



Proposed Substation Location

Project Boundaries

Surficial Soil Parent Material Name

- eolian deposits derived from igneous and sedimentary rock
- alluvium derived from calcareous sedimentary rock
- alluvium derived from granite
- alluvium derived from igneous and sedimentary rock
- alluvium derived from sandstone
- alluvium derived from sedimentary rock
- lacustrine deposits derived from igneous and sedimentary rock

Soil Survey Staff, NRCS USDA. Soil Survey Geographic (SSURGO) Database. Available online at: https://sdmdataaccess.sc.egov.usda.gov Accessed 3/23/2024.

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 5

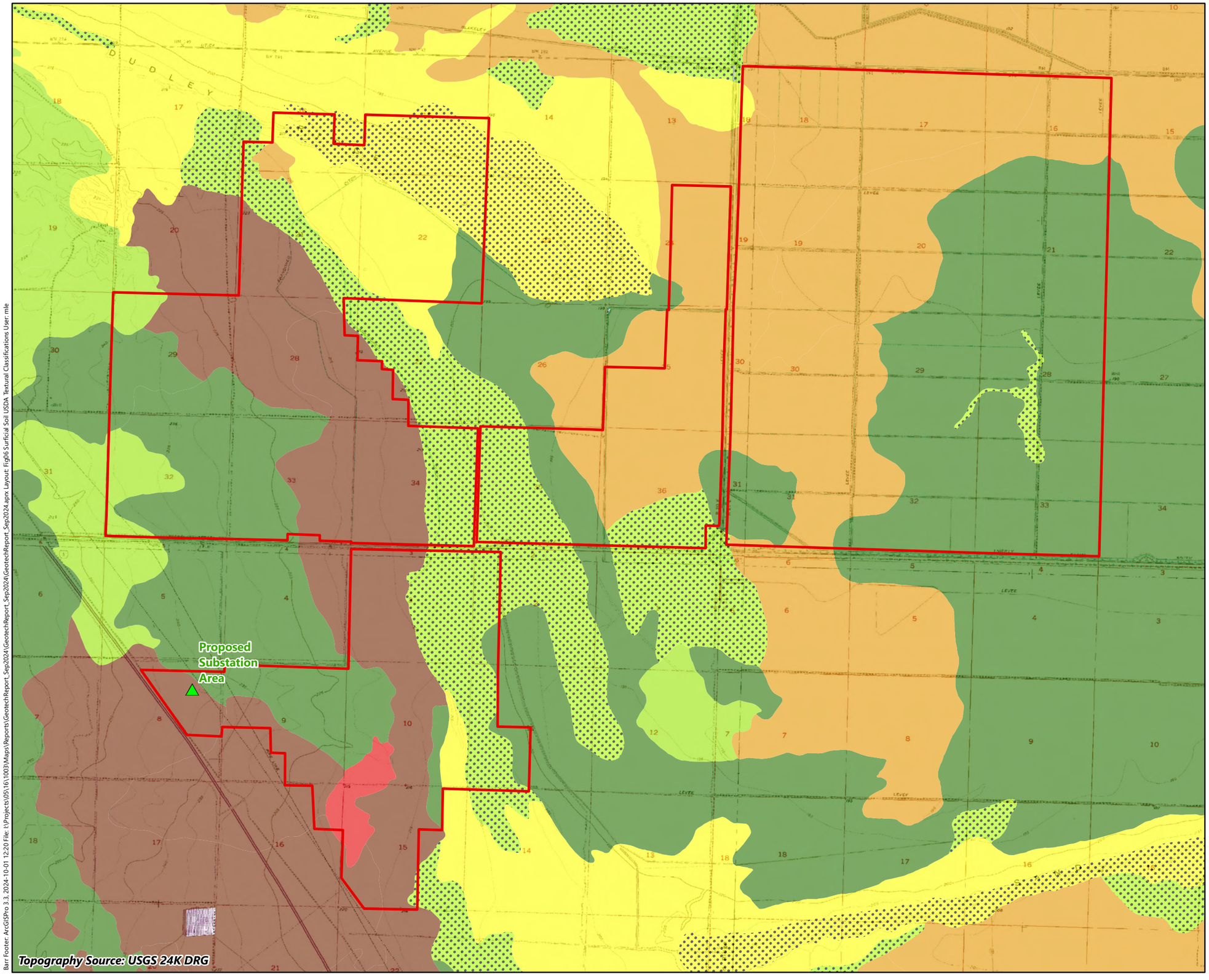
SURFICIAL SOIL PARENT MATERIAL NAMES

Rex Solaire Solar Project

Terra-Gen LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig05 Surficial Soil Parent Material Names User: mle



Proposed Substation Location

Project Boundaries

Surficial Soil USDA Textural Classification

- Loamy sand
- Loamy fine sand
- Sandy loam
- Fine sandy loam
- Loam
- Clay loam
- Silty clay
- Clay

Soil Survey Staff, NRCS USDA. Soil Survey Geographic (SSURGO) Database. Available online at: https://sdmdataaccess.sc.egov.usda.gov Accessed 3/23/2024.

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 6

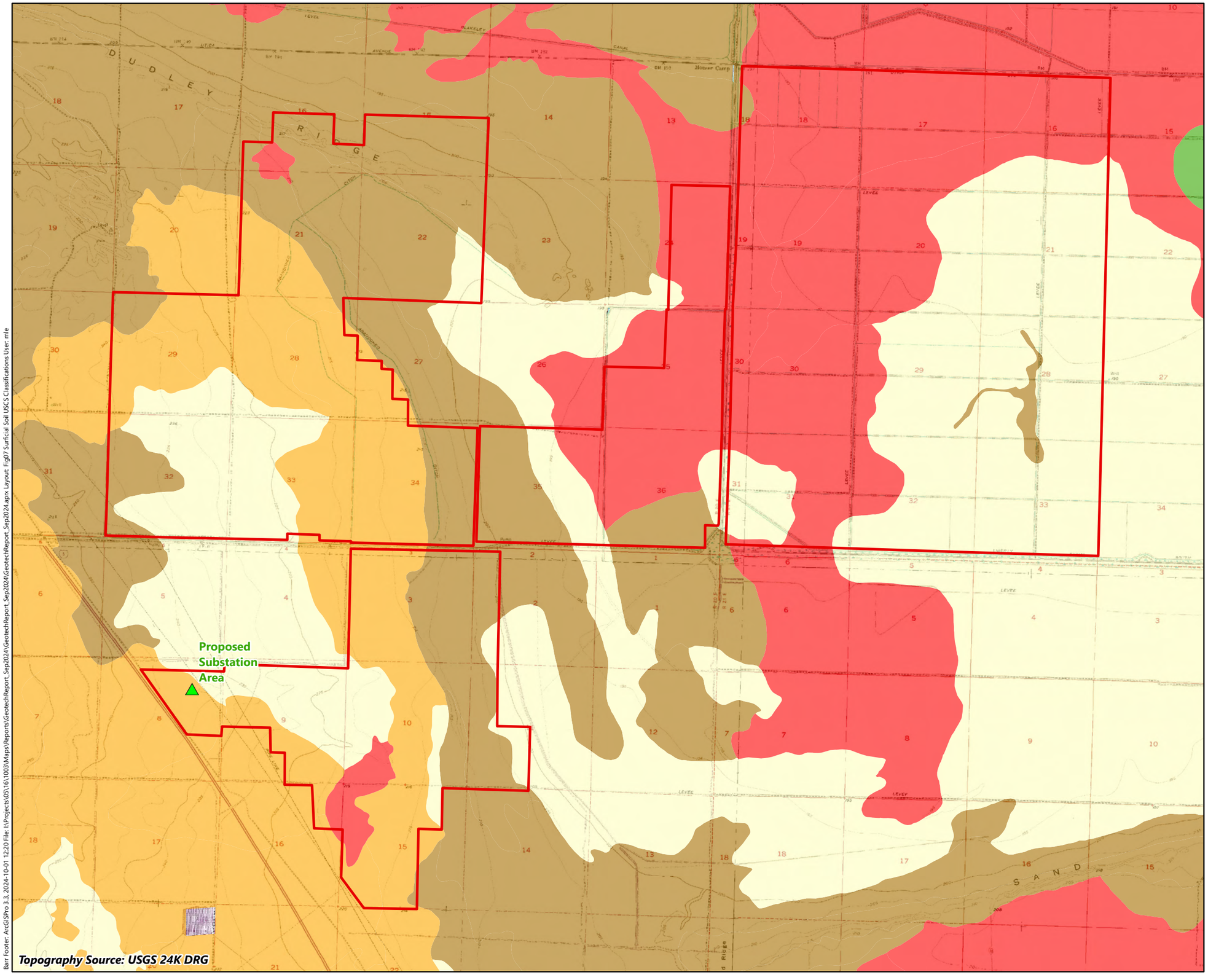
SURFICIAL SOIL USDA TEXTURAL CLASSIFICATIONS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\1611003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig06 Surficial Soil USDA Textural Classifications User: mle



Proposed Substation Location

Project Boundaries

Surficial Soil USCS Classification

- SM - Silty sand
- ML - Silt
- CL-ML - Silty clay
- CL - Lean clay
- CH - Fat clay

Soil Survey Staff, NRCS USDA. Soil Survey Geographic (SSURGO) Database. Available online at: https://sdmdataaccess.sc.egov.usda.gov Accessed 3/23/2024.

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 7

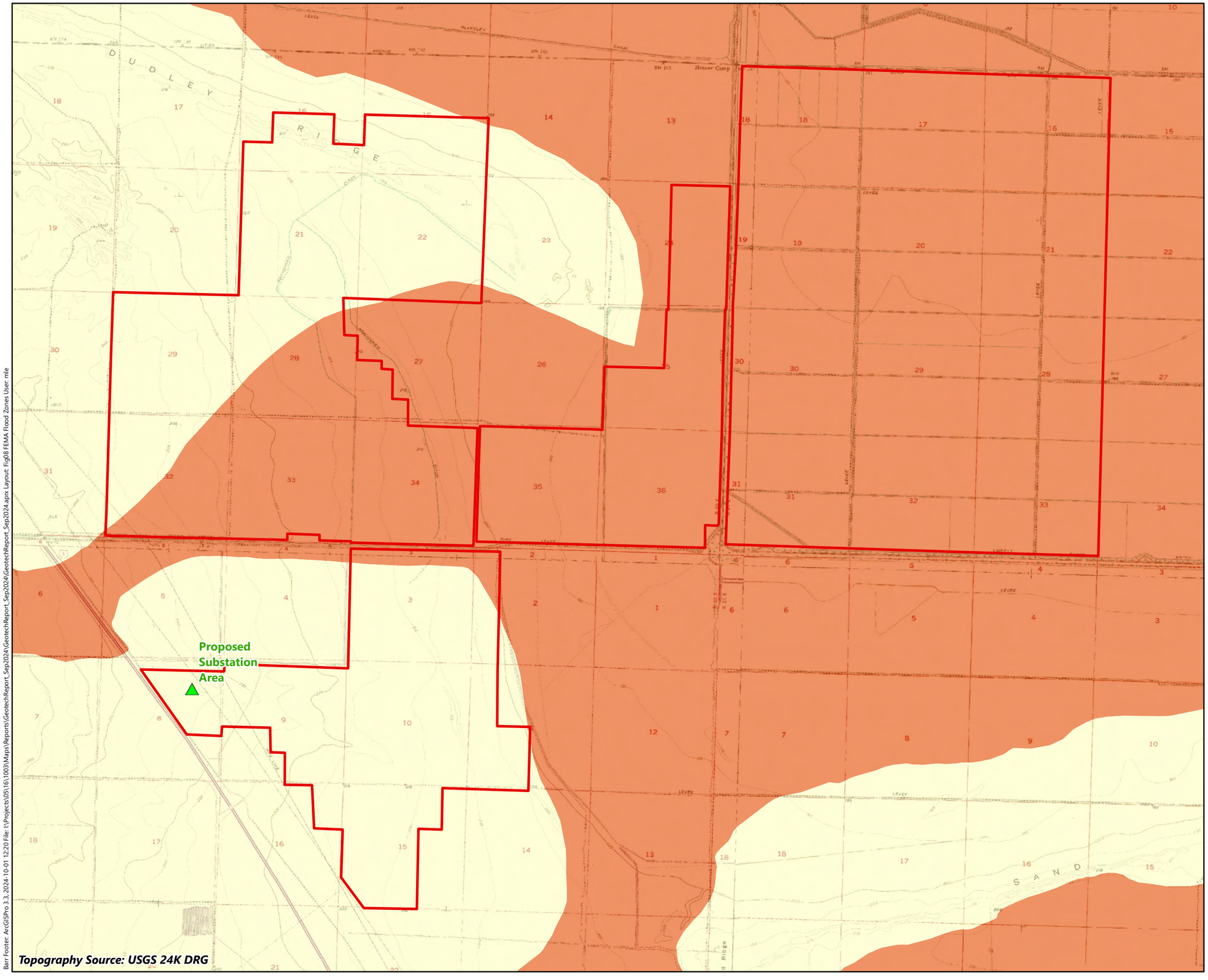
SURFICIAL SOIL USCS CLASSIFICATIONS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig07 Surficial Soil USCS Classifications User: mle



Proposed Substation Location

Project Boundaries

FEMA Flood Zone

Flood Zone A - 100 year-floodplain, 1% annual chance

Food Zone X - Area of Minimal Hazard

FEMA NFHL-County.
Product ID: NFHL_06031C
Latest Study Effective Date: 09/16/2015
Accessed 10/1/2024. Available online at:
https://msc.fema.gov/portal/home

Feet

4,000 0 4,000

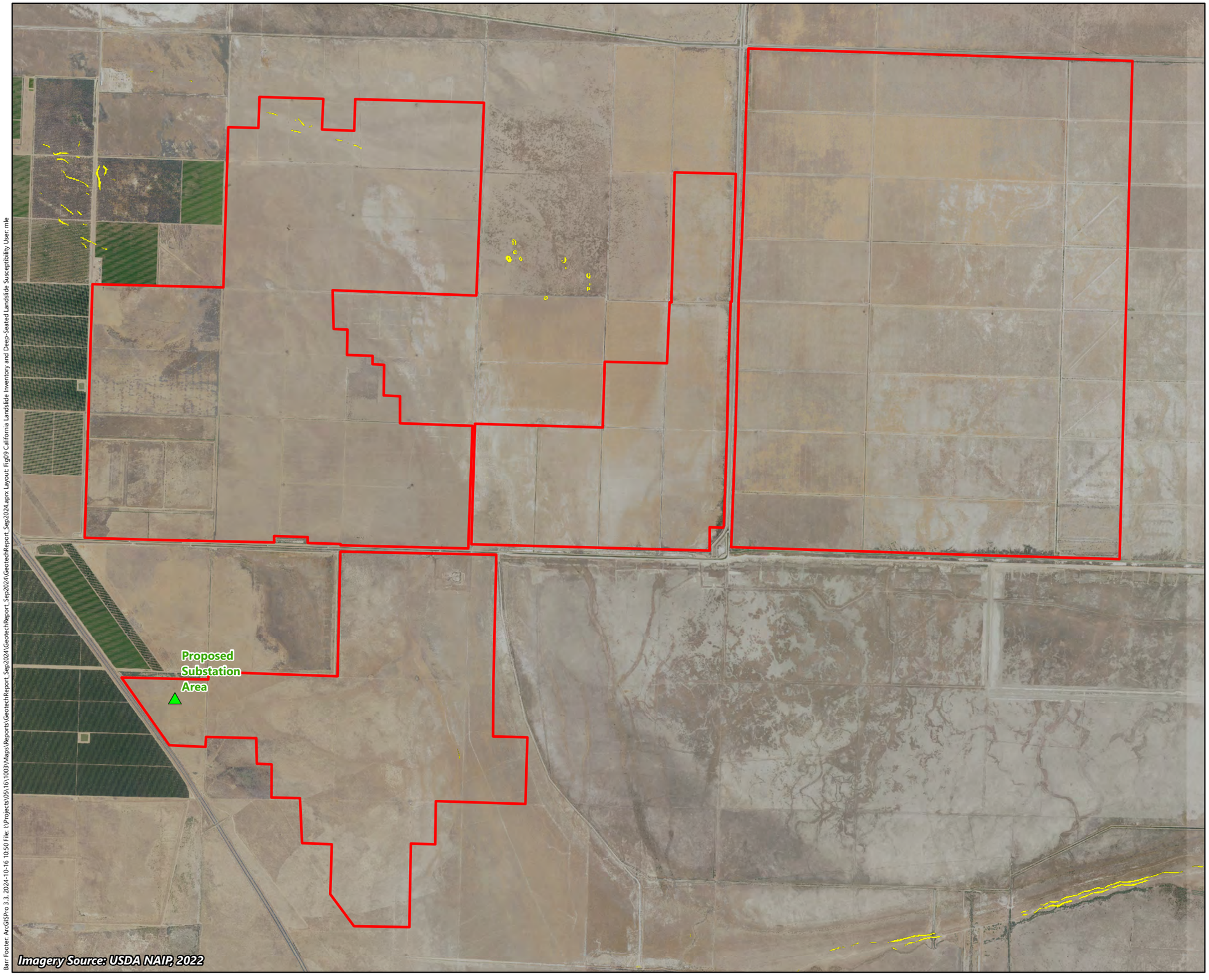
Meters

1,200 0 1,200

Figure 8

FEMA Flood Zones
Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-01 12:20 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig08 FEMA Flood Zones User: mle



Proposed Substation Location

Project Boundaries

Landslide Susceptibility Class

VII

Susceptibility to Deep-Seated Landslides in California 2011 C. J. Wills, F. G. Perez and C. I. Gutierrez. California Department of Conservation, California Geological Survey. Accessed online 10/15/2024 at https://gis.conservation.ca.gov/portal/home/item.html?id=87289025c11d4ba7ae65f0f472bf7c2d

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 9

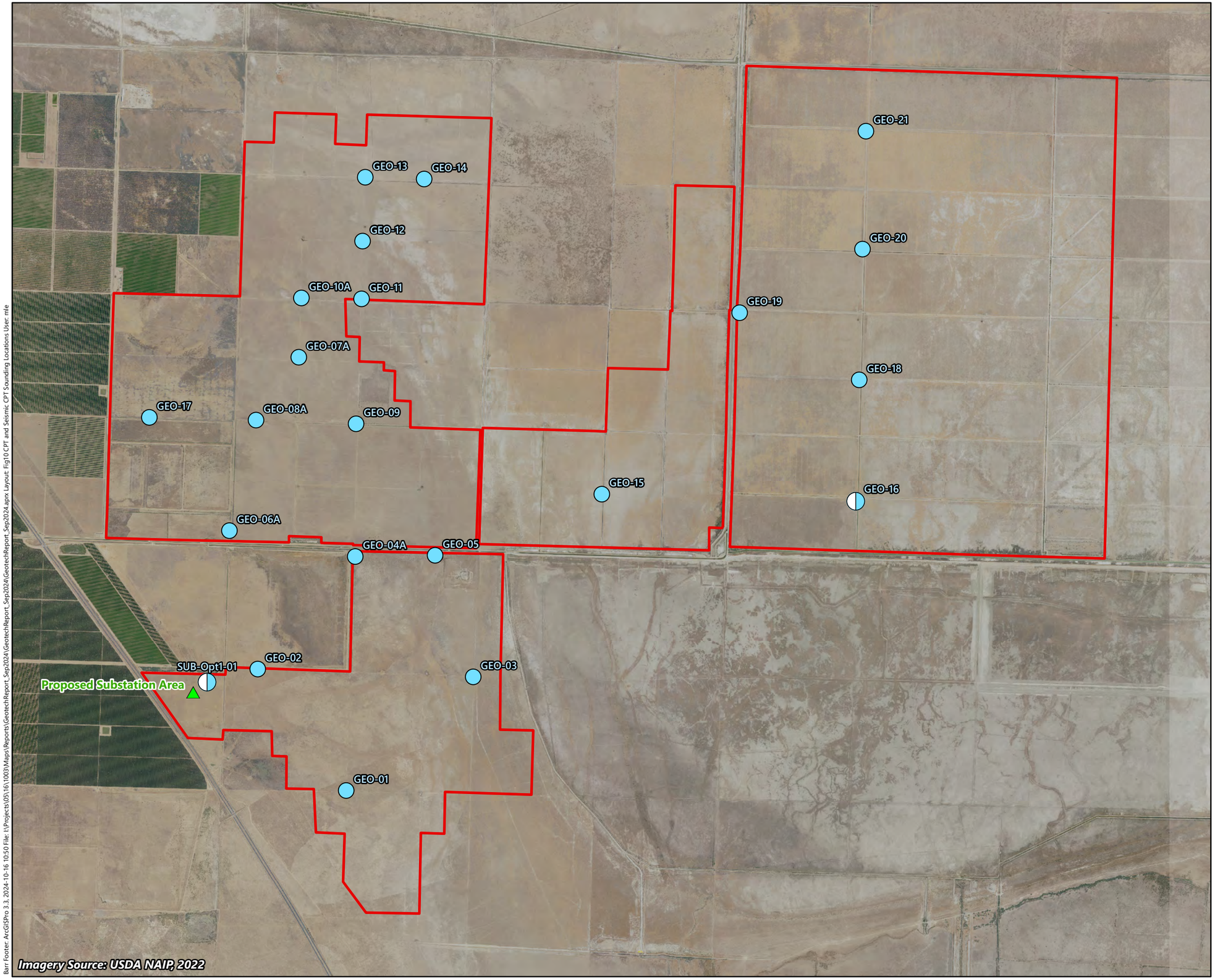
DEEP-SEATED LANDSLIDE SUSCEPTIBILTY

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05\16\10031\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig09 California Landslide Inventory and Deep-Seated Landslide Susceptibility User: mle



Proposed Substation Location

Project Boundaries

CPT and Seismic CPT Sounding Location

CPT Sounding Location

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 10

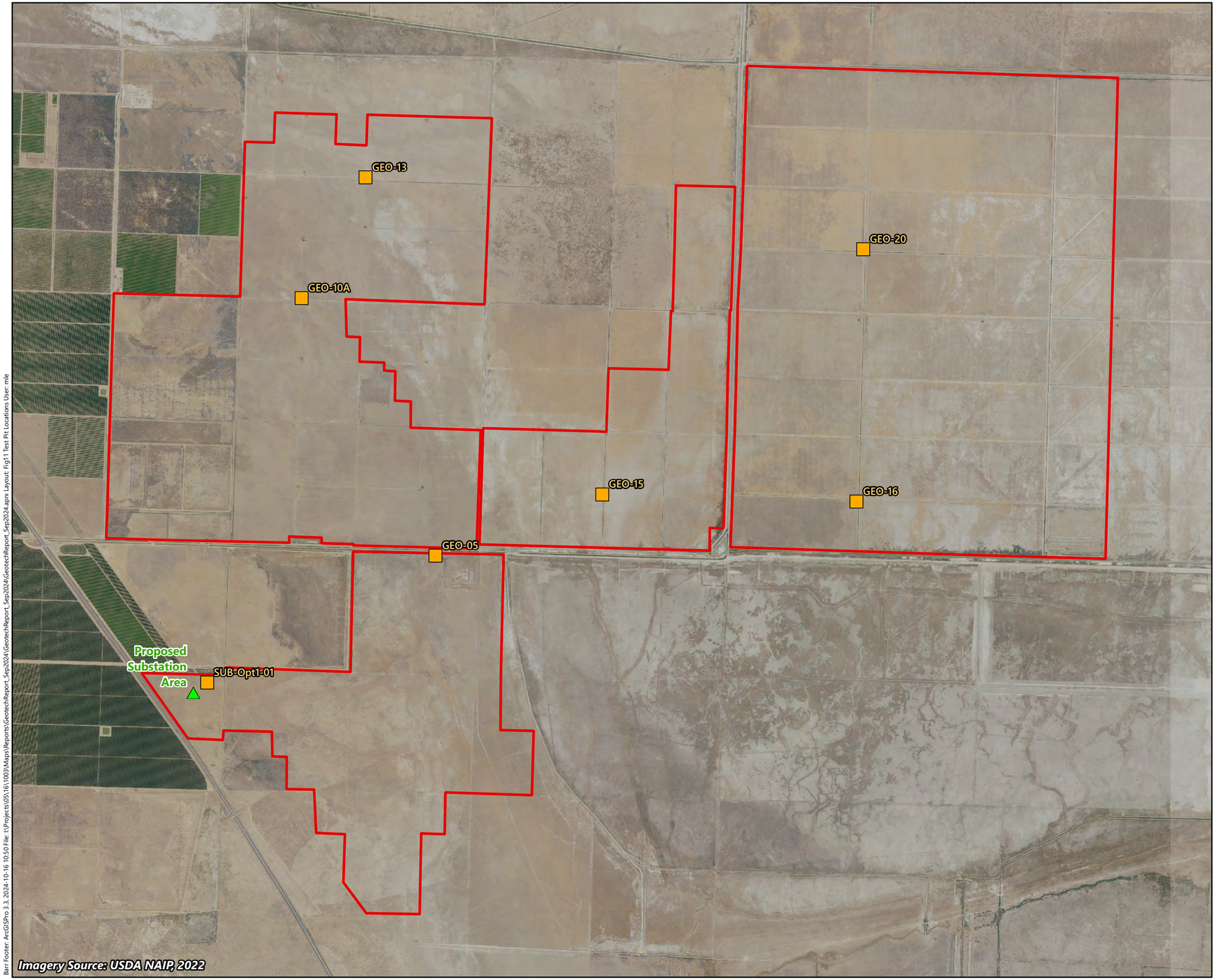
CPT AND SEISMIC CPT SOUNDING LOCATIONS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05116\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig10 CPT and Seismic CPT Sounding Locations User: mle



Proposed Substation Location
Project Boundaries
Test Pit Location

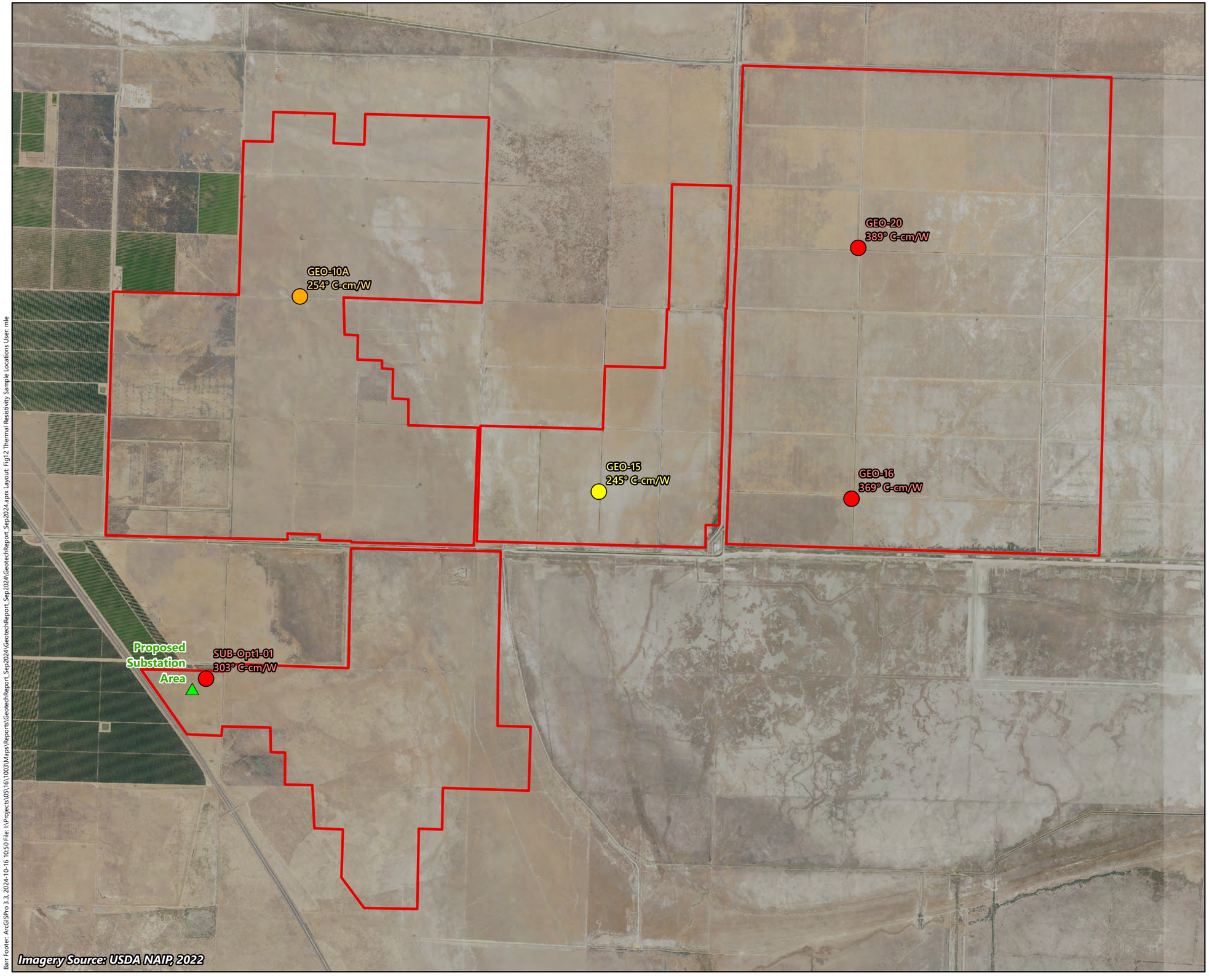
Feet
4,000 0 4,000

Meters
1,200 0 1,200

Figure 11

TEST PIT LOCATIONS
Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig11 Test Pit Locations User: mle



Substation Location

Project Boundaries

Thermal Resistivity Sample Location
Dry Condition Thermal Resistivity*

- <250° C-cm/W
- 250 -300° C-cm/W
- >300° C-cm/W

*Dry condition thermal resistivity values for bulk samples reconstituted to approximately 85% of maximum standard Proctor density at as-received moisture content.

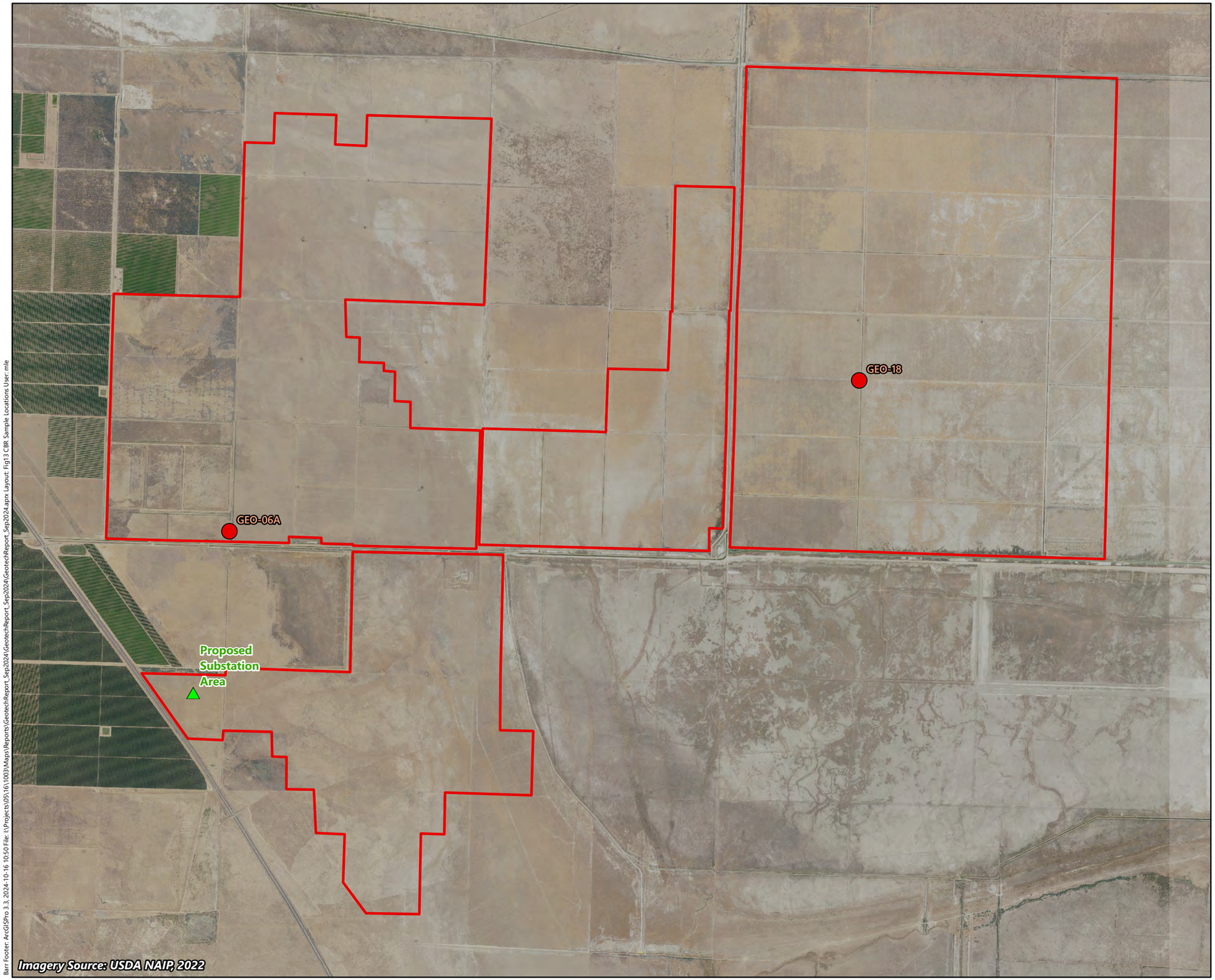
Feet
4,000 0 4,000

Meters
1,200 0 1,200

Figure 12

THERMAL RESISTIVITY SAMPLE LOCATIONS
Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig12 Thermal Resistivity Sample Locations User: mle



Proposed Substation Location

Project Boundaries

CBR Sample Location

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 13

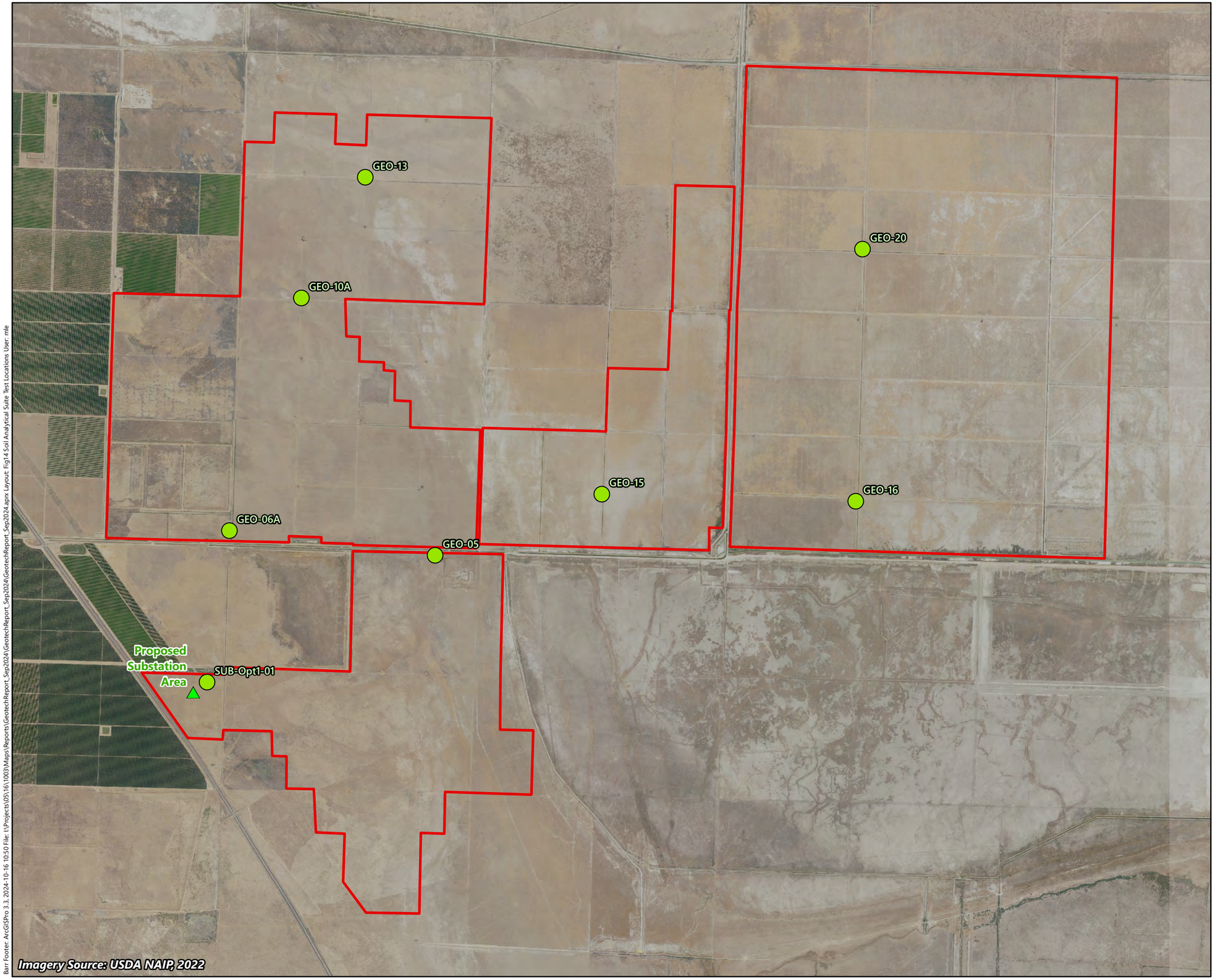
CBR SAMPLE LOCATIONS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig13 CBR Sample Locations User: mle



Proposed Substation Location

Project Boundaries

Soil Analytical Suite Test Location

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 14

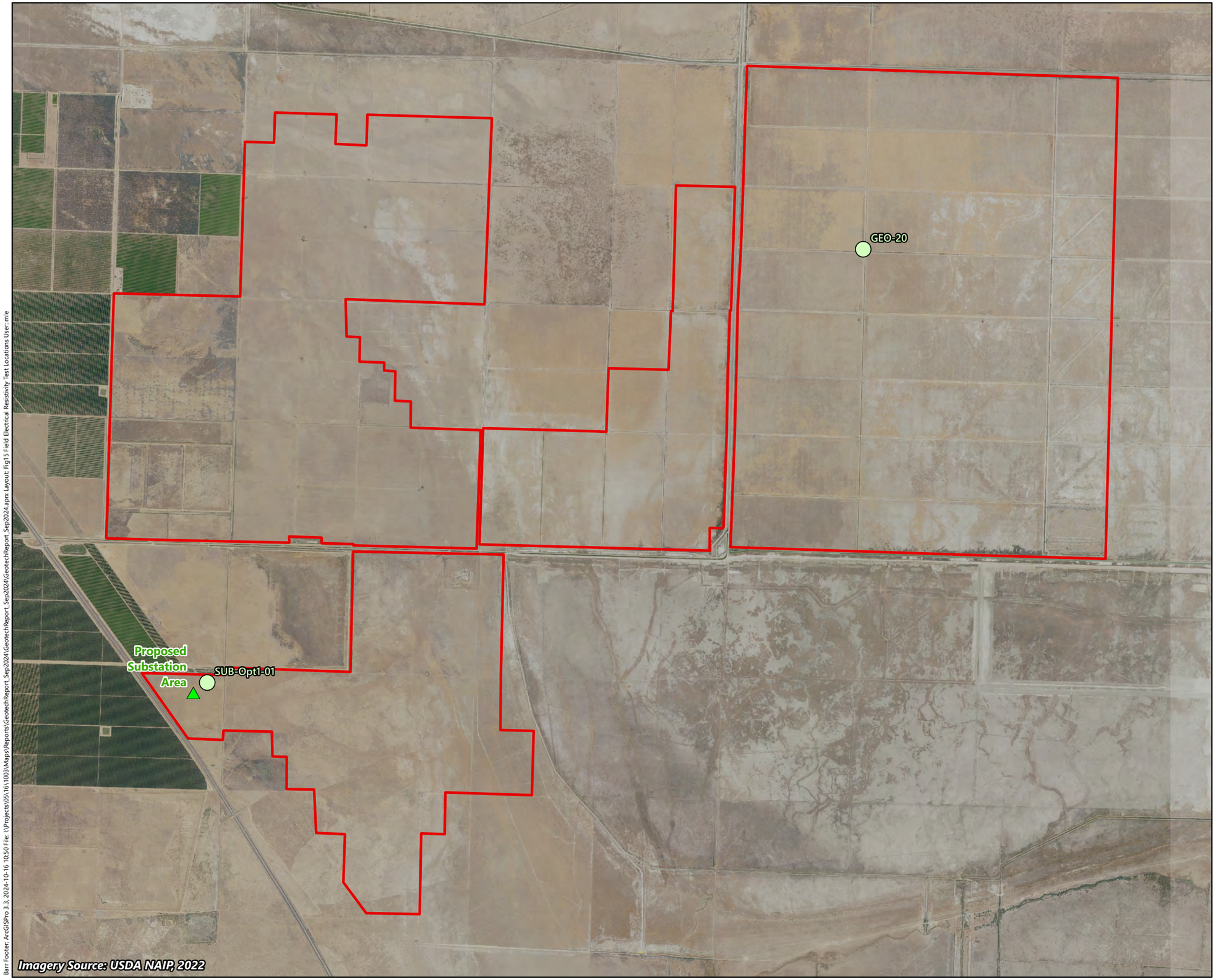
SOIL ANALYTICAL SUITE TEST LOCATIONS

Rex Solaire Solar Project

Terra-Gen Development Company, LLC

Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05116\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig14 Soil Analytical Suite Test Locations User: mle



Proposed Substation Location
Project Boundaries
Field Electrical Resistivity Test Location

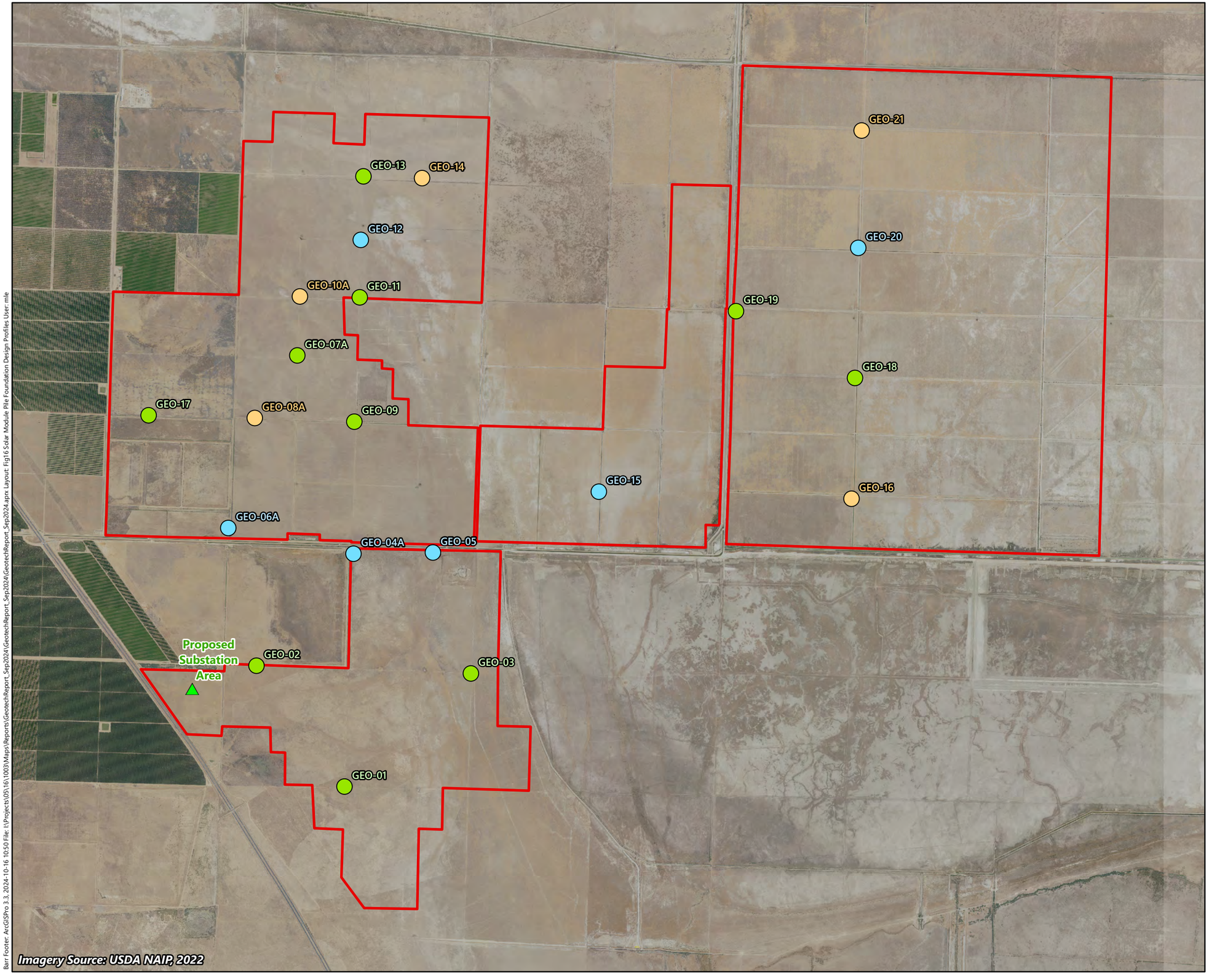
Feet
4,000 0 4,000

Meters
1,200 0 1,200

Figure 15

FIELD ELECTRICAL RESISTIVITY TEST LOCATIONS
Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05\16\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig15 Field Electrical Resistivity Test Locations User: mle



Proposed Substation Location

Project Boundaries

Solar Module Pile Foundation Design Profiles

- 1
- 2
- 3

Feet

4,000 0 4,000

Meters

1,200 0 1,200

Figure 16

SOLAR MODULE PILE FOUNDATION DESIGN PROFILES

Rex Solaire Solar Project
Terra-Gen Development Company, LLC
Kings County, California

Barr Footer: ArcGISPro 3.3, 2024-10-16 10:50 File: I:\Projects\05116\1003\Maps\Reports\GeotechReport_Sep2024\GeotechReport_Sep2024\GeotechReport_Sep2024.aprx Layout: Fig16 Solar Module Pile Foundation Design Profiles User: mle

Appendix A

Preliminary Geotechnical Investigation Summary

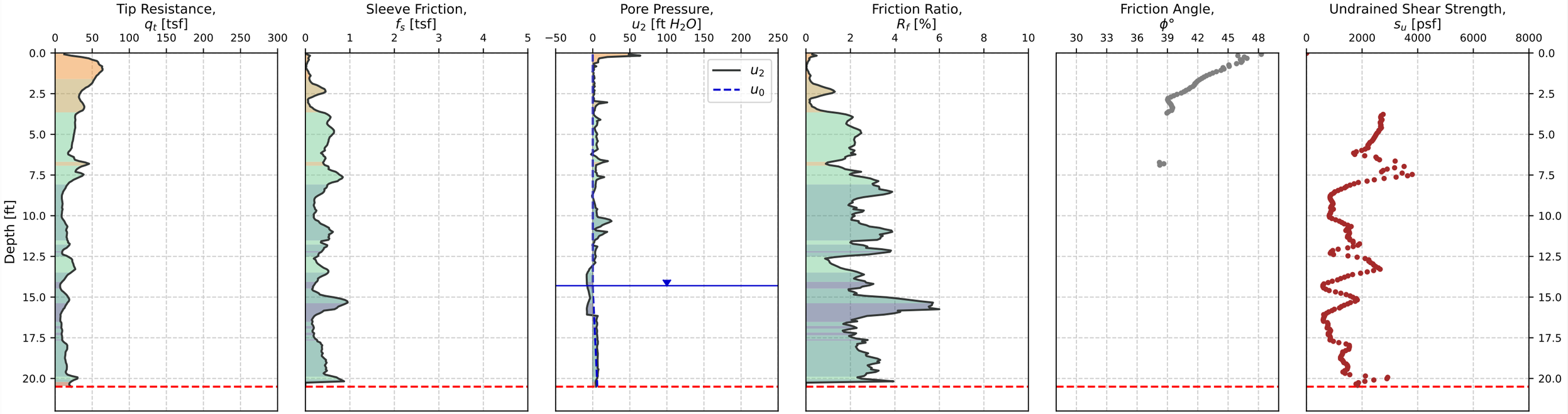
Appendix A

Preliminary Geotechnical Investigation Summary

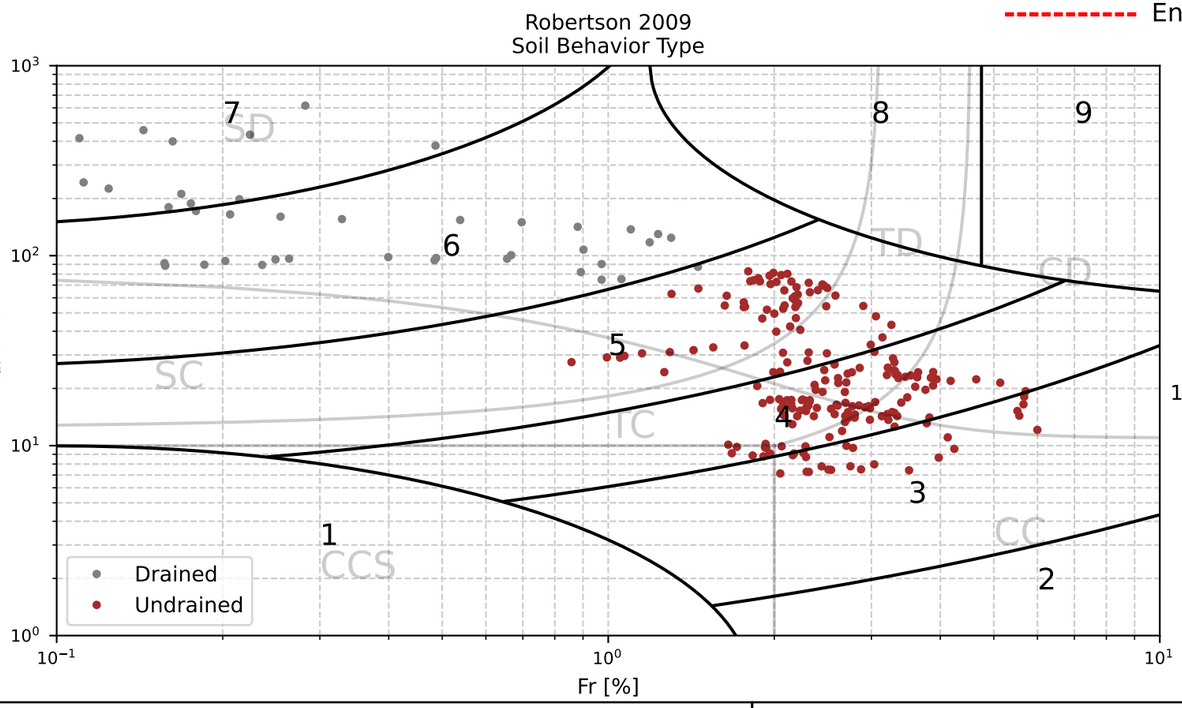
Investigation ID	Status/ Structure	Geographic Coordinates, WGS84		CPT Sounding		Seismic CPT	Test Pit	Thermal Resistivity	Field Electrical Resistivity	CBR	Analytical Testing Sample
		Latitude	Longitude	20 ft	100 ft						
GEO-01	Solar Array	35.84758	-119.80798	X							
GEO-02	Solar Array	35.86152	-119.82123	X							
GEO-03	Solar Array	35.86133	-119.79016	X							
GEO-04A	Solar Array	35.87504	-119.80765	X							
GEO-05	Solar Array	35.87546	-119.79615	X			X				X
GEO-06A	Solar Array	35.87765	-119.82588	X						X	X
GEO-07A	Solar Array	35.89822	-119.81661	X							
GEO-08A	Solar Array	35.89070	-119.82251	X							
GEO-09	Solar Array	35.89061	-119.80808	X							
GEO-10A	Solar Array	35.90518	-119.81648	X			X	X			X
GEO-11	Solar Array	35.90528	-119.80781	X							
GEO-12	Solar Array	35.91205	-119.80790	X							
GEO-13	Solar Array	35.91955	-119.80778	X			X				X
GEO-14	Solar Array	35.91954	-119.79928	X							
GEO-15	Solar Array	35.88318	-119.77239	X			X	X			X
GEO-16	Solar Array	35.88319	-119.73577	X		X	X	X			X
GEO-17	Solar Array	35.89071	-119.72567	X							
GEO-18	Solar Array	35.89744	-119.73576	X						X	
GEO-19	Solar Array	35.90491	-119.75326	X							
GEO-20	Solar Array	35.91278	-119.73581	X			X	X	X		X
GEO-21	Solar Array	35.92662	-119.73580	X							
SUB-Opt1-01	Substation	35.85982	-119.82847		X	X	X	X	X		X

Appendix B

CPT Sounding Logs



End of test or refusal



SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

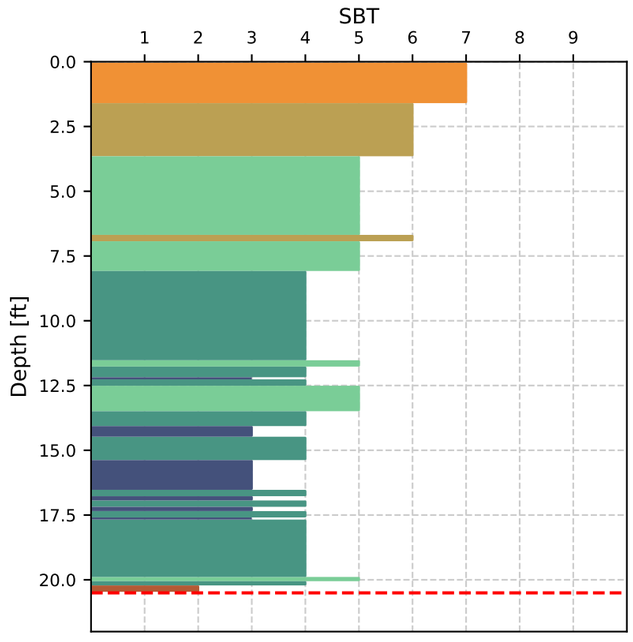
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



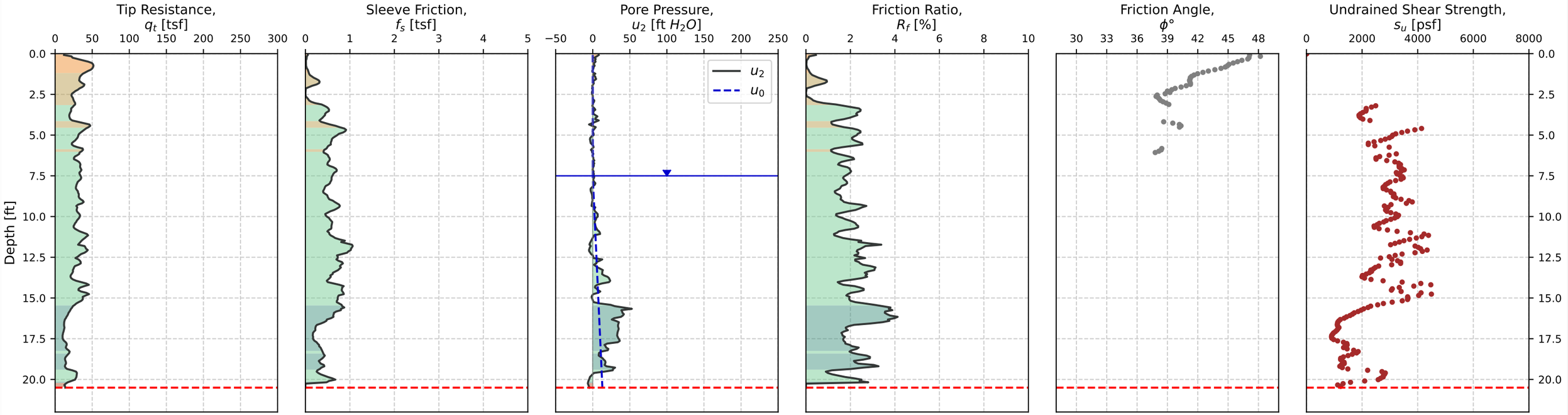
Sounding: GEO-01
Test Date: 2024-09-11
Long: -119.807983° W
Lat: 35.847578° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP01.COR

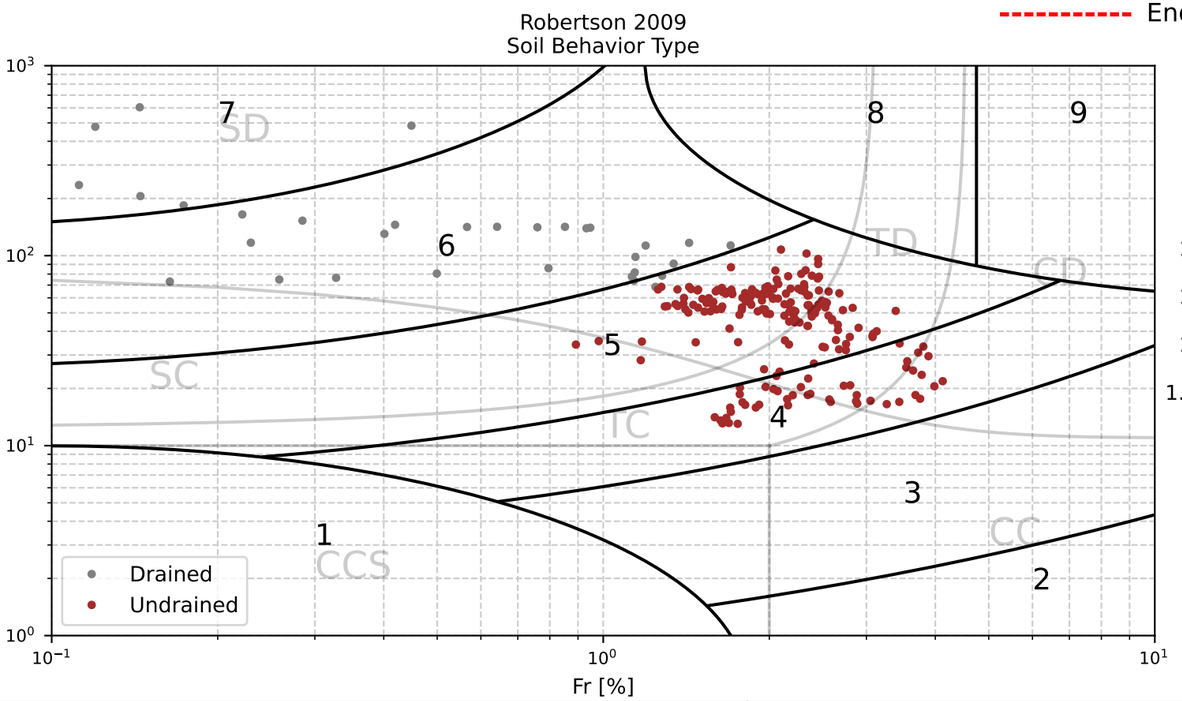
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

p:\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\Field\CPT\ConeTec_CPT_Tool\Deliverables\CPT_Plots\CPT_Plots_20241009.pdf





SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

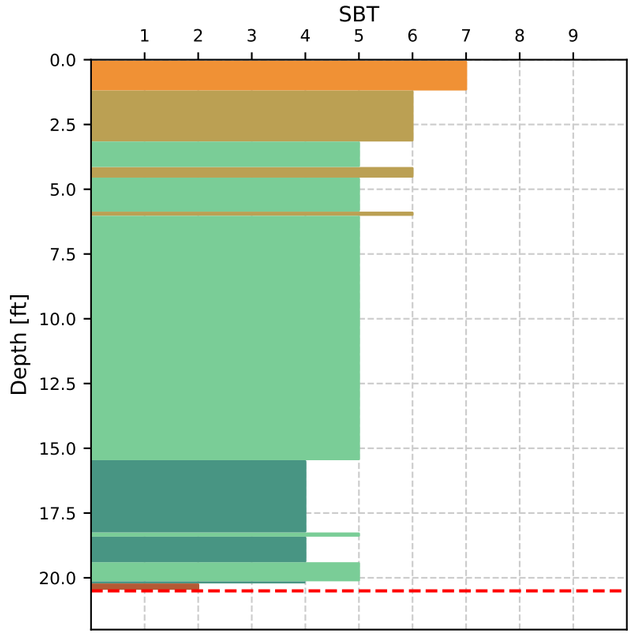
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



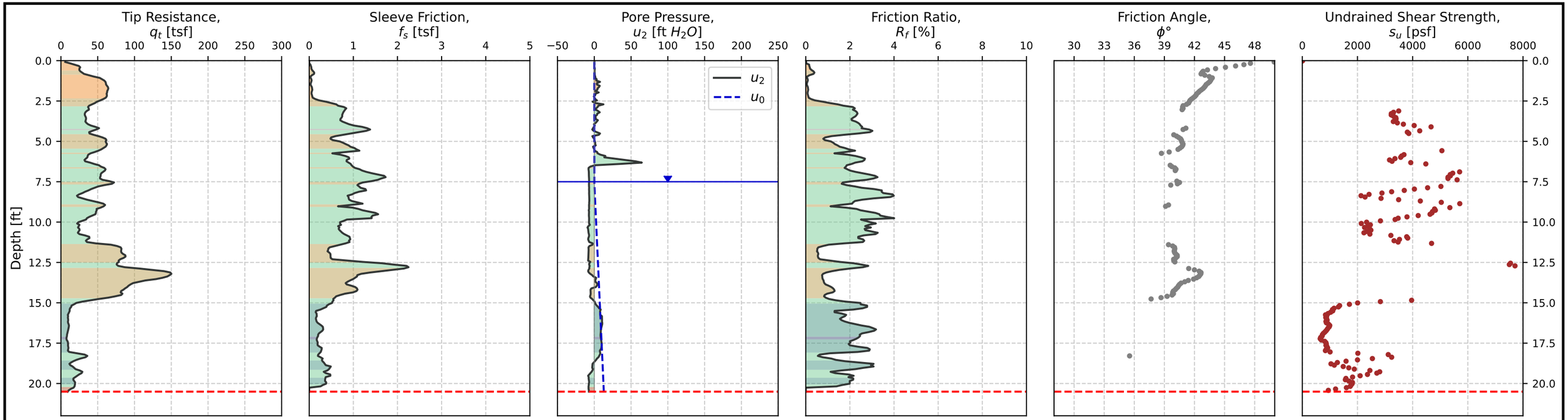
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Test Date: 2024-09-10
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Lat: 35.861521° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP02.COR

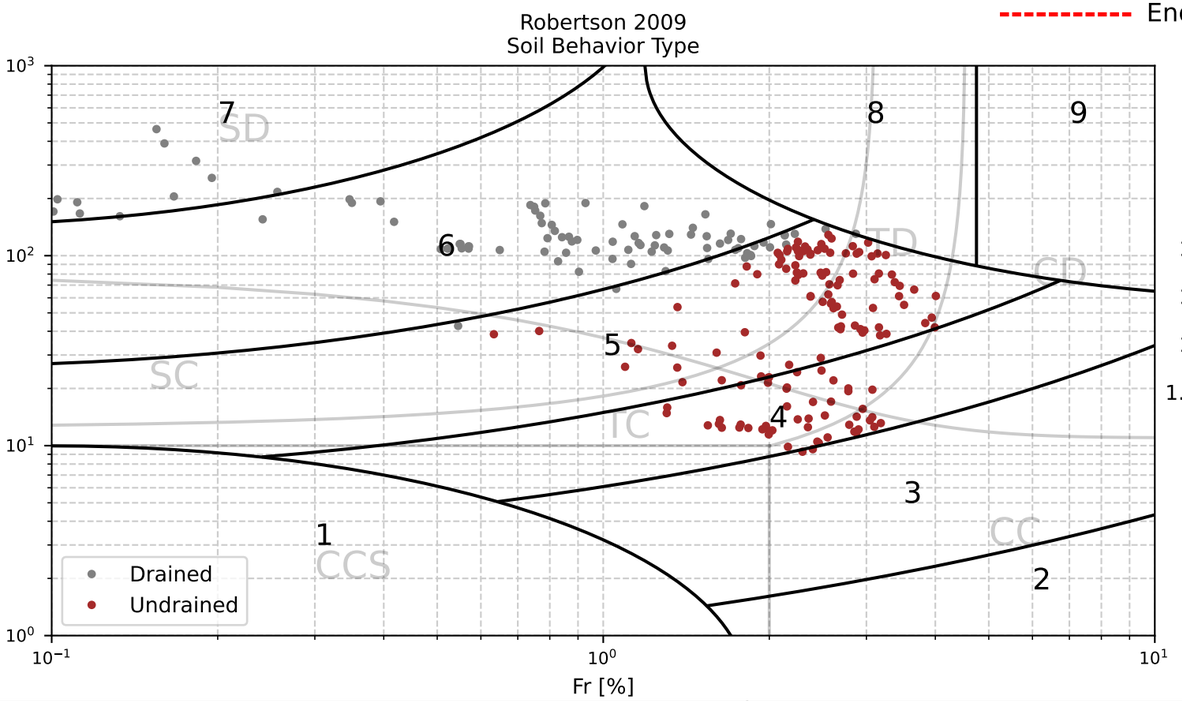
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

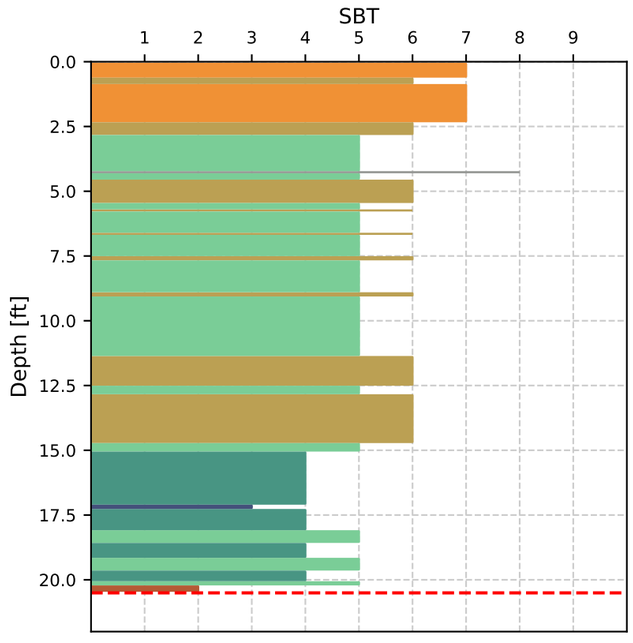
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



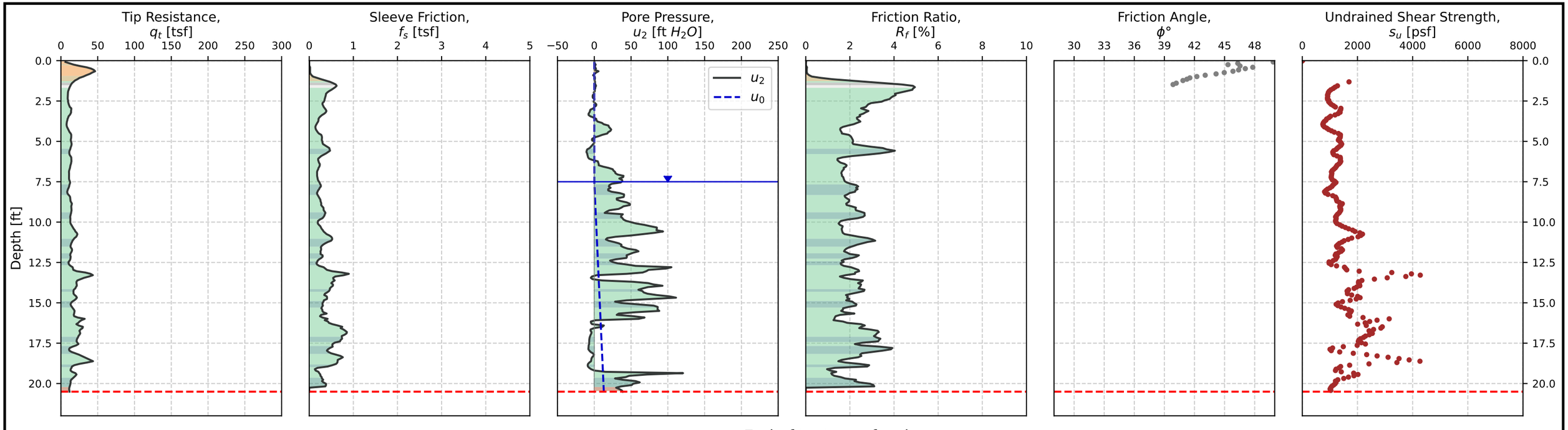
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Horiz. Datum: WGS 84
Native File: 24-56-28295_CP03.COR

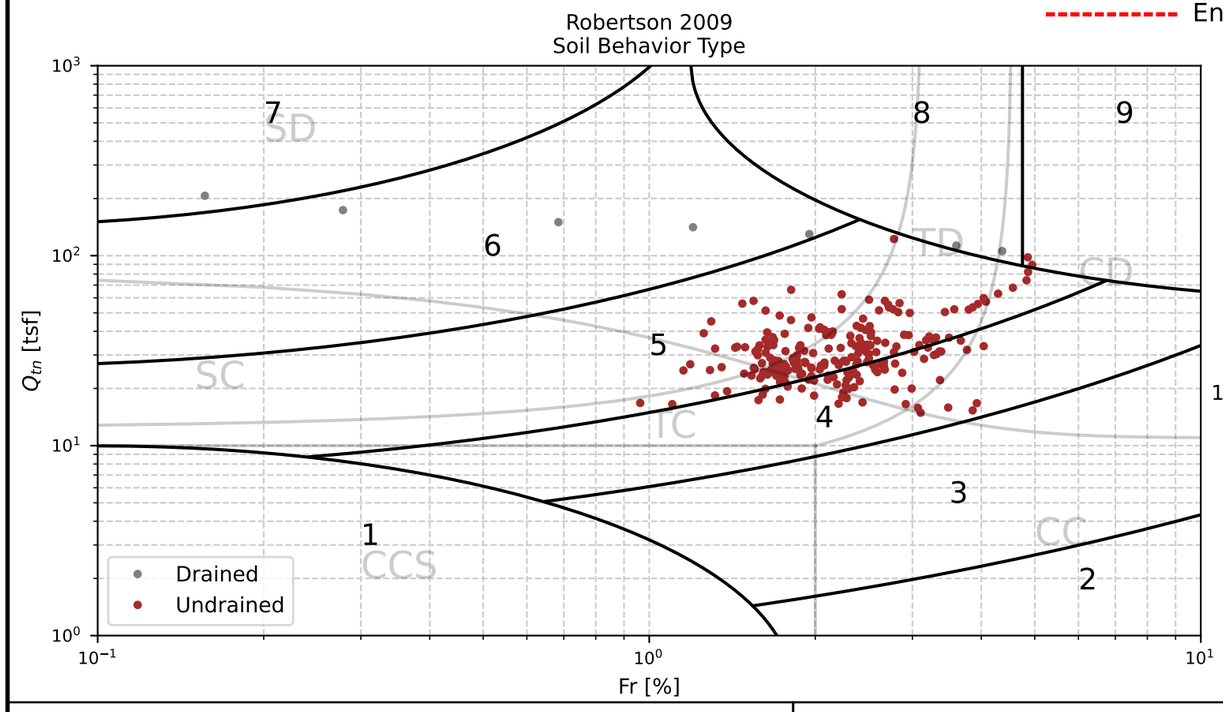
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

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$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

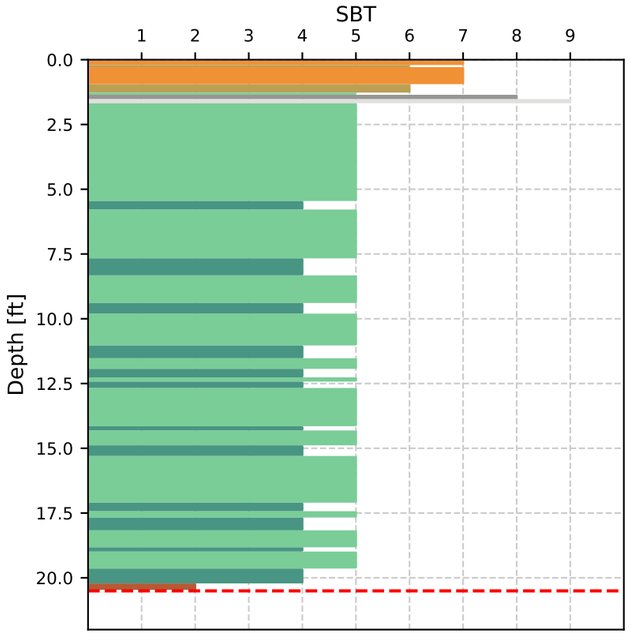
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-04A

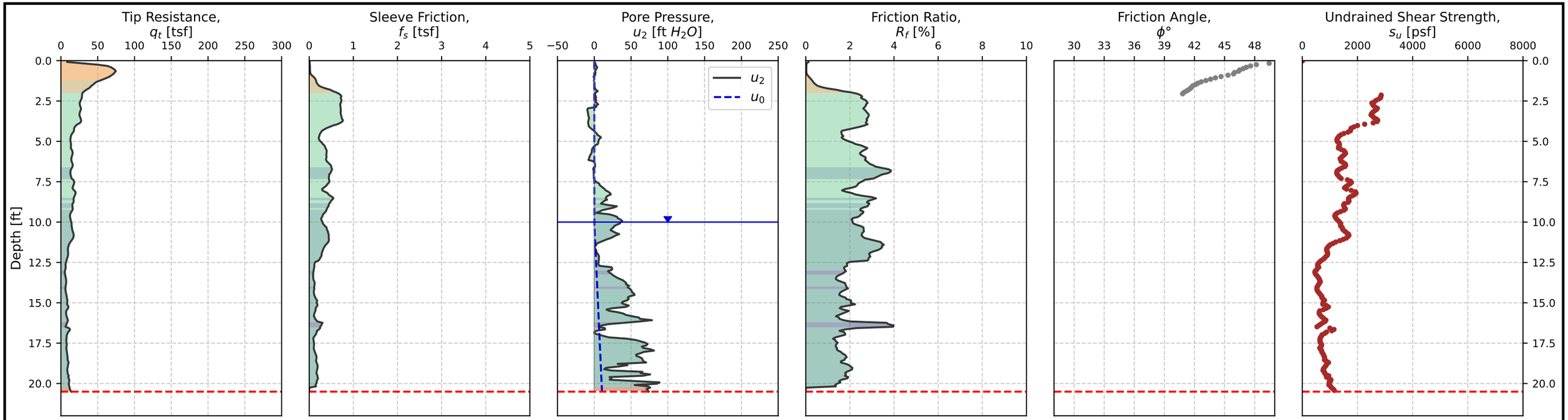
Test Date: 2024-09-10
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Lat: 35.875044° N
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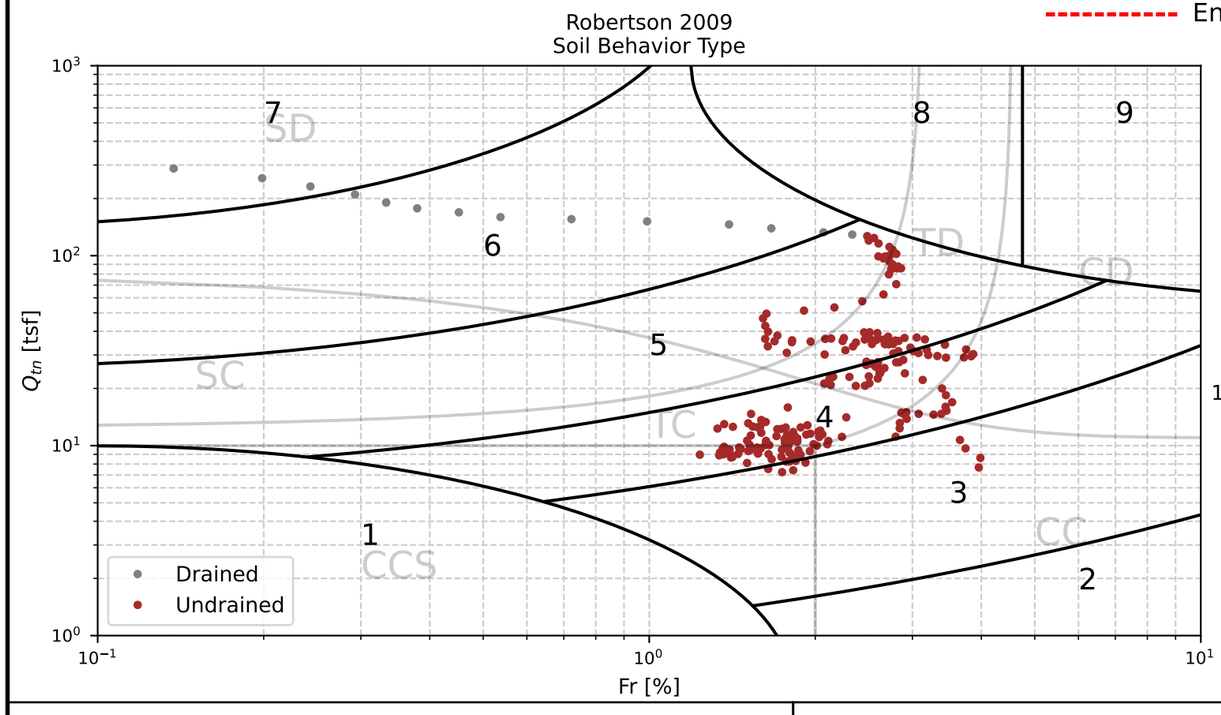
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

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$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

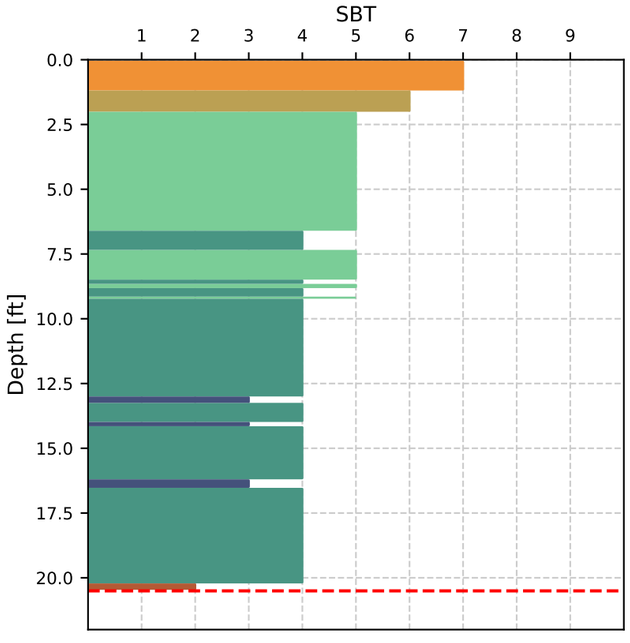
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-05

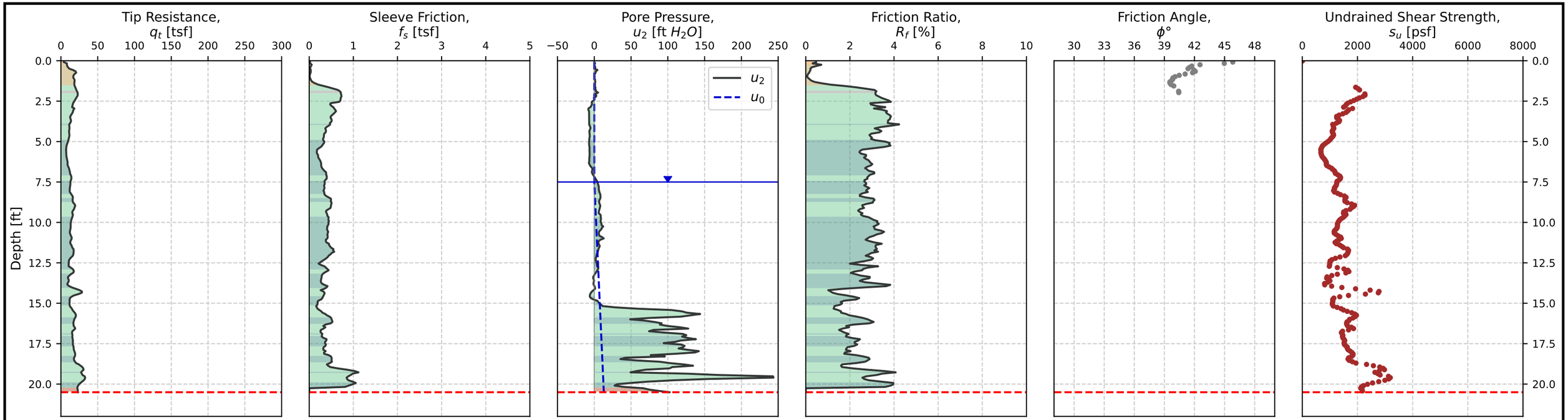
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Horiz. Datum: WGS 84
Native File: 24-56-28295_CP05.COR

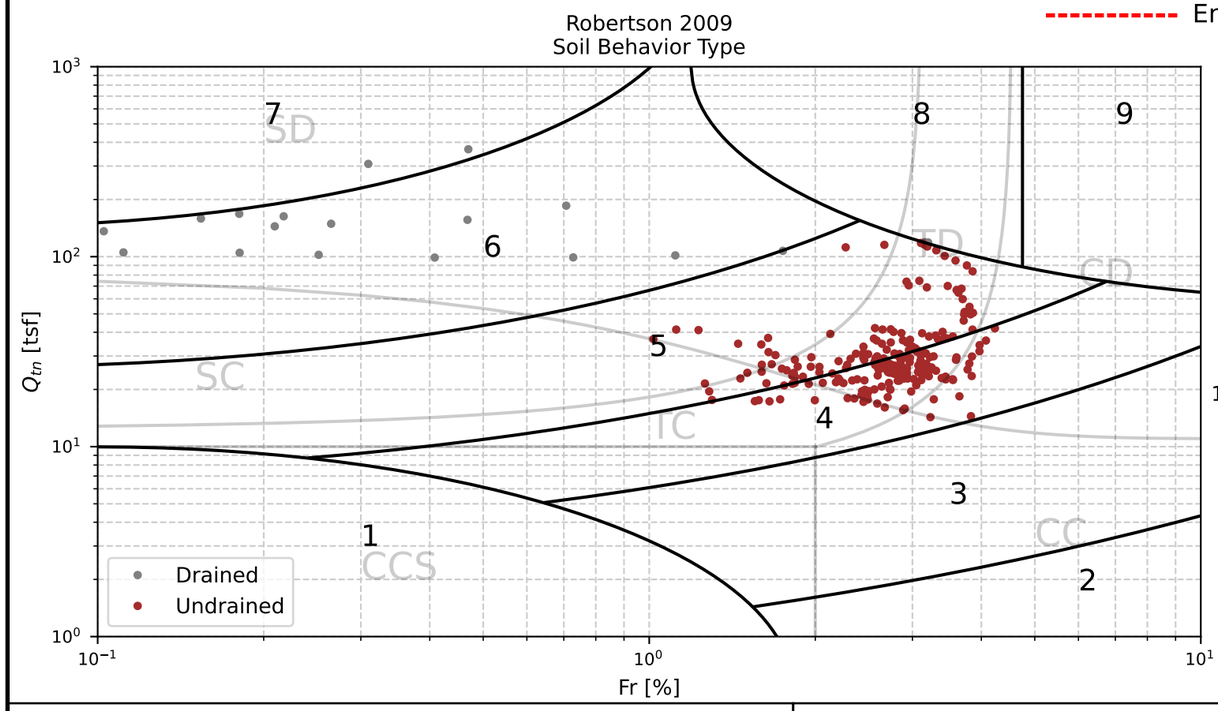
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

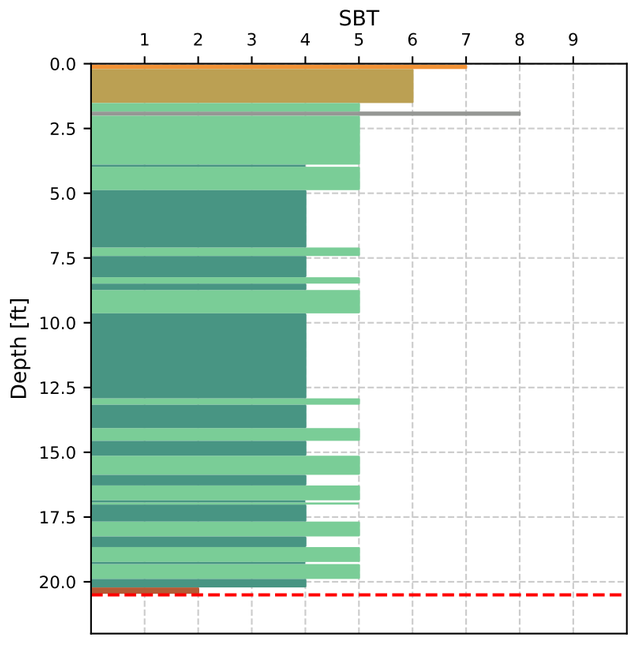
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-06A

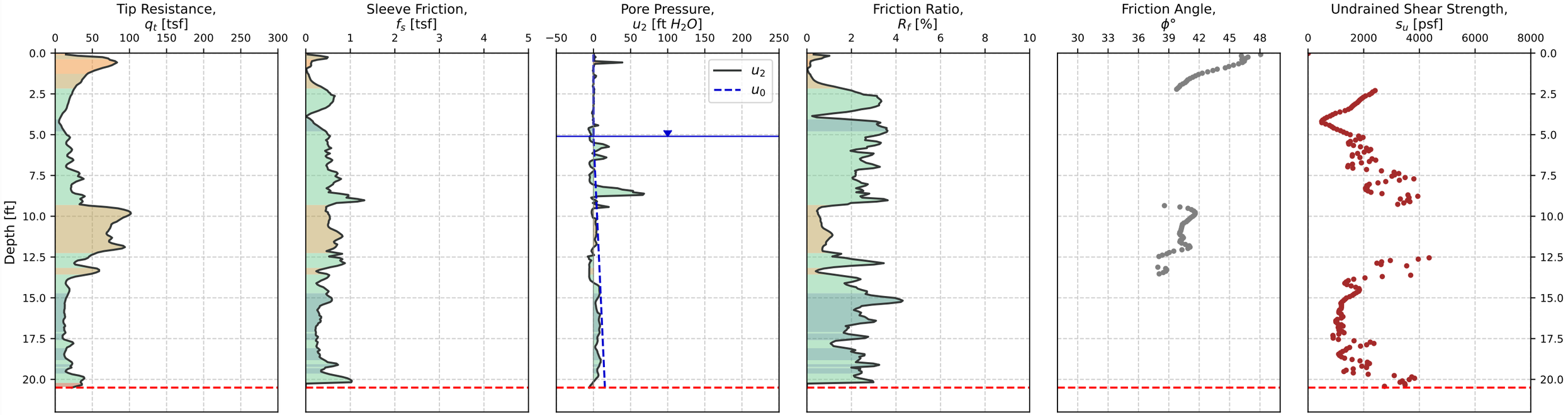
Test Date: 2024-09-11
Long: -119.825883° W
Lat: 35.877654° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP06A.COR

Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

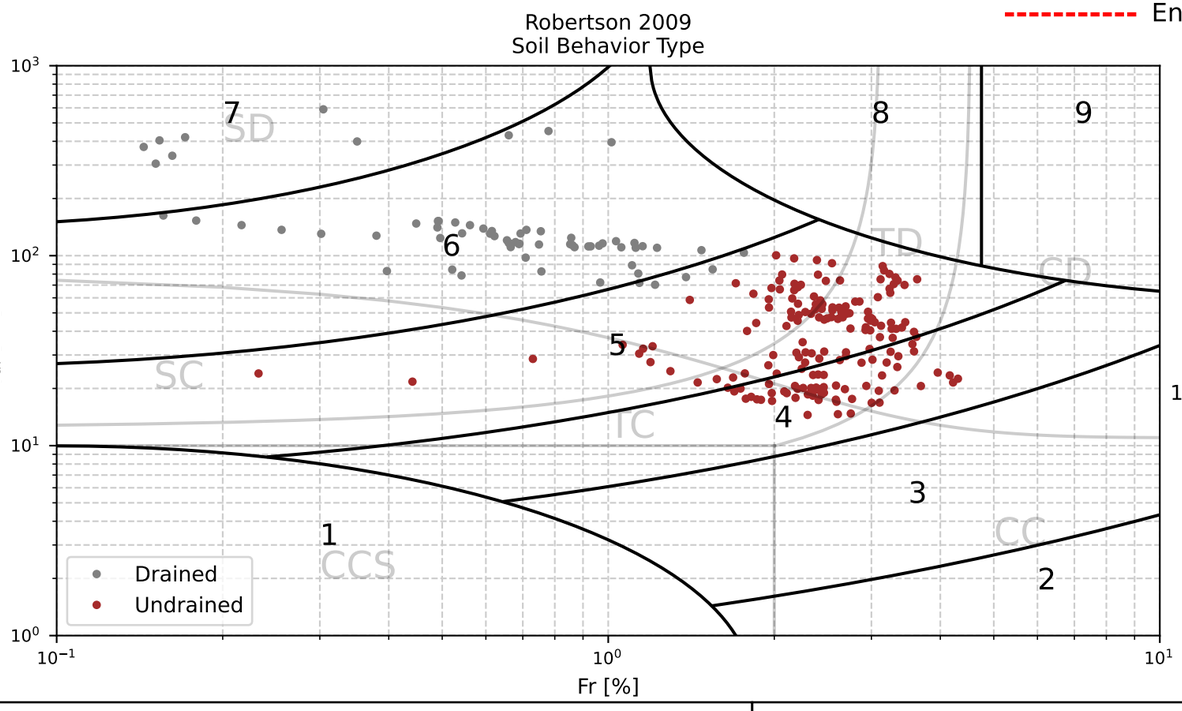
Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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End of test or refusal



SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

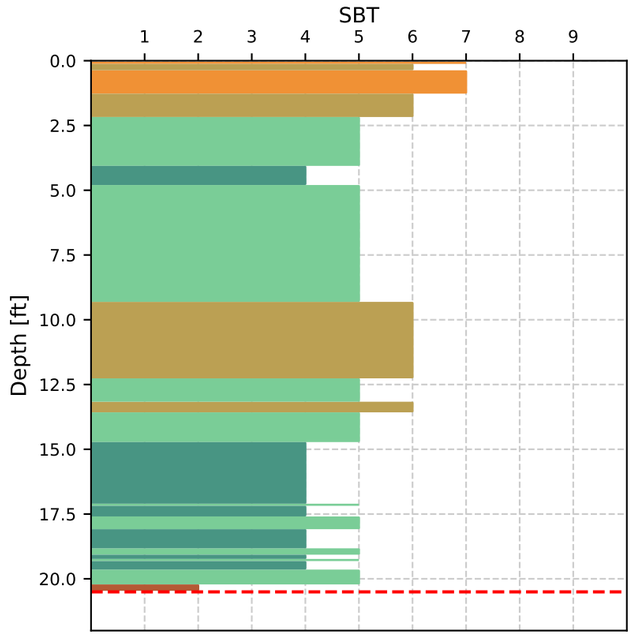
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



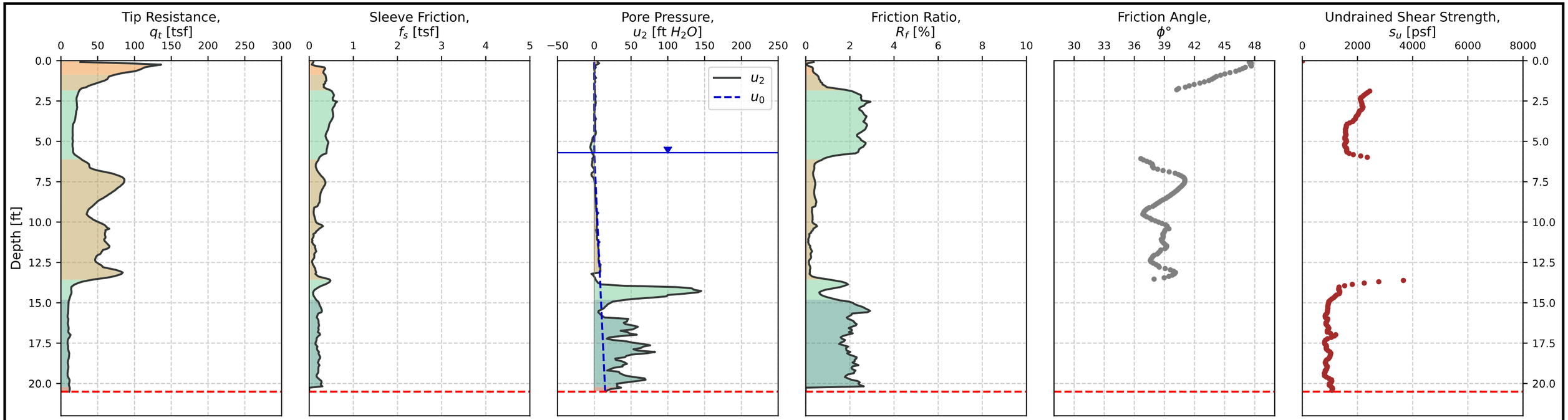
Sounding: GEO-07A
Test Date: 2024-09-11
Long: -119.816609° W
Lat: 35.89822° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP07A.COR

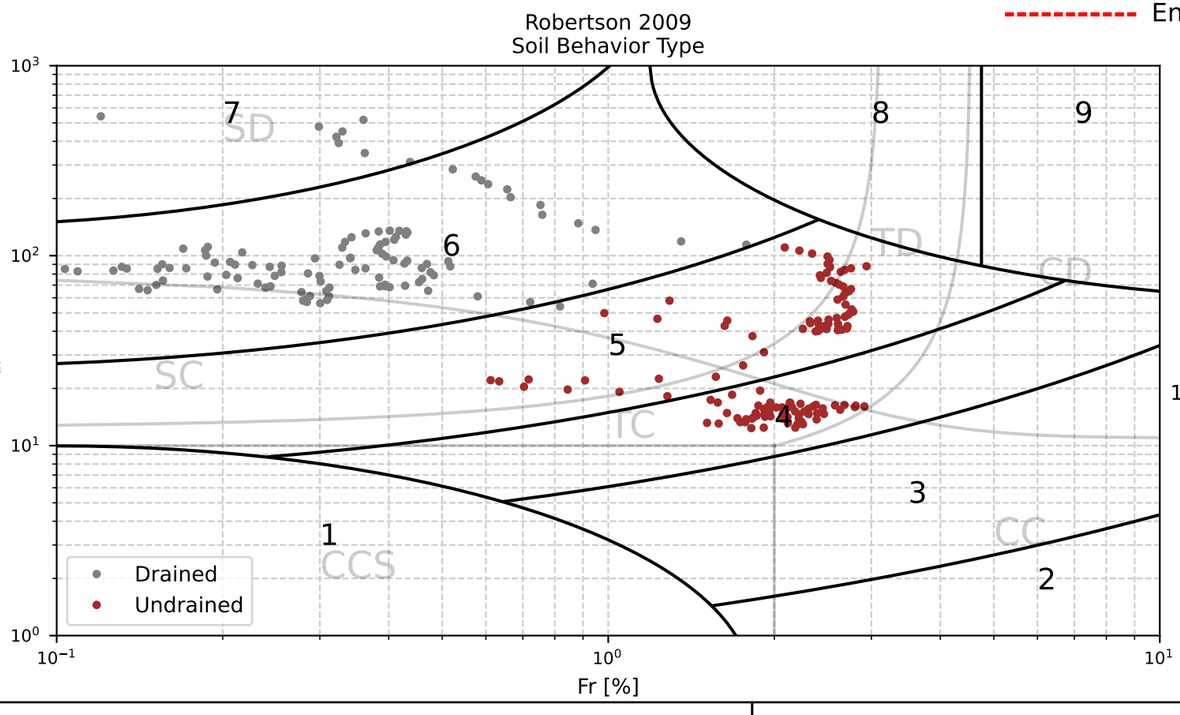
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

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$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

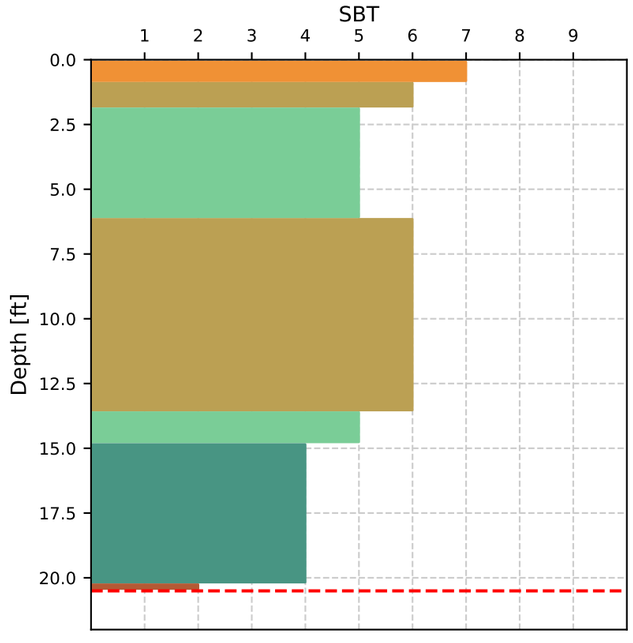
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-08A

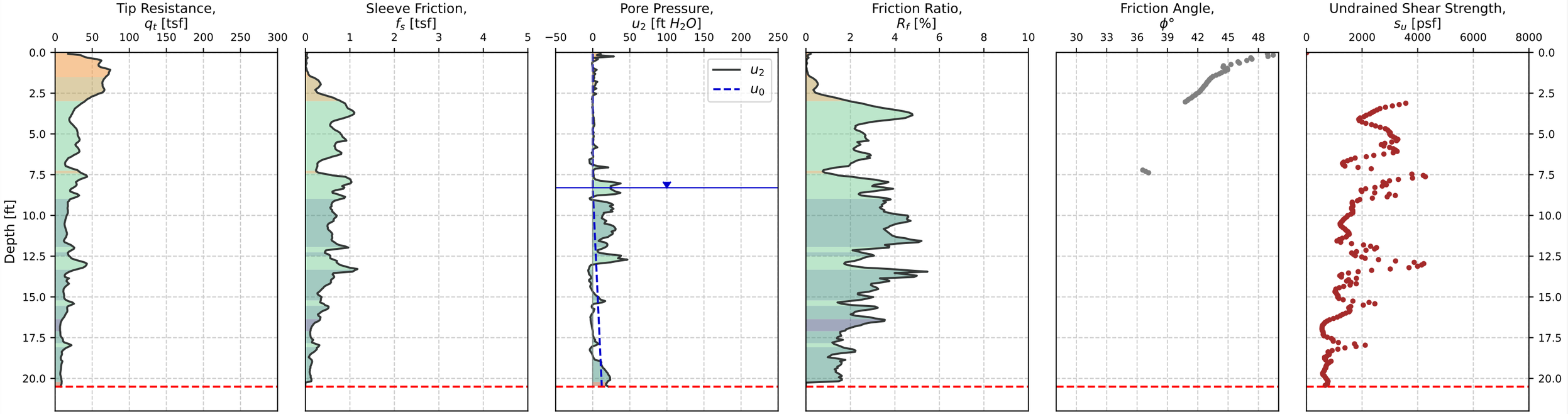
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Long: -119.822508° W
Lat: 35.890704° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP08A.COR

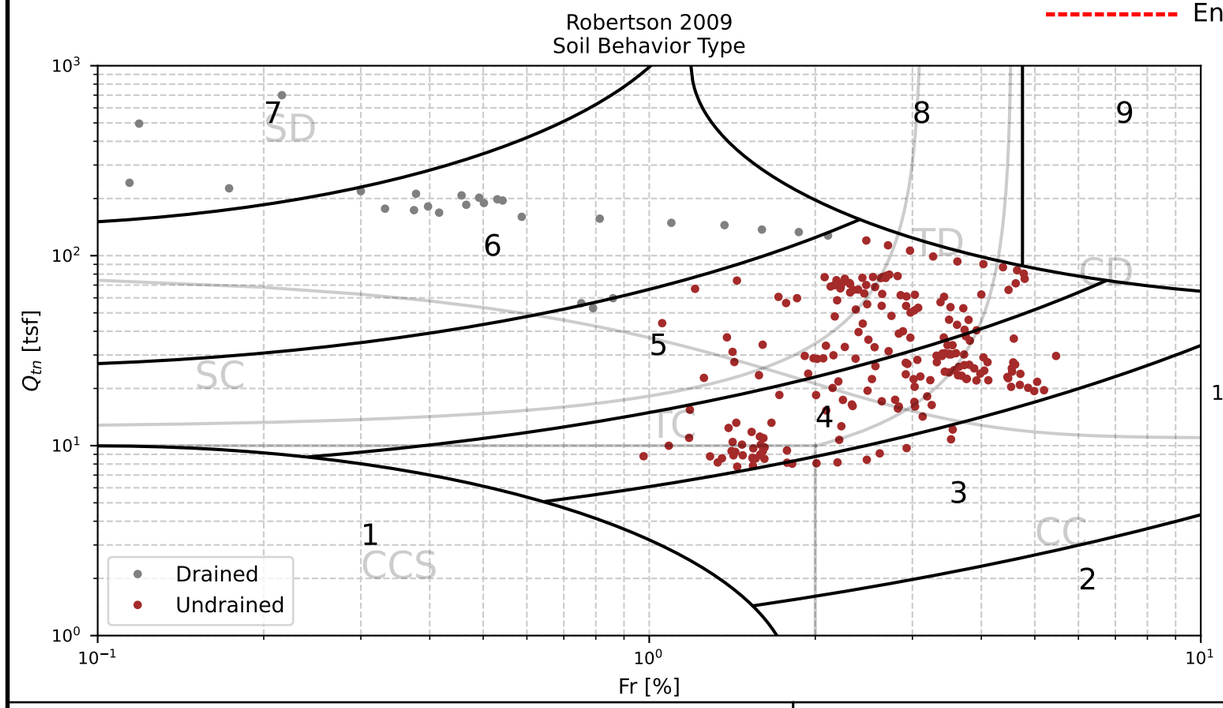
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

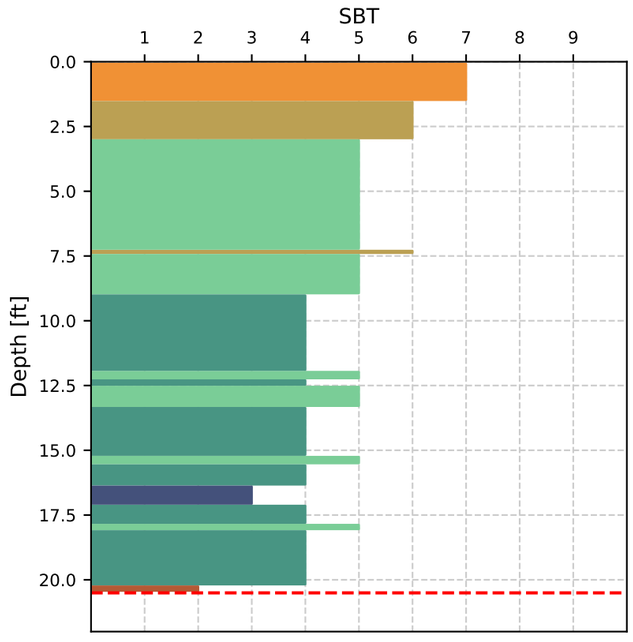
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-09
Test Date: 2024-09-11
Long: -119.808078° W
Lat: 35.890609° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP09.COR

Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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Tip Resistance, q_t [tsf]

Sleeve Friction, f_s [tsf]

Pore Pressure, u_2 [ft H_2O]

Friction Ratio, R_f [%]

Friction Angle, $\phi°$

Undrained Shear Strength, s_u [psf]

Depth [ft]

u_2

u_0

End of test or refusal

Robertson 2009
Soil Behavior Type

Q_{tn} [tsf]

Fr [%]

Drained

Undrained

SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

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$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained

SBT

Depth [ft]

Sounding: GEO-10A

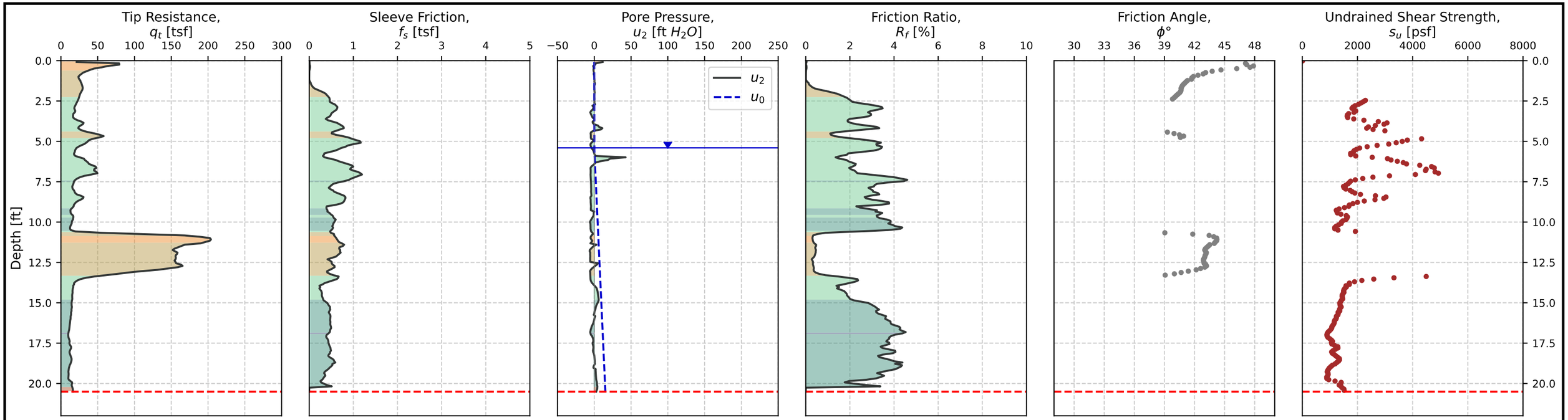
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Lat: 35.905179° N
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Native File: 24-56-28295_CP10A.COR

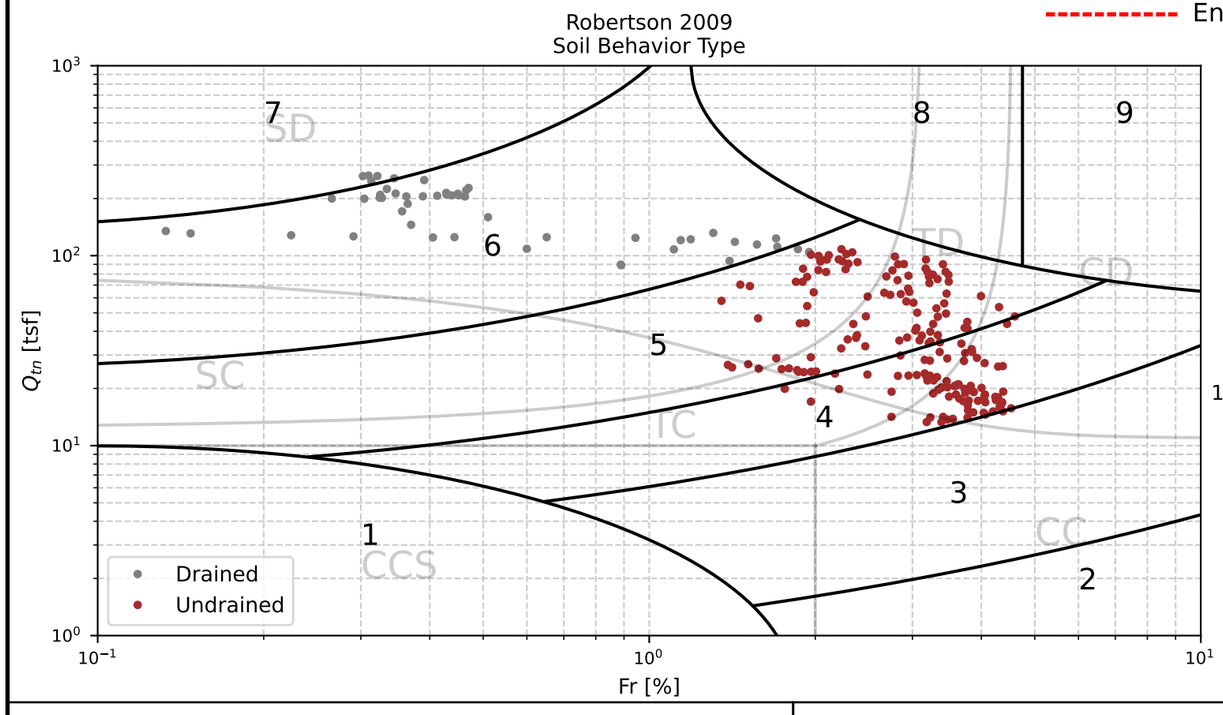
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

p:\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\Work Files\Field\CPT\ConeTec_CPT_Tool\Deliverables\CPT_Plots\CPT_Plots_20241009.pdf





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$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

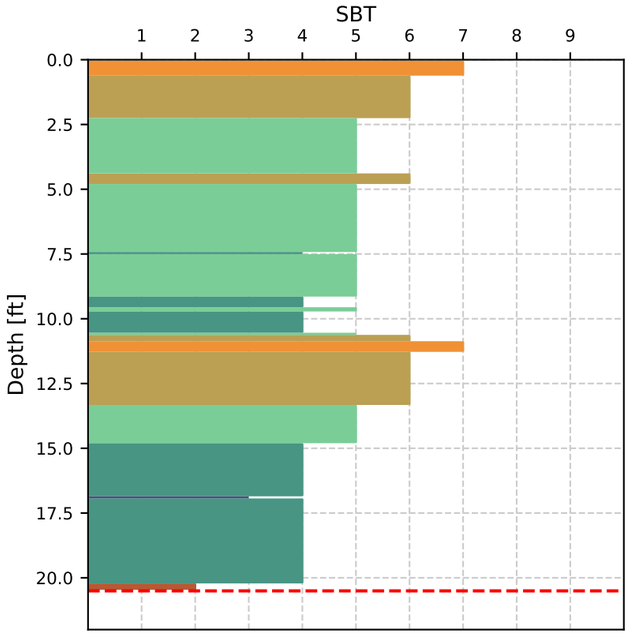
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-11

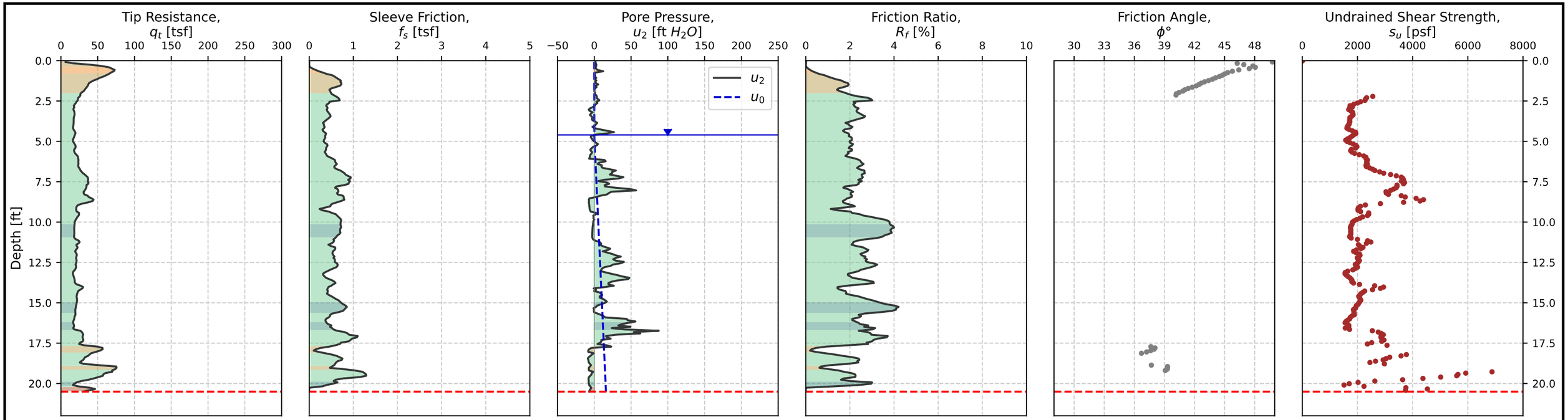
Test Date: 2024-09-11
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Horiz. Datum: WGS 84
Native File: 24-56-28295_CP11.COR

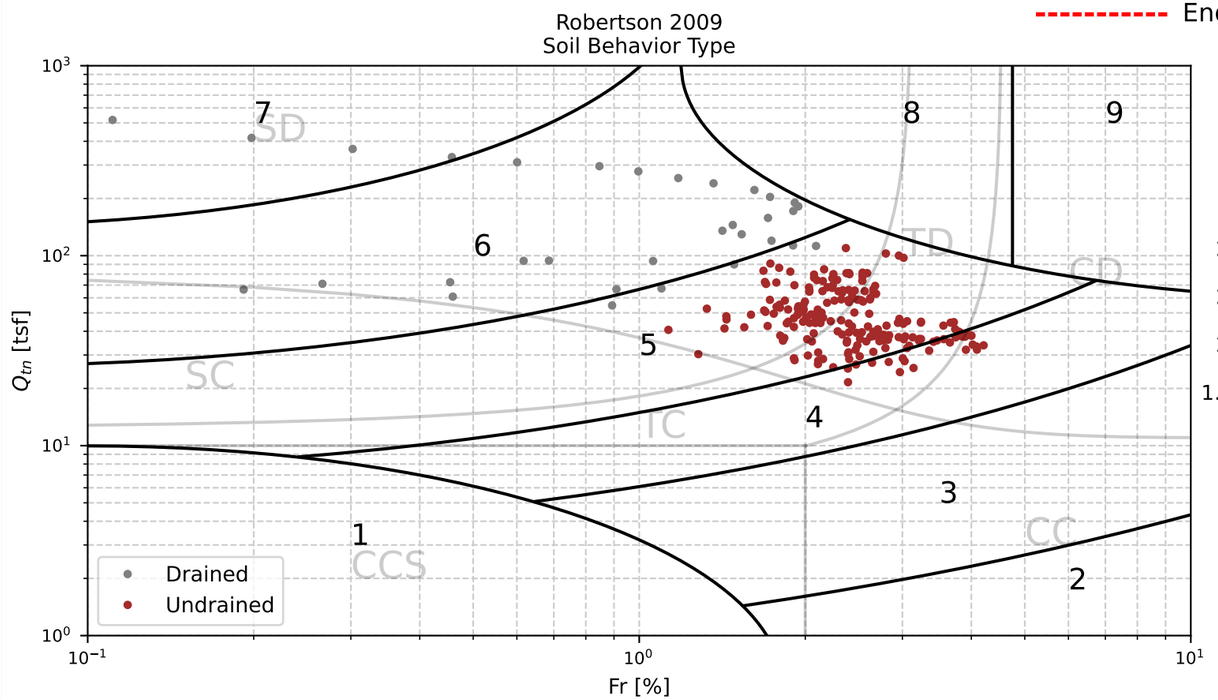
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

p:\Mpls\05 CA\161\05161003 Rex Solaire Solar Project\Work Files\Field\CPT\ConeTec_CPT_Tool\Deliverables\CPT_Plots\CPT_Plots_20241009.pdf





SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

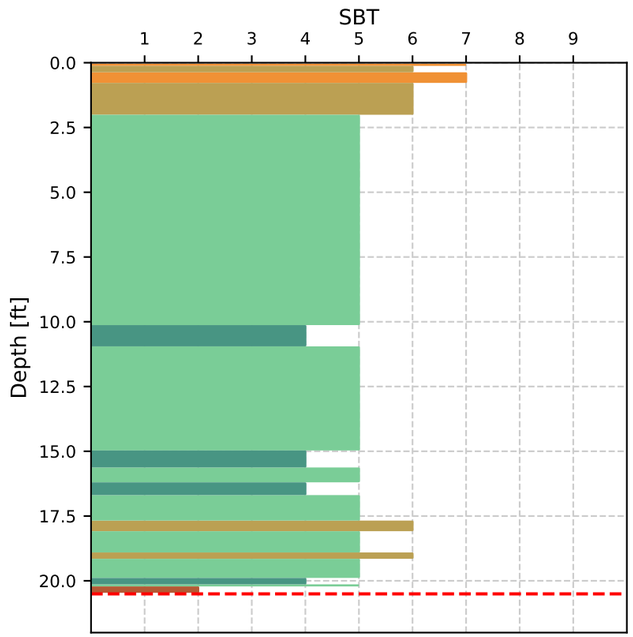
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-12

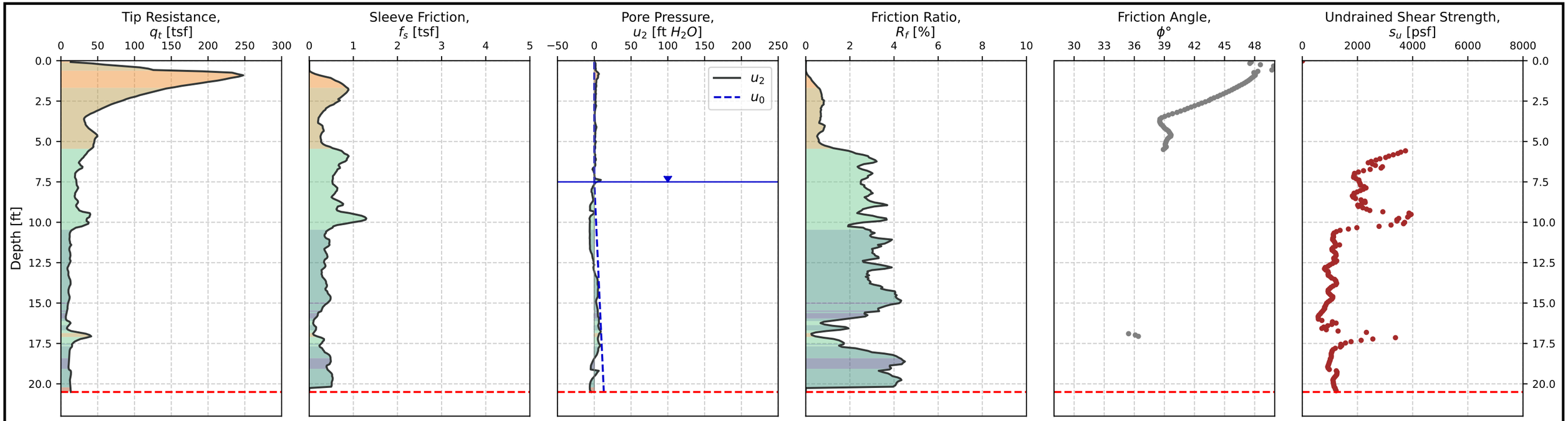
Test Date: 2024-09-12
Long: -119.807899° W
Lat: 35.912052° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP12.COR

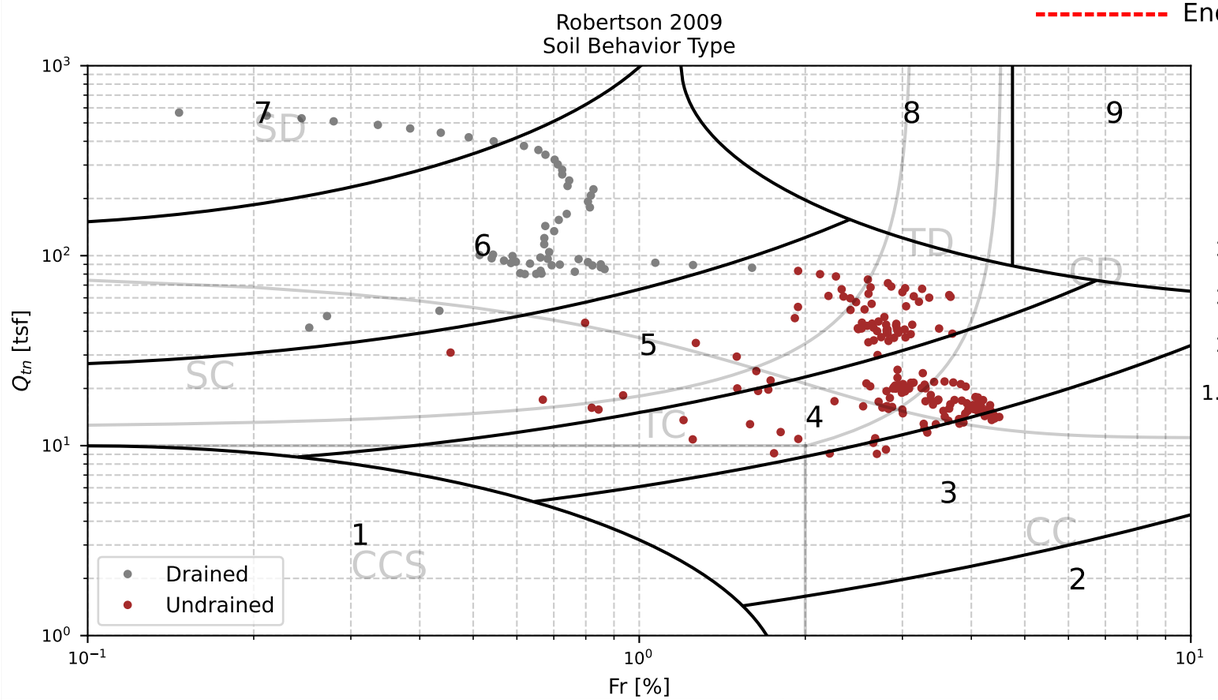
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

3.6<I_c 2 = Organic soils - peat

2.95<I_c<3.6 3 = Clays - clay to silty clay

2.6<I_c<2.95 4 = Silt mixtures, clayey silt to silty clay

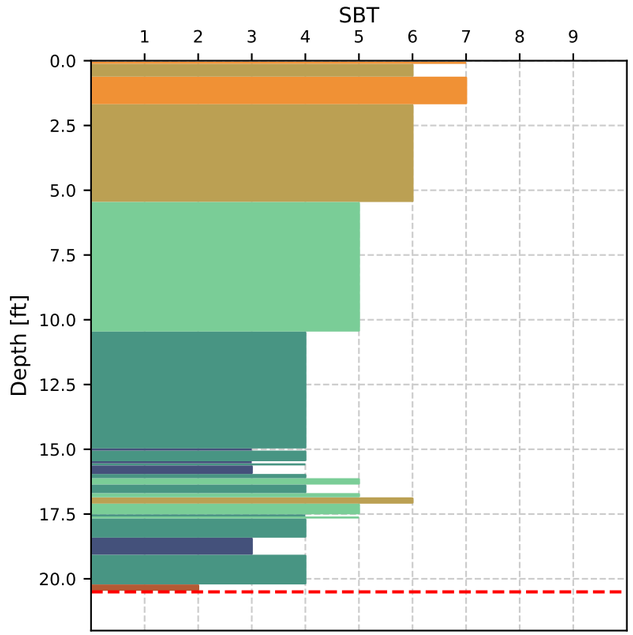
2.05<I_c<2.6 5 = Sand mixtures, silty sand to sandy silt

1.31<I_c<2.05 6 = Sands, clean sands to silty sands

I_c<1.31 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-13

Test Date: 2024-09-12
Long: -119.807779° W
Lat: 35.919549° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP13.COR

Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

p:\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\Field\CPT\ConeTec_CPT_Tool\Deliverables\CPT_Plots\CPT_Plots_20241009.pdf

Tip Resistance, q_t [tsf]

Sleeve Friction, f_s [tsf]

Pore Pressure, u_2 [ft H_2O]

Friction Ratio, R_f [%]

Friction Angle, $\phi°$

Undrained Shear Strength, s_u [psf]

Depth [ft]

u_2

u_0

End of test or refusal

Robertson 2009
Soil Behavior Type

Q_{tn} [tsf]

Fr [%]

Drained

Undrained

SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained

SBT

Depth [ft]

Sounding: GEO-14

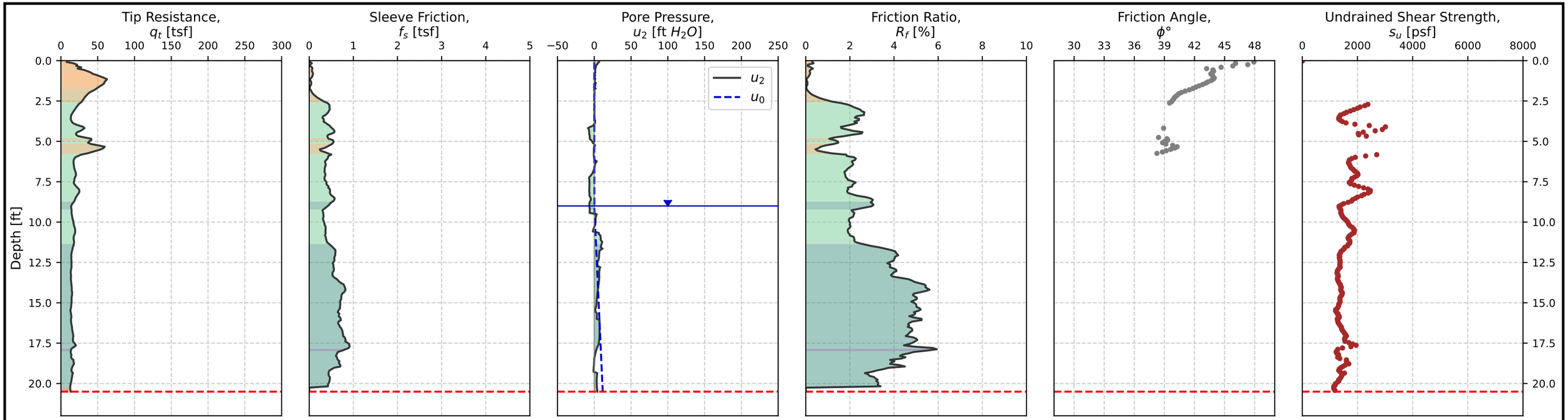
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Lat: 35.91954° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP14.COR

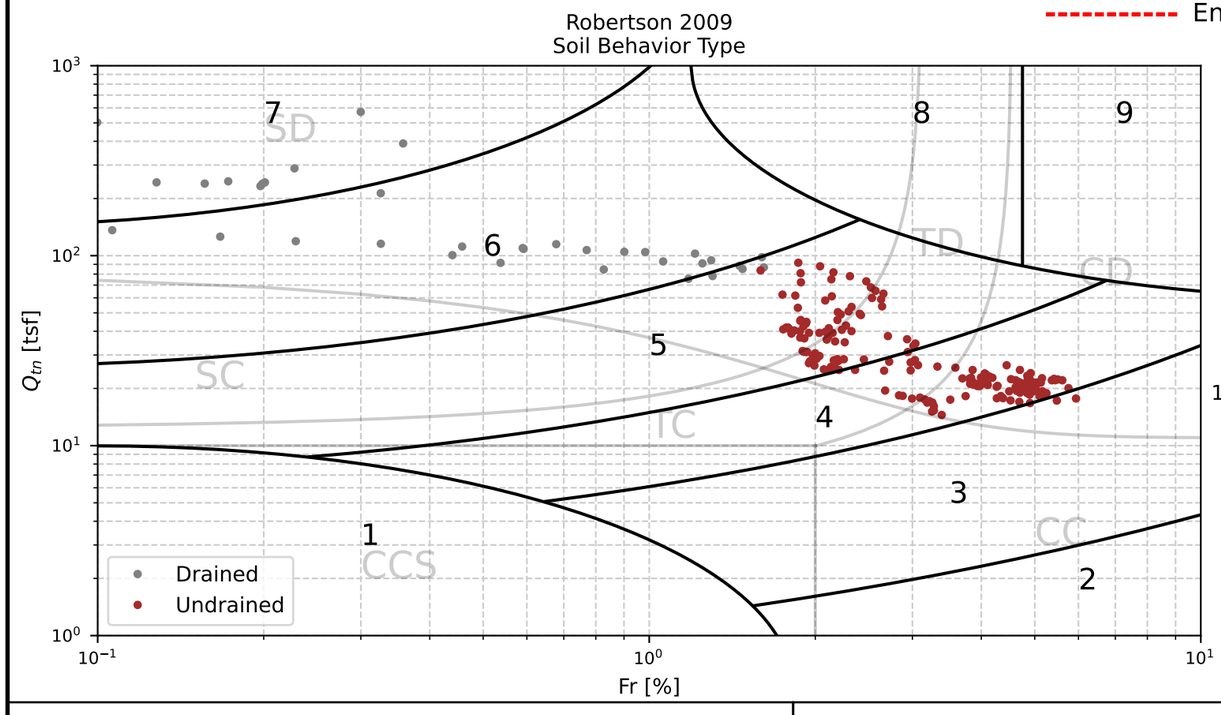
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

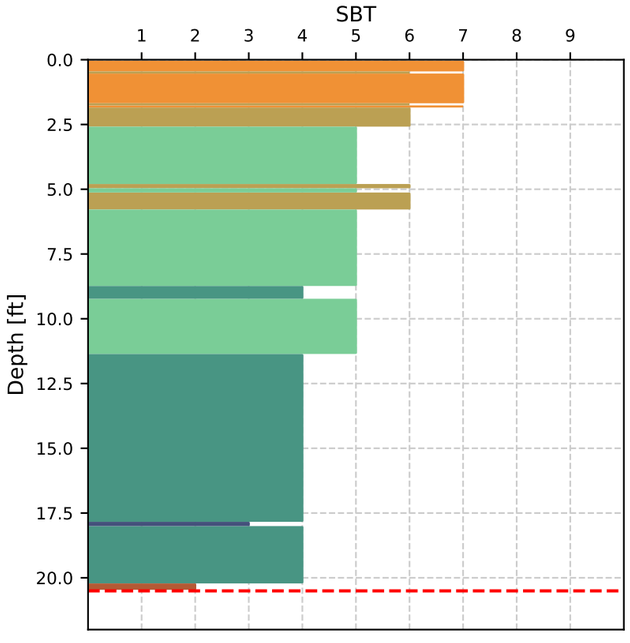
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-15

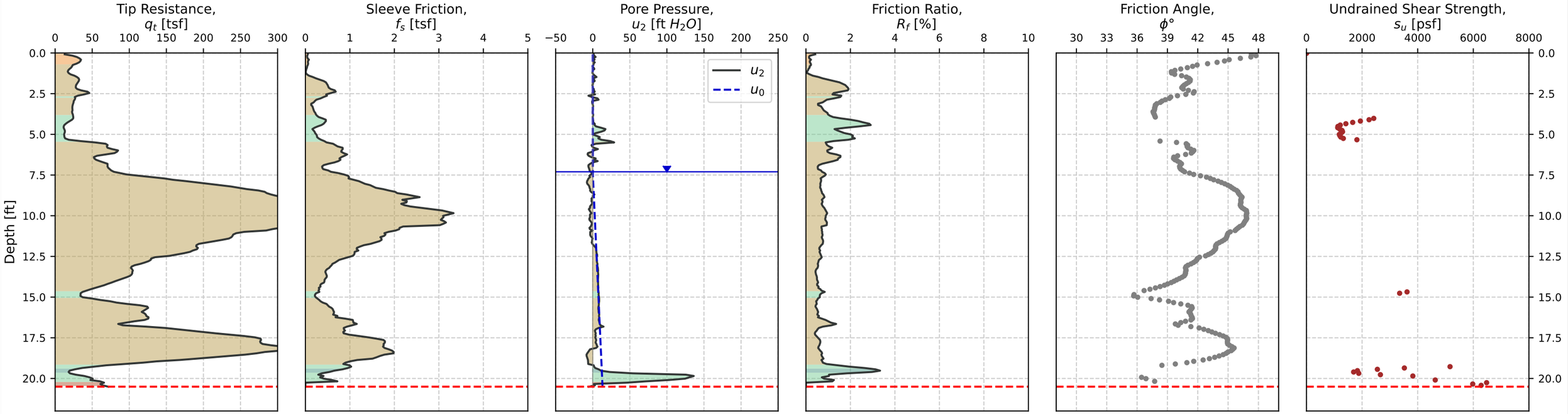
Test Date: 2024-09-12
Long: -119.772385° W
Lat: 35.883182° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP15.COR

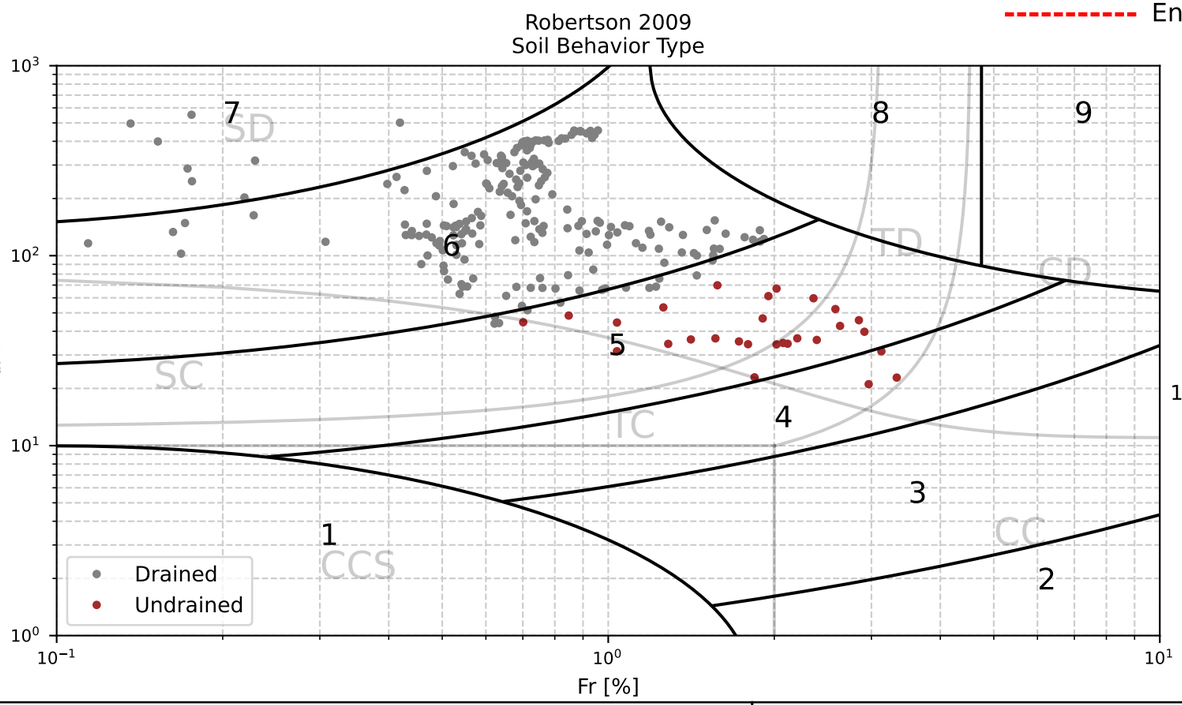
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

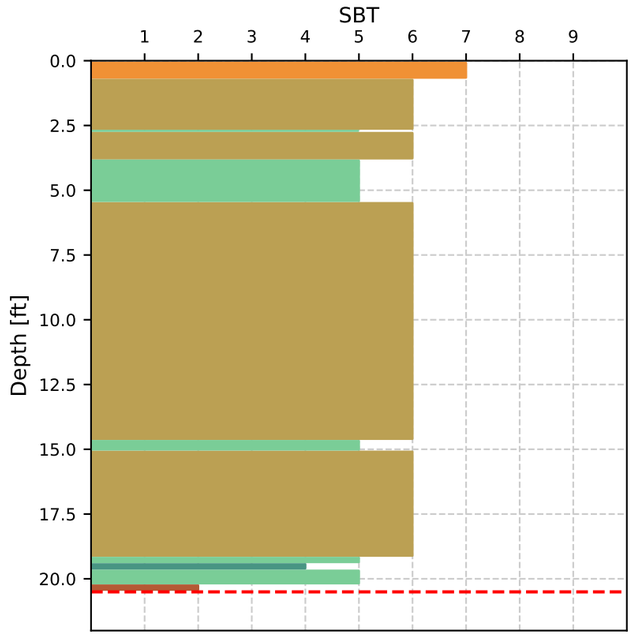
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



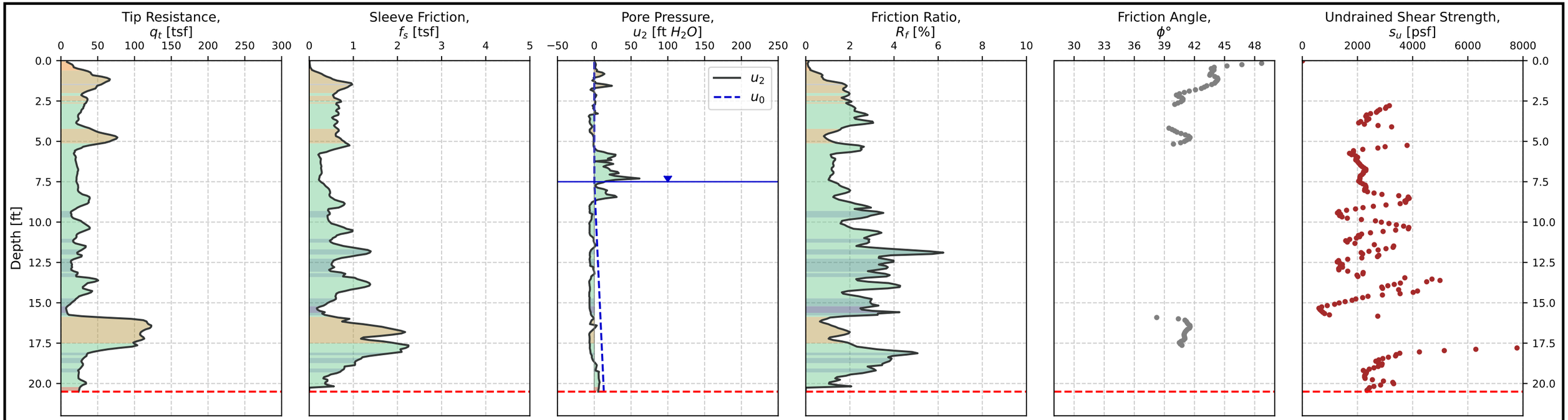
Sounding: GEO-16
Test Date: 2024-09-12
Long: -119.73577° W
Lat: 35.88319° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_SP16.COR

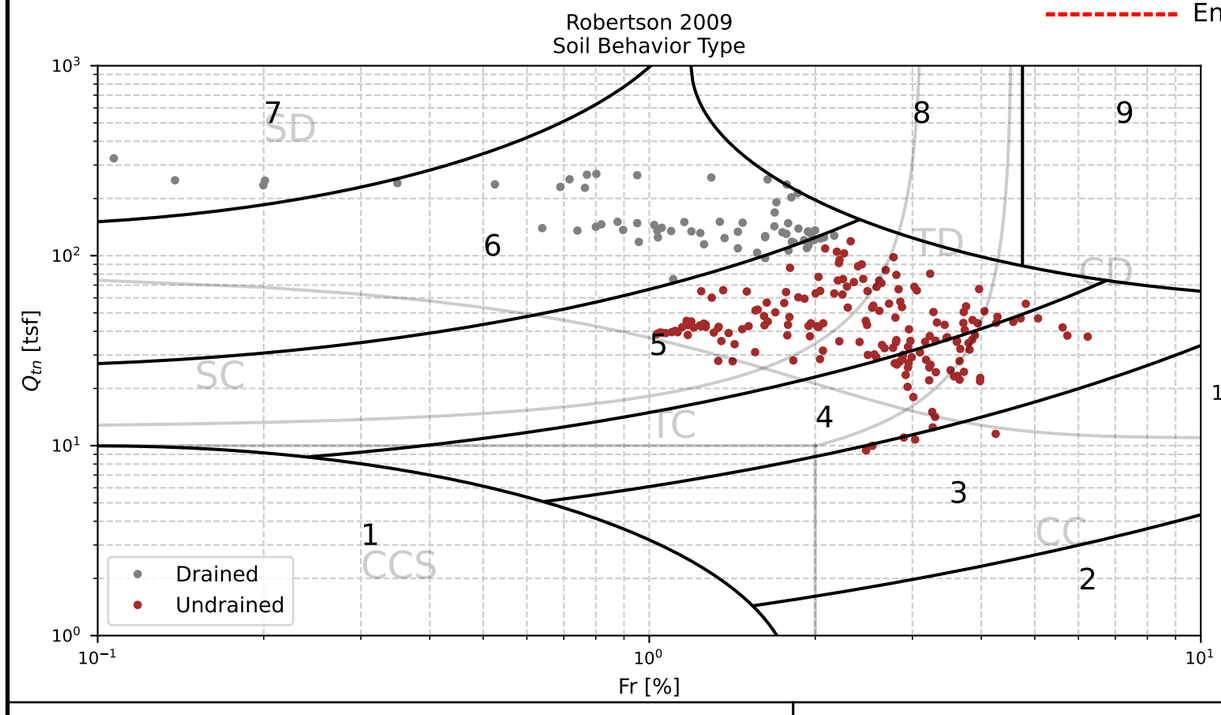
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

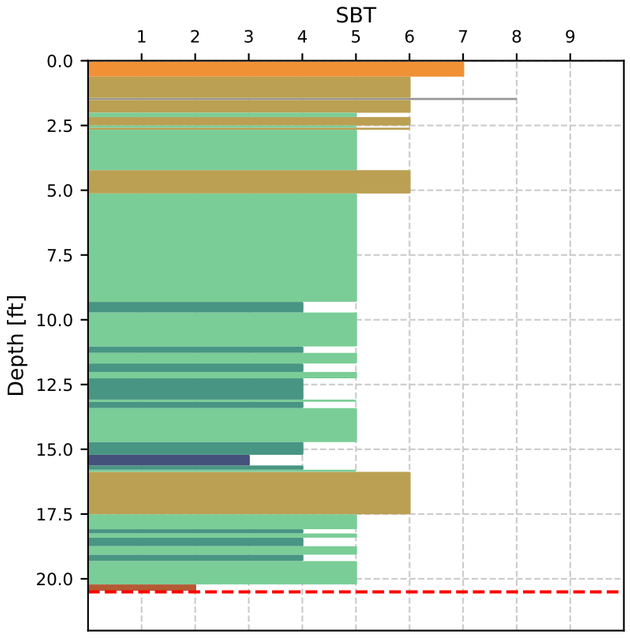
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-17

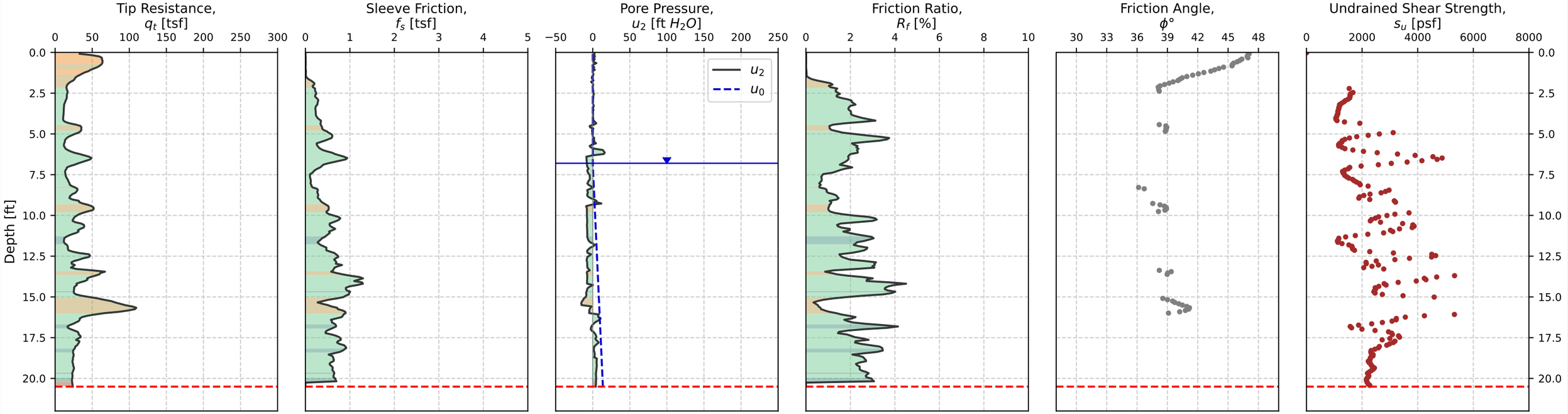
Test Date: 2024-09-12
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Lat: 35.890706° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP17.COR

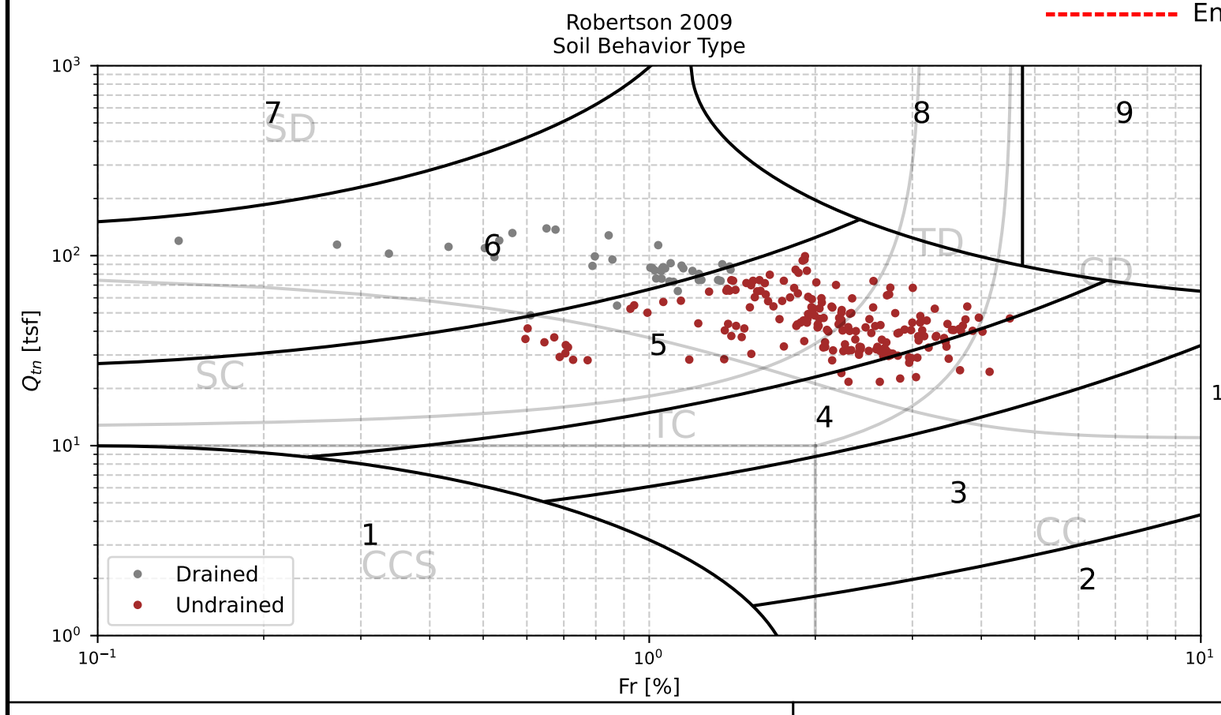
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

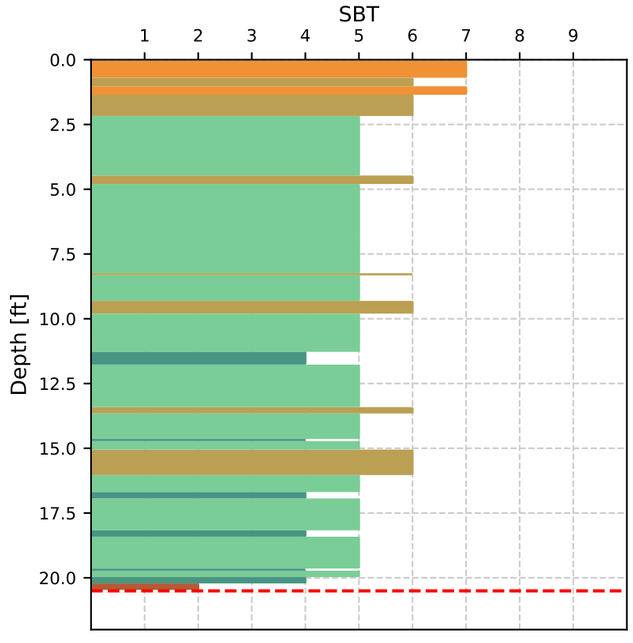
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-18

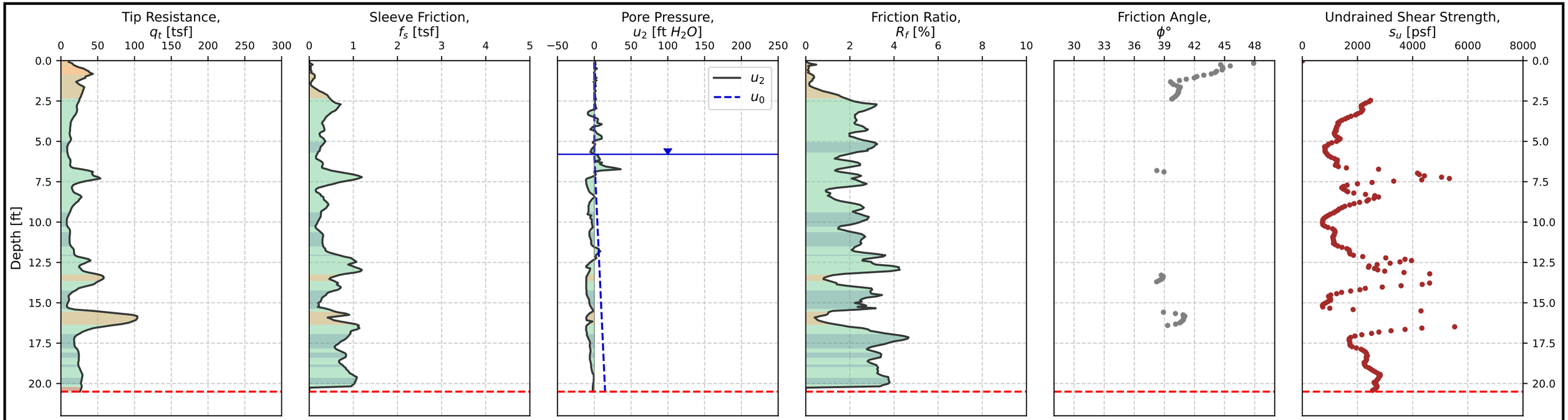
Test Date: 2024-09-12
Long: -119.735756° W
Lat: 35.897437° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP18.COR

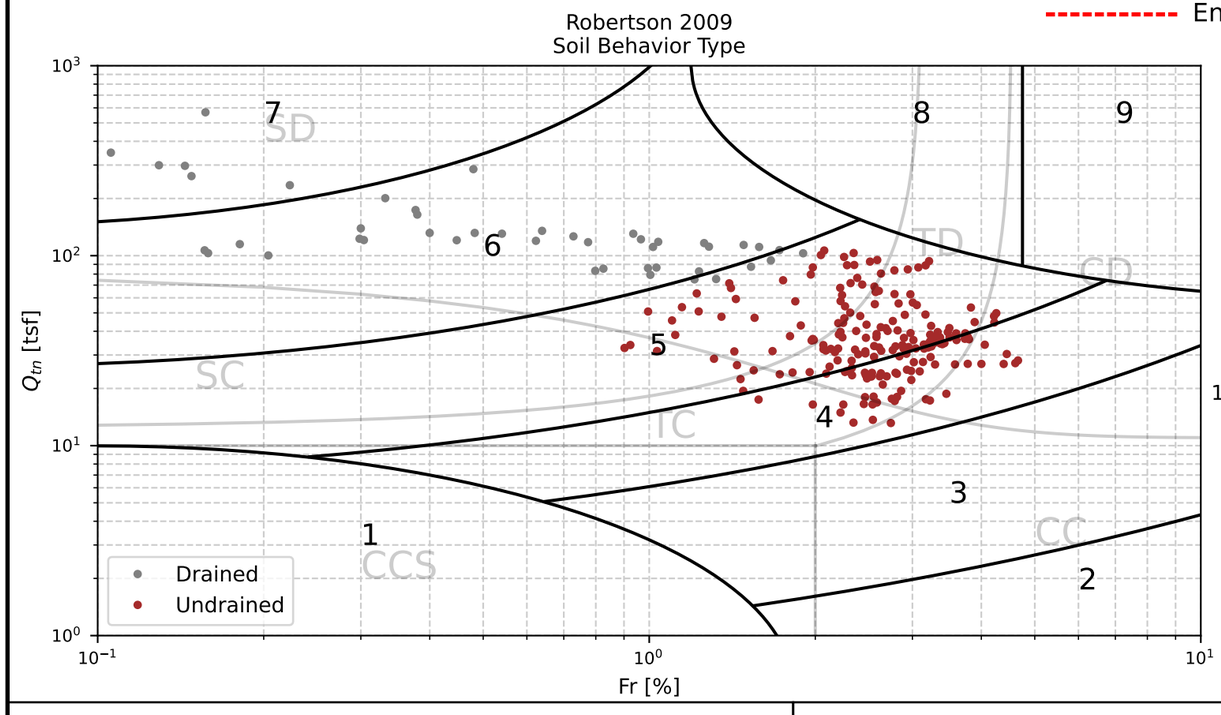
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

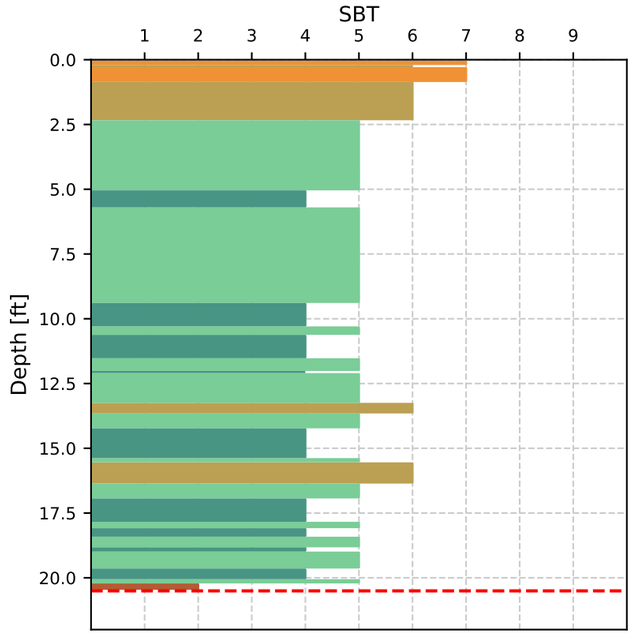
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-19

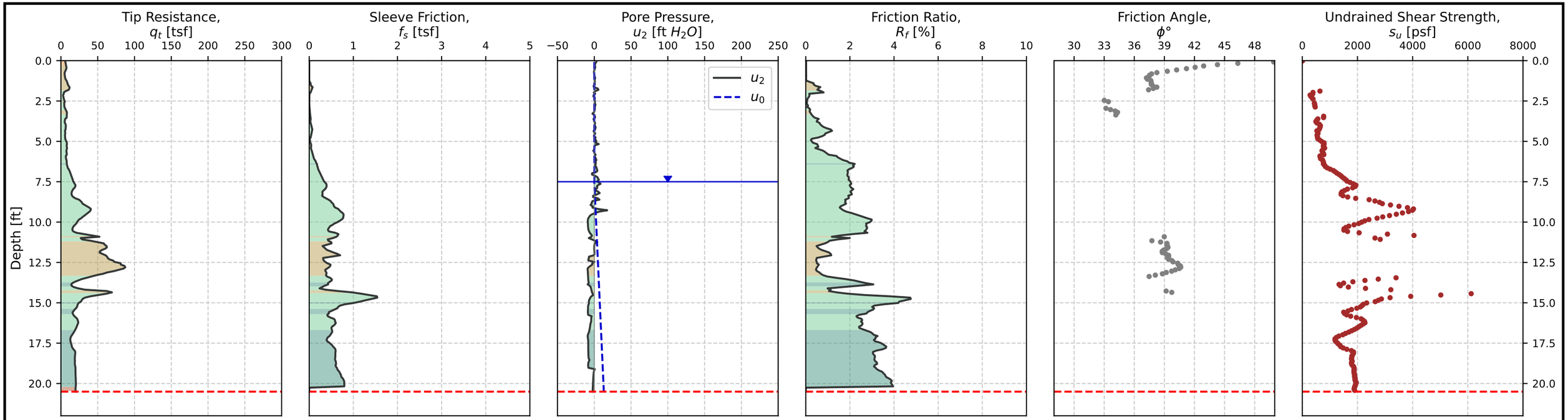
Test Date: 2024-09-13
Long: -119.753263° W
Lat: 35.904912° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP19.COR

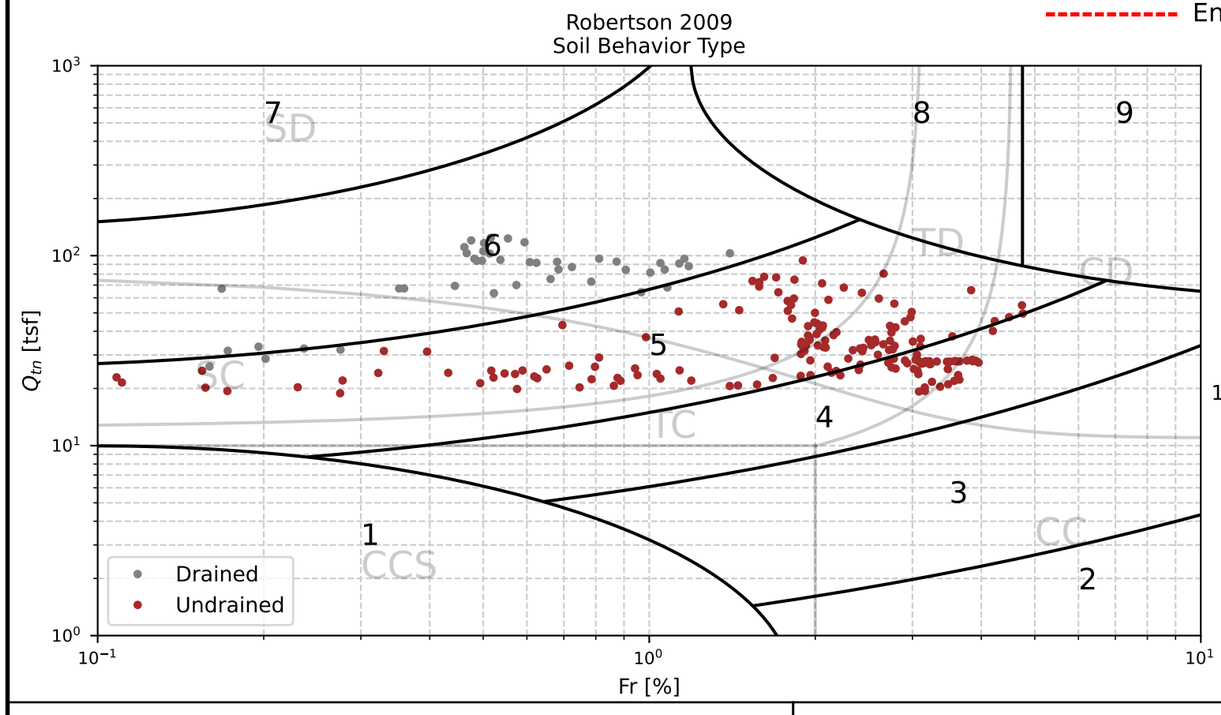
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

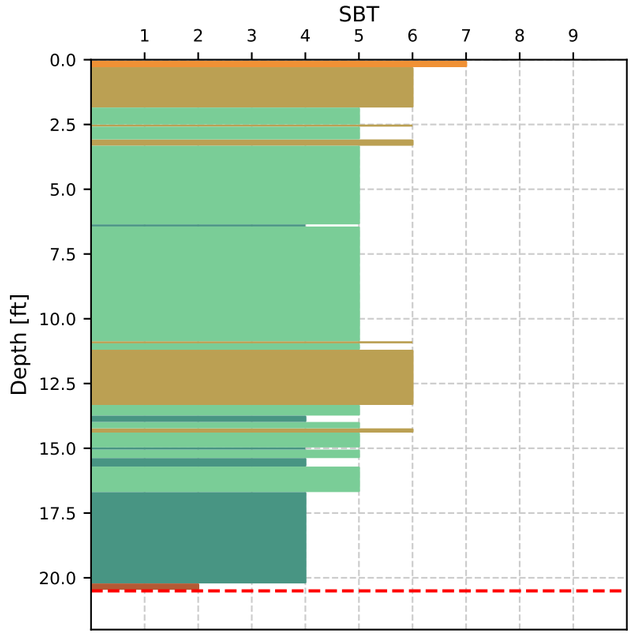
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-20

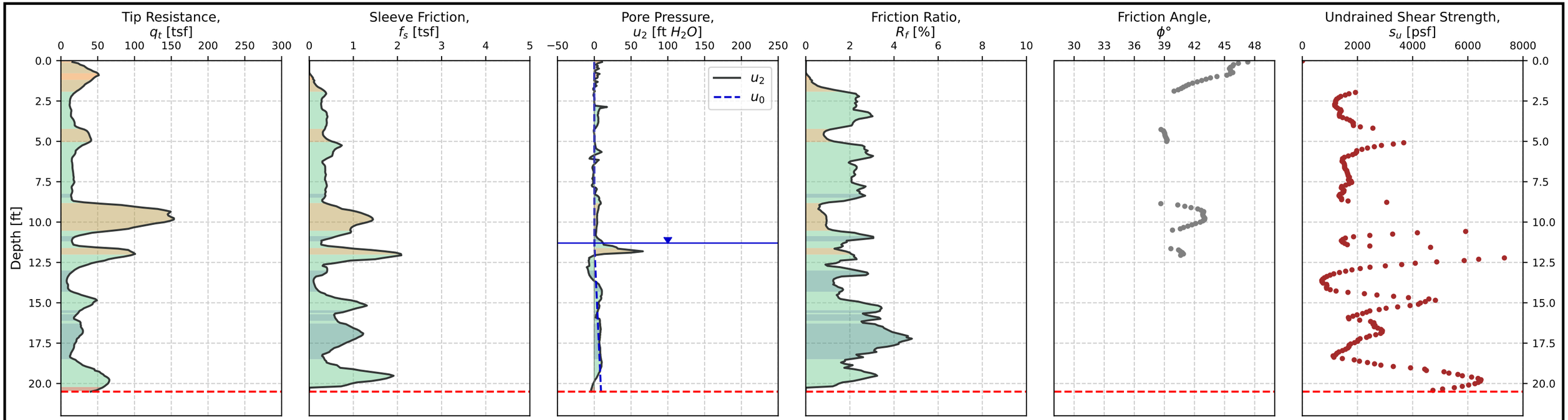
Test Date: 2024-09-13
Long: -119.735812° W
Lat: 35.912777° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP20.COR

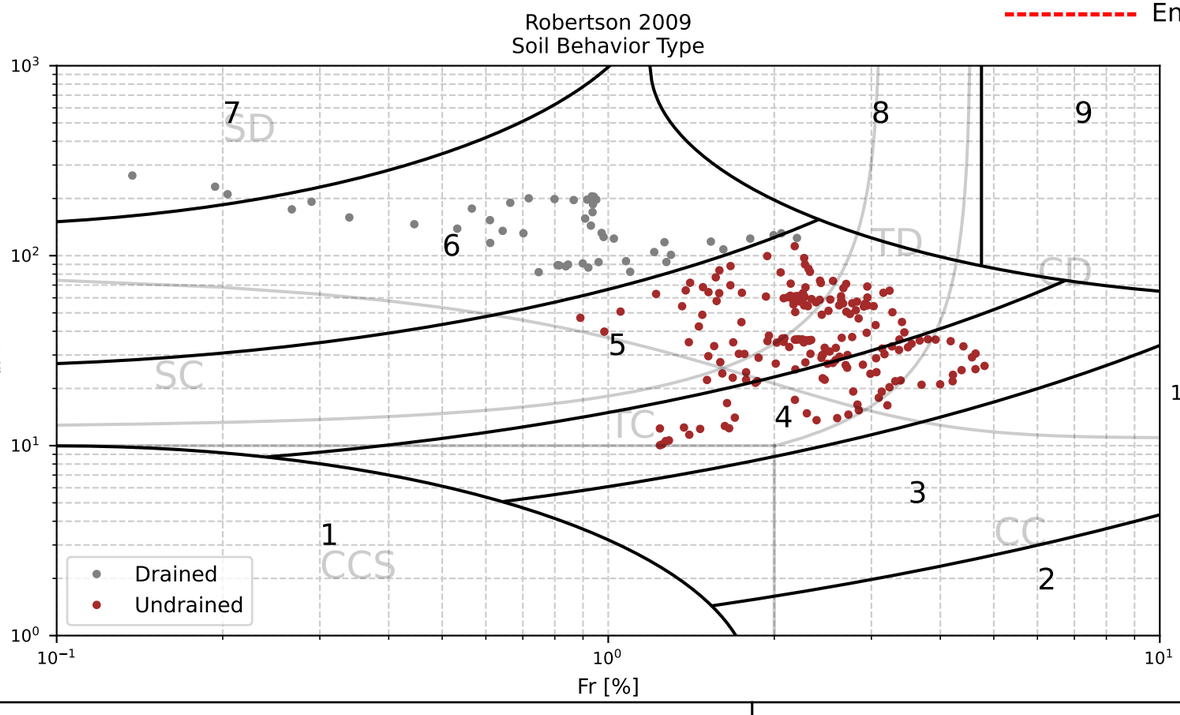
Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

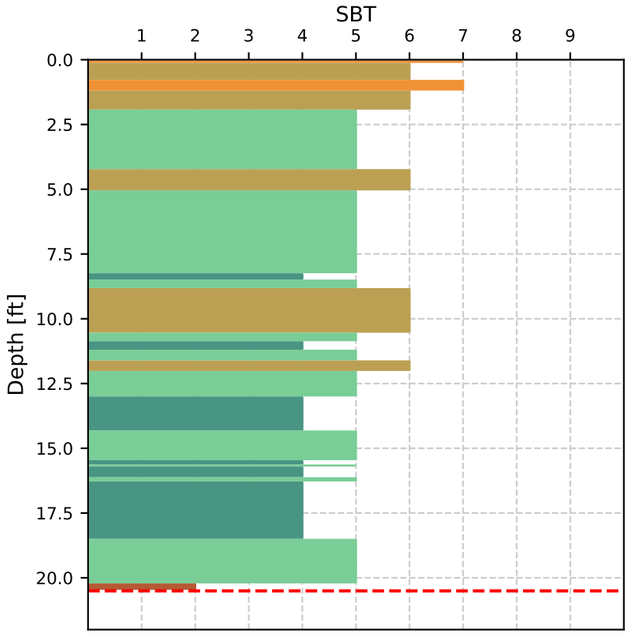
$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained



Sounding: GEO-21

Test Date: 2024-09-13
Long: -119.735797° W
Lat: 35.926615° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_CP21.COR

Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

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Tip Resistance, q_t [tsf]

Sleeve Friction, f_s [tsf]

Pore Pressure, u_2 [ft H_2O]

Friction Ratio, R_f [%]

Friction Angle, $\phi°$

Undrained Shear Strength, s_u [psf]

Depth [ft]

u_2

u_0

End of test or refusal

Robertson 2009
Soil Behavior Type

Q_{tn} [tsf]

Fr [%]

Drained

Undrained

SBT per Robertson (2009)

1 = Sensitive, fine grained

$3.6<I_c$ 2 = Organic soils - peat

$2.95<I_c<3.6$ 3 = Clays - clay to silty clay

$2.6<I_c<2.95$ 4 = Silt mixtures, clayey silt to silty clay

$2.05<I_c<2.6$ 5 = Sand mixtures, silty sand to sandy silt

$1.31<I_c<2.05$ 6 = Sands, clean sands to silty sands

$I_c<1.31$ 7 = Gravelly sand to sand

8 = Very stiff sand to clayey sand

9 = Very stiff fine grained

SBT

Depth [ft]

Sounding: SUB-Opt1-01

Test Date: 2024-09-10
Long: -119.828469° W
Lat: 35.859817° N
Horiz. Datum: WGS 84
Native File: 24-56-28295_SP01SUB-OPT1-01.COR

Drained/Undrained transition based on I_c = 2.1.
Robertson and Wride (1998) Soil Behavior Index.
Undrained Shear Strength per Lunne et al. (1997). Nkt value: 20.
Friction Angle per Kulhaway and Mayne (2006).

Barr Engineering Co.

Project Name: Rex Solaire Solar Project
Project Location: Kings County, CA
Project Number: 5161003.0

p:\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\Work Files\Field\CPT\ConeTec_CPT_Tool\Deliverables\CPT_Plots\CPT_Plots_20241009.pdf

Appendix C

Seismic CPT Results

Job No:	24-56-28295
Client:	Barr Engineering Co.
Project:	Rex Solaire Solar Project
Sounding ID:	GEO-16
Date:	2024-09-12
Seismic Source:	Beam
Seismic Offset (ft):	2.13
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.99	4.33	4.83			
10.89	10.24	10.46	5.63	11.88	474
15.03	14.37	14.53	4.07	6.62	615
20.51	19.85	19.96	5.44	7.80	697

Job No:	24-56-28295
Client:	Barr Engineering Co.
Project:	Rex Solaire Solar Project
Sounding ID:	Sub-Opt1-01
Date:	2024-09-10
Seismic Source:	Beam
Seismic Offset (ft):	2.13
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
6.00	5.35	5.76			
9.35	8.69	8.95	3.19	5.29	604
12.63	11.97	12.16	3.21	5.29	607
15.91	15.26	15.40	3.24	6.17	525
19.19	18.54	18.66	3.25	5.79	562
22.47	21.82	21.92	3.26	5.29	617
25.75	25.10	25.19	3.27	5.29	618
29.04	28.38	28.46	3.27	4.65	704
32.32	31.66	31.73	3.27	3.88	844
35.60	34.94	35.01	3.27	3.46	945
38.88	38.22	38.28	3.28	3.47	945
42.16	41.50	41.56	3.28	3.56	921
45.44	44.78	44.83	3.28	3.63	902
48.72	48.06	48.11	3.28	3.52	932
52.00	51.35	51.39	3.28	3.59	913
55.28	54.63	54.67	3.28	3.49	938
58.56	57.91	57.95	3.28	3.68	890
61.84	61.19	61.22	3.28	3.76	872
65.12	64.47	64.50	3.28	3.10	1058
68.41	67.75	67.78	3.28	3.37	972
71.69	71.03	71.06	3.28	3.59	914
74.97	74.31	74.34	3.28	3.59	914
78.25	77.59	77.62	3.28	3.52	932
81.53	80.87	80.90	3.28	3.35	979
84.81	84.15	84.18	3.28	3.77	871
88.09	87.43	87.46	3.28	3.79	867
91.37	90.72	90.74	3.28	4.10	800
94.65	94.00	94.02	3.28	4.07	805
97.93	97.28	97.30	3.28	4.13	795
100.56	99.90	99.92	2.62	3.37	779

Appendix D

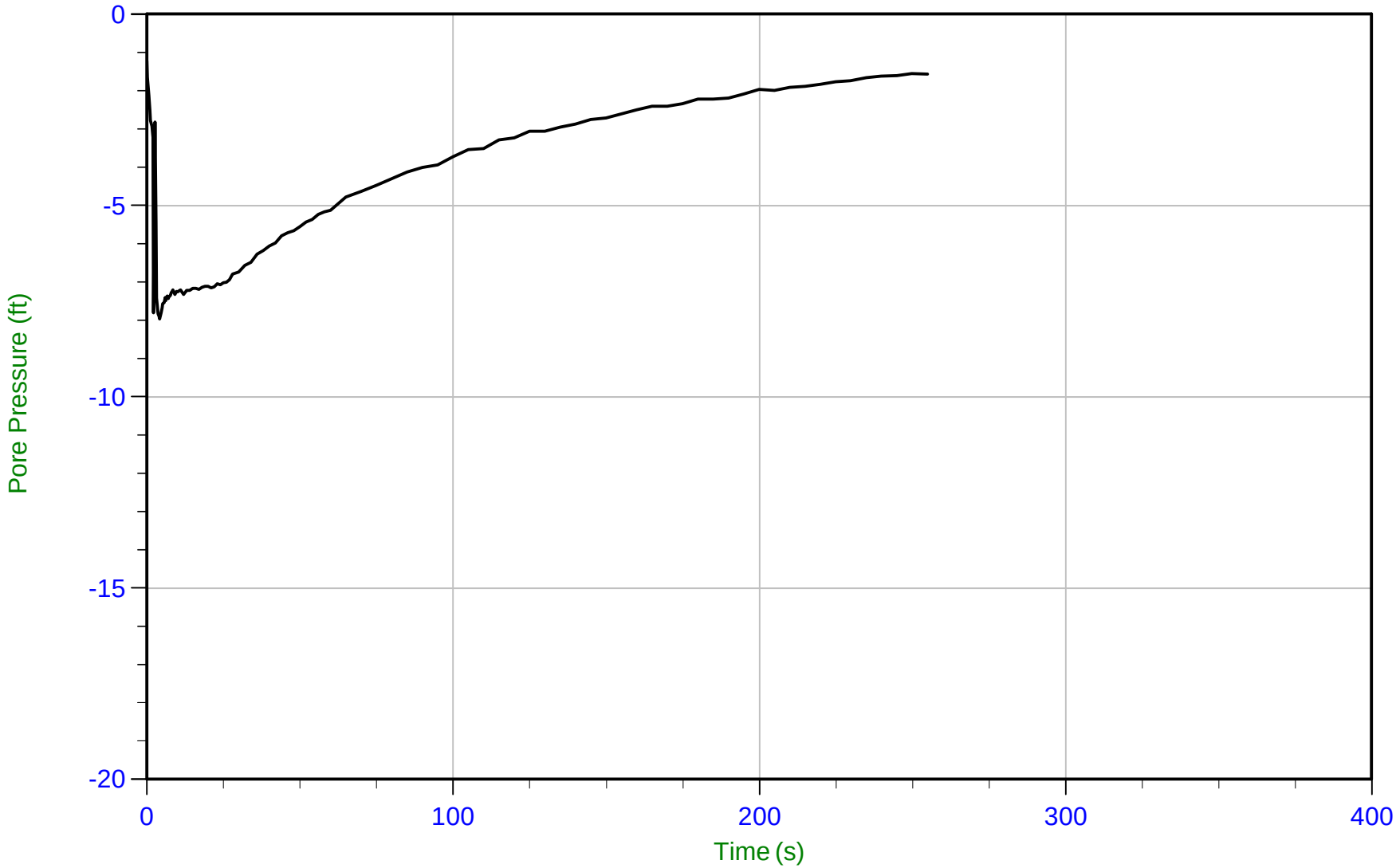
Pore Pressure Dissipation Test Results

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 07:41
Site: Rex Solaire Solar Project

Sounding: GEO-01
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP01.PPF2
Depth: 3.600 m / 11.811 ft
Duration: 255.0 s

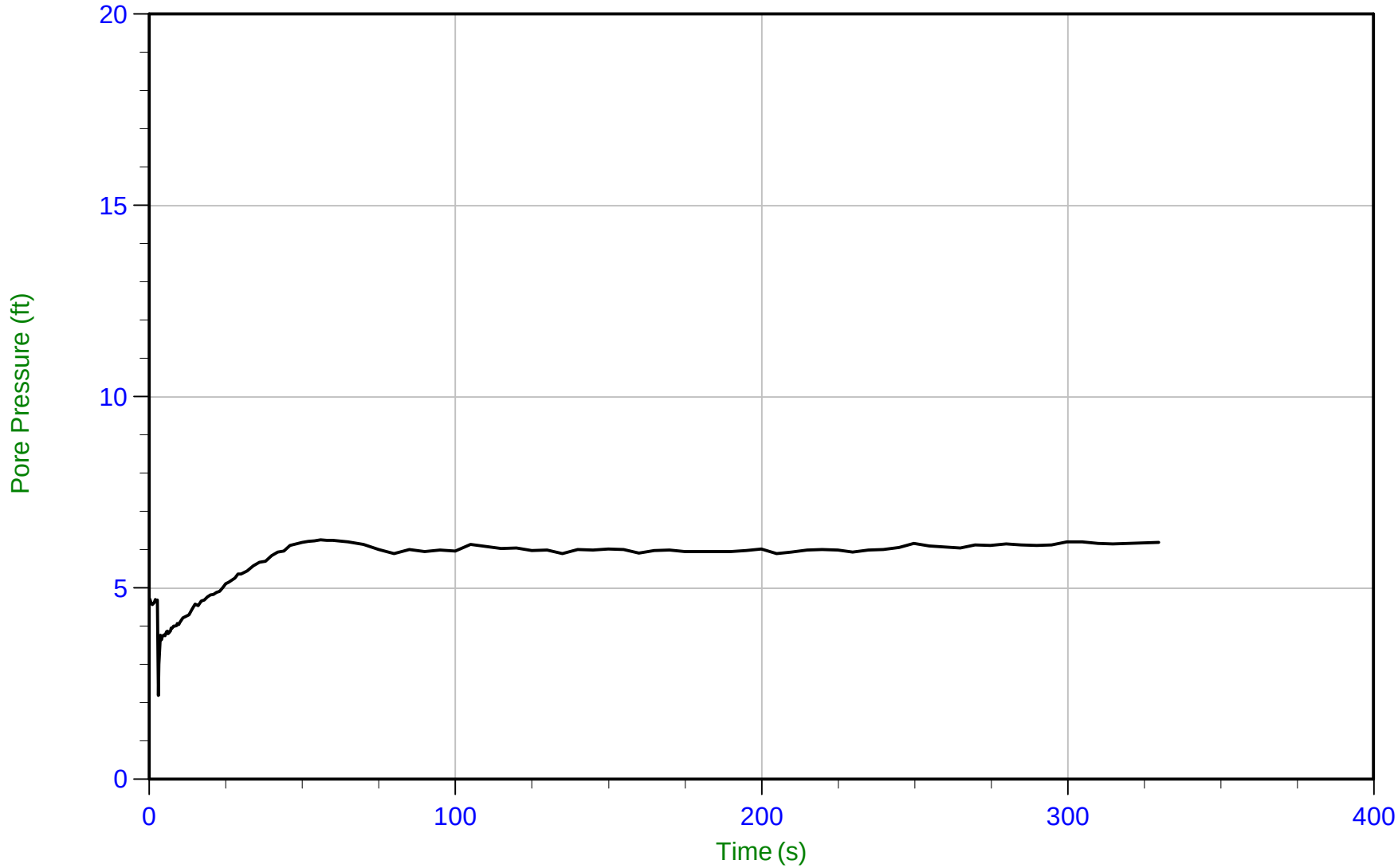
u Min: -8.0 ft
u Max: -1.2 ft
u Final: -1.6 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 07:41
Site: Rex Solaire Solar Project

Sounding: GEO-01
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP01.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 329.9 s

u Min: 2.2 ft
u Max: 6.3 ft
u Final: 6.2 ft

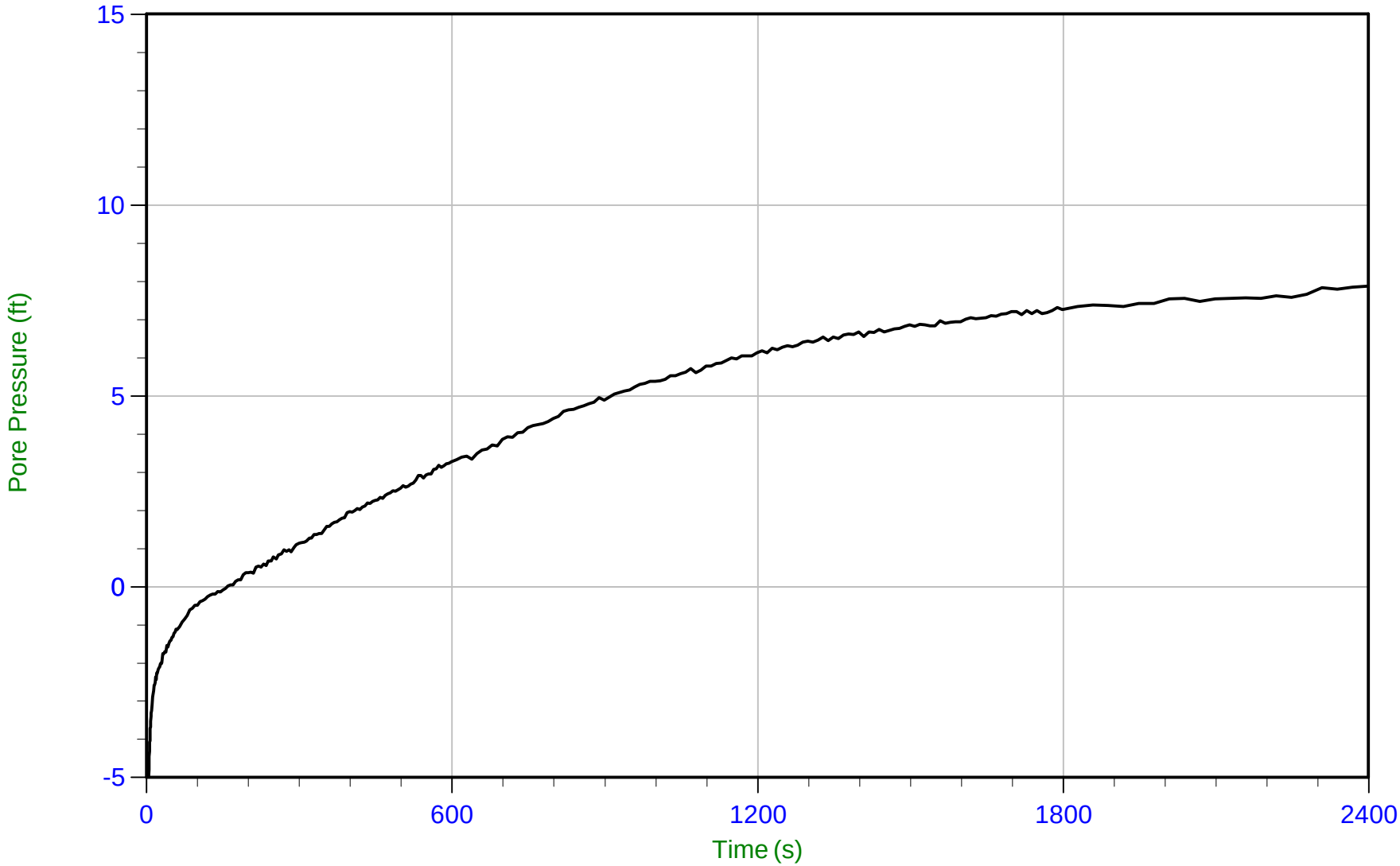
WT: 4.4 m / 14.3 ft
Ueq: 6.2 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-10 11:51
Site: Rex Solaire Solar Project

Sounding: GEO-02
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP02.PPF2
Depth: 3.575 m / 11.729 ft
Duration: 2399.6 s

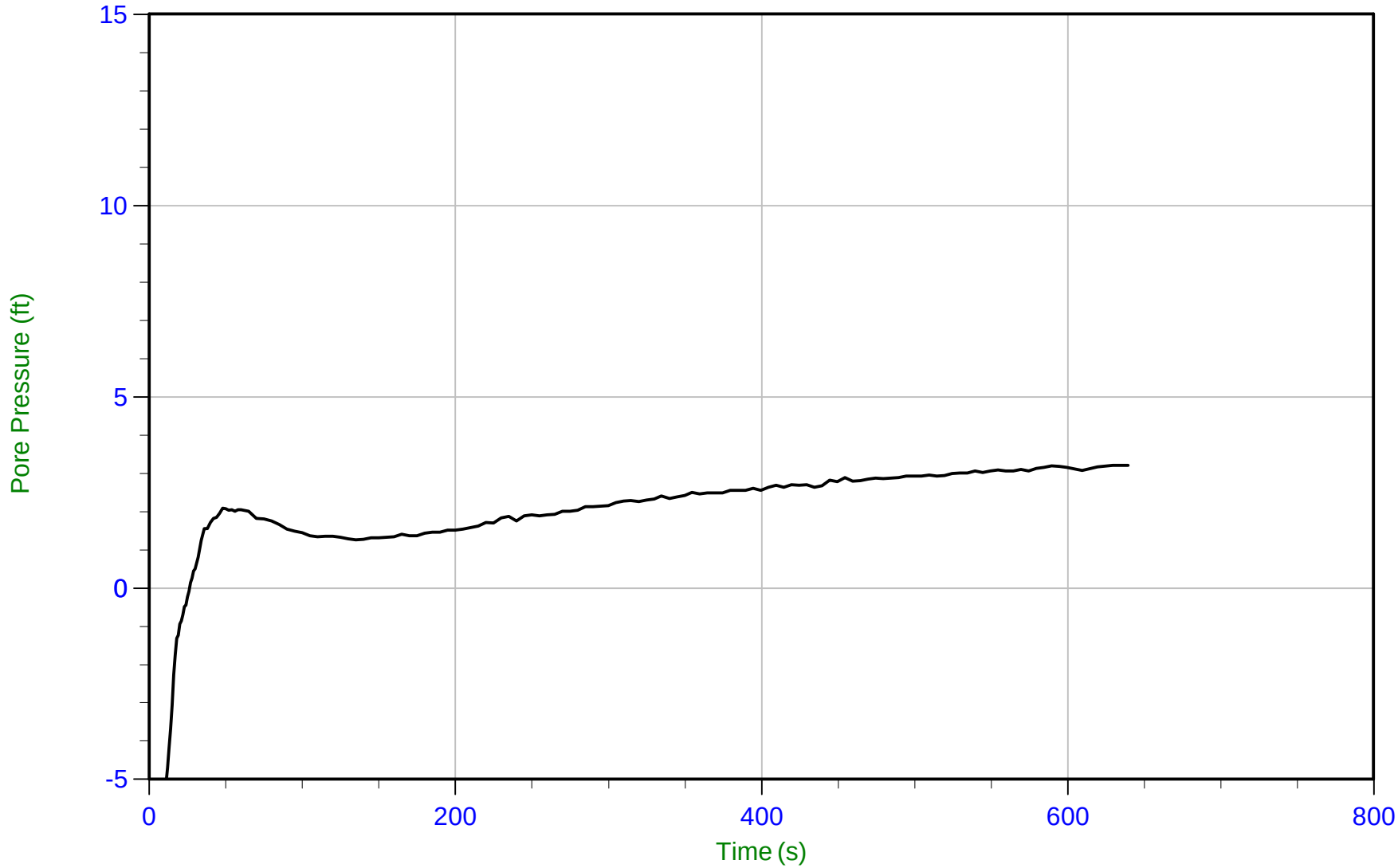
u Min: -6.3 ft
u Max: 7.9 ft
u Final: 7.9 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-10 10:37
Site: Rex Solaire Solar Project

Sounding: GEO-03
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP03.PPF2
Depth: 3.075 m / 10.088 ft
Duration: 639.9 s

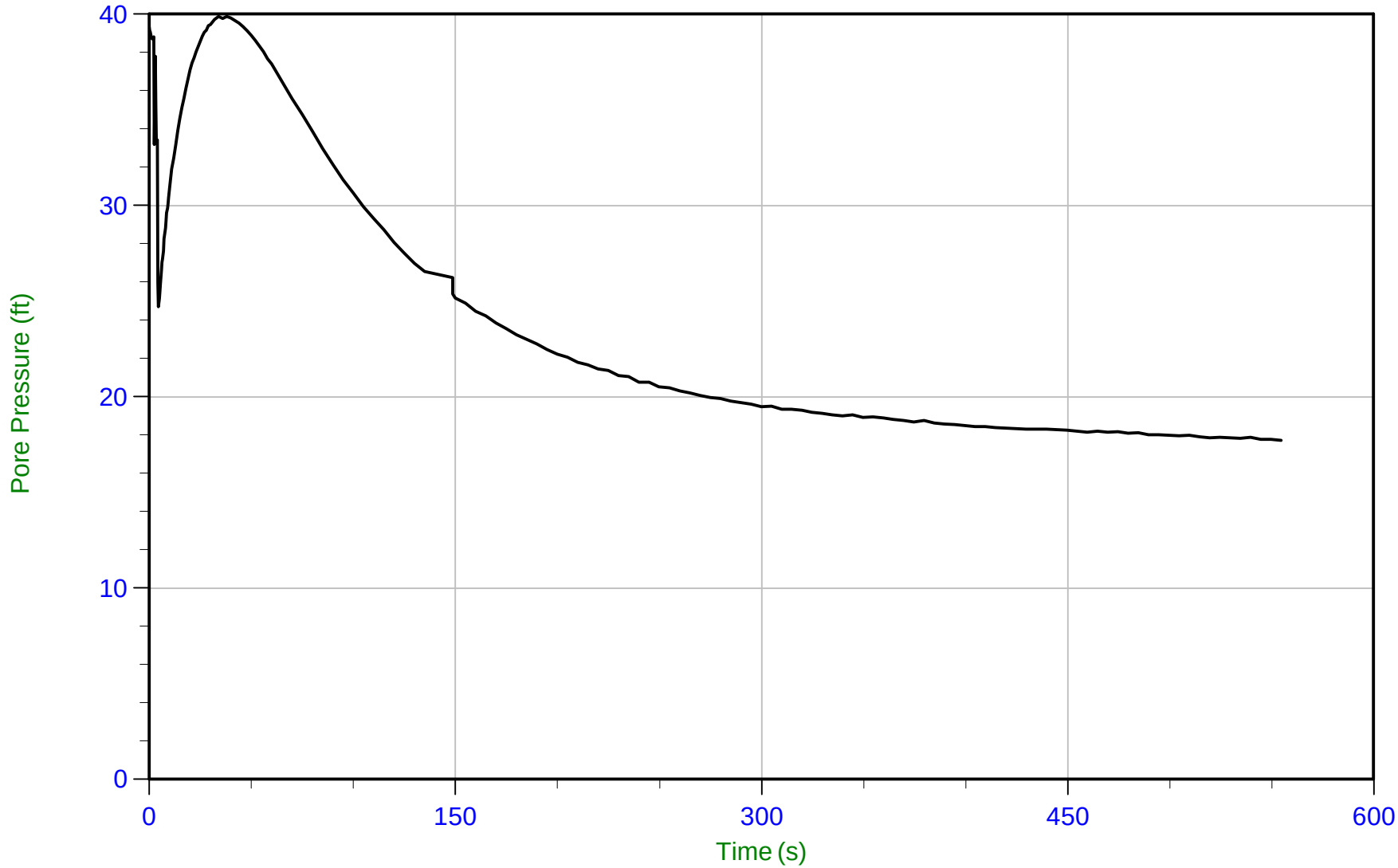
u Min: -7.1 ft
u Max: 3.2 ft
u Final: 3.2 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-10 07:44
Site: Rex Solaire Solar Project

Sounding: GEO-04A
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP04A.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 554.9 s

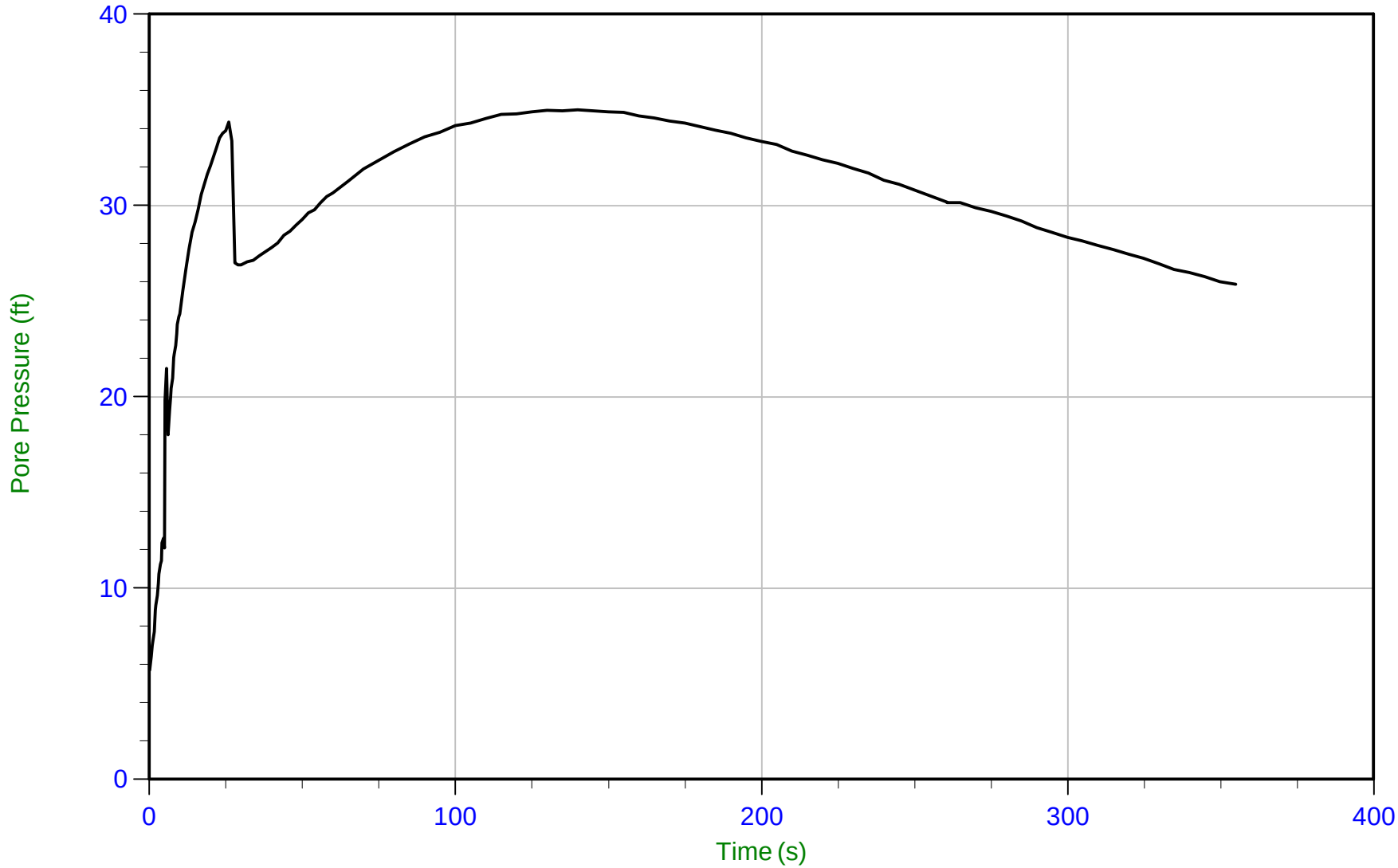
u Min: 17.7 ft
u Max: 39.9 ft
u Final: 17.7 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-10 09:05
Site: Rex Solaire Solar Project

Sounding: GEO-05
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP05.PPF2
Depth: 5.025 m / 16.486 ft
Duration: 355.0 s

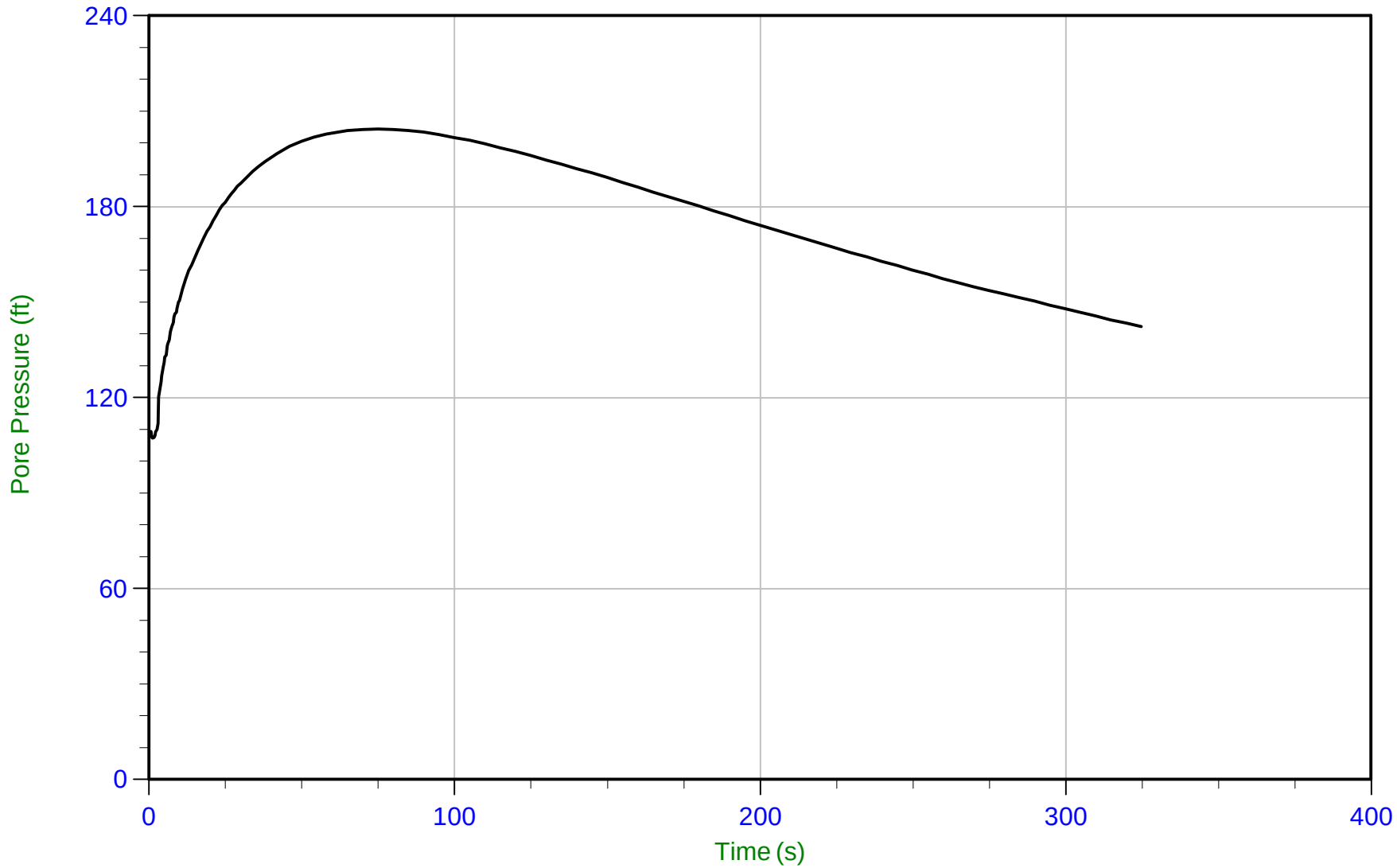
u Min: 5.7 ft
u Max: 35.0 ft
u Final: 25.9 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 09:23
Site: Rex Solaire Solar Project

Sounding: GEO-06A
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP06A.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 324.9 s

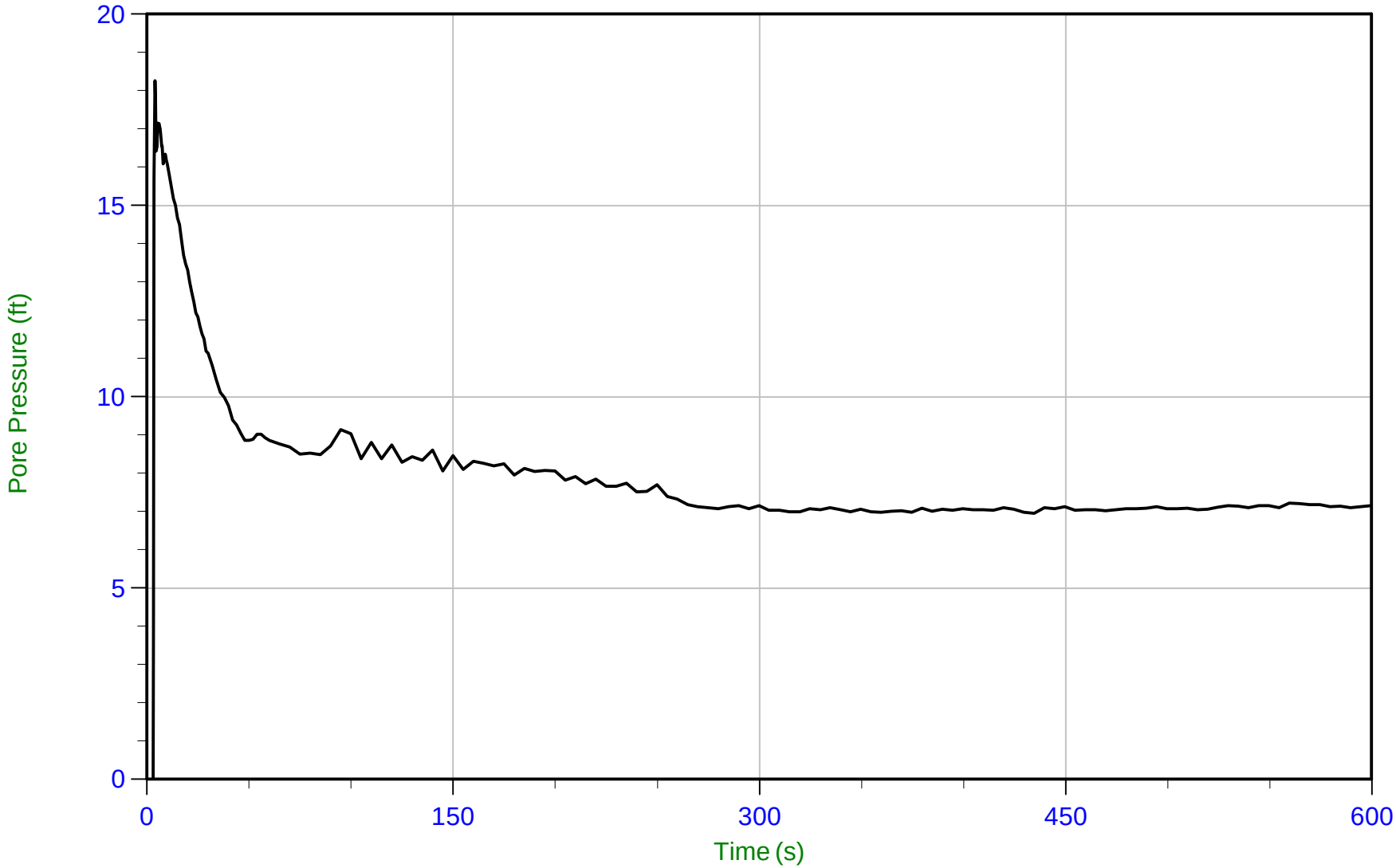
u Min: 107.2 ft
u Max: 204.4 ft
u Final: 142.3 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 12:52
Site: Rex Solaire Solar Project

Sounding: GEO-07A
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP07A.PPF2
Depth: 3.750 m / 12.303 ft
Duration: 599.9 s

u Min: -2.7 ft
u Max: 18.3 ft
u Final: 7.2 ft

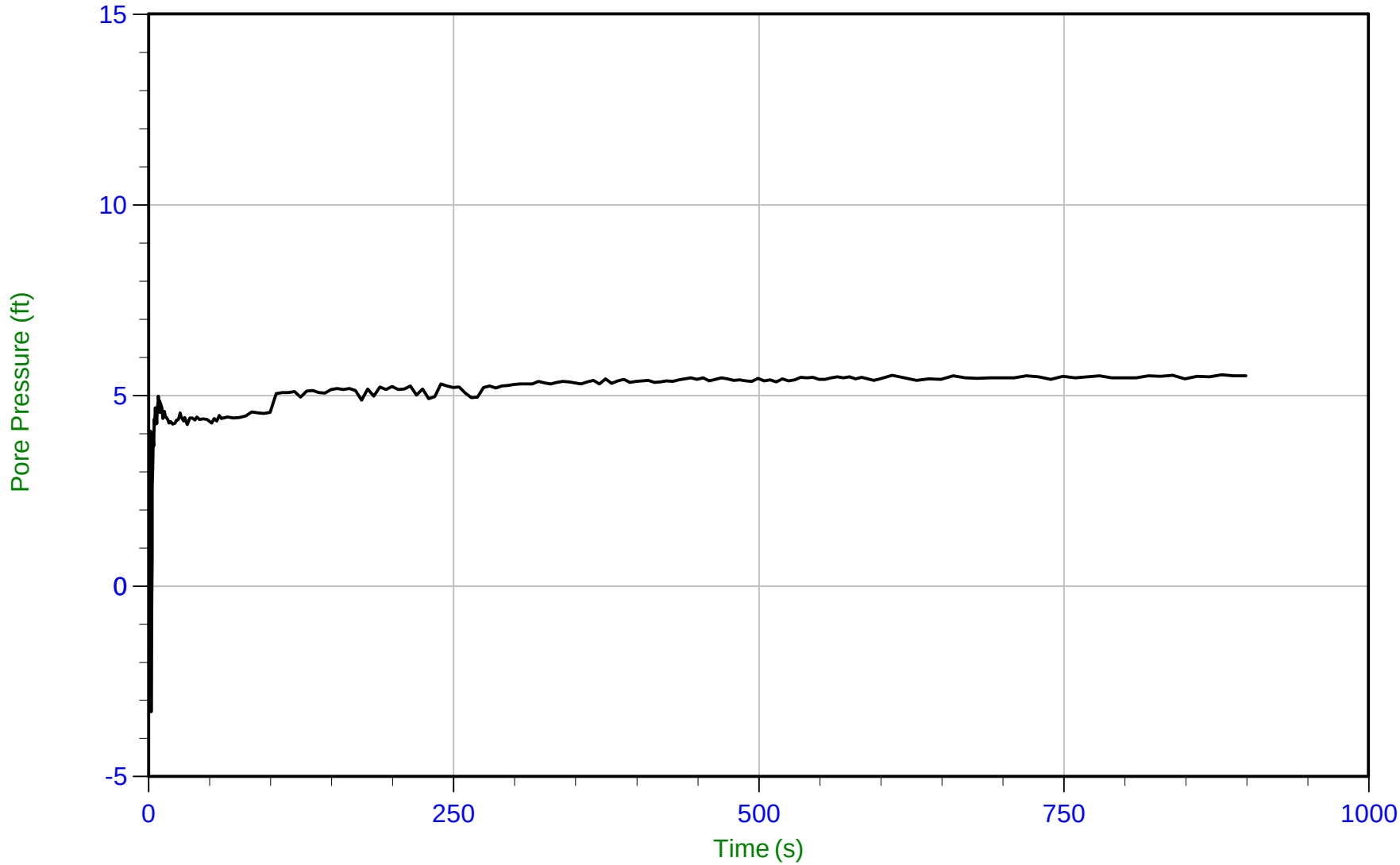
WT: 1.6 m / 5.1 ft
Ueq: 7.2 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 11:44
Site: Rex Solaire Solar Project

Sounding: GEO-08A
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP08A.PPF2
Depth: 3.425 m / 11.237 ft
Duration: 899.9 s

u Min: -3.3 ft
u Max: 5.5 ft
u Final: 5.5 ft

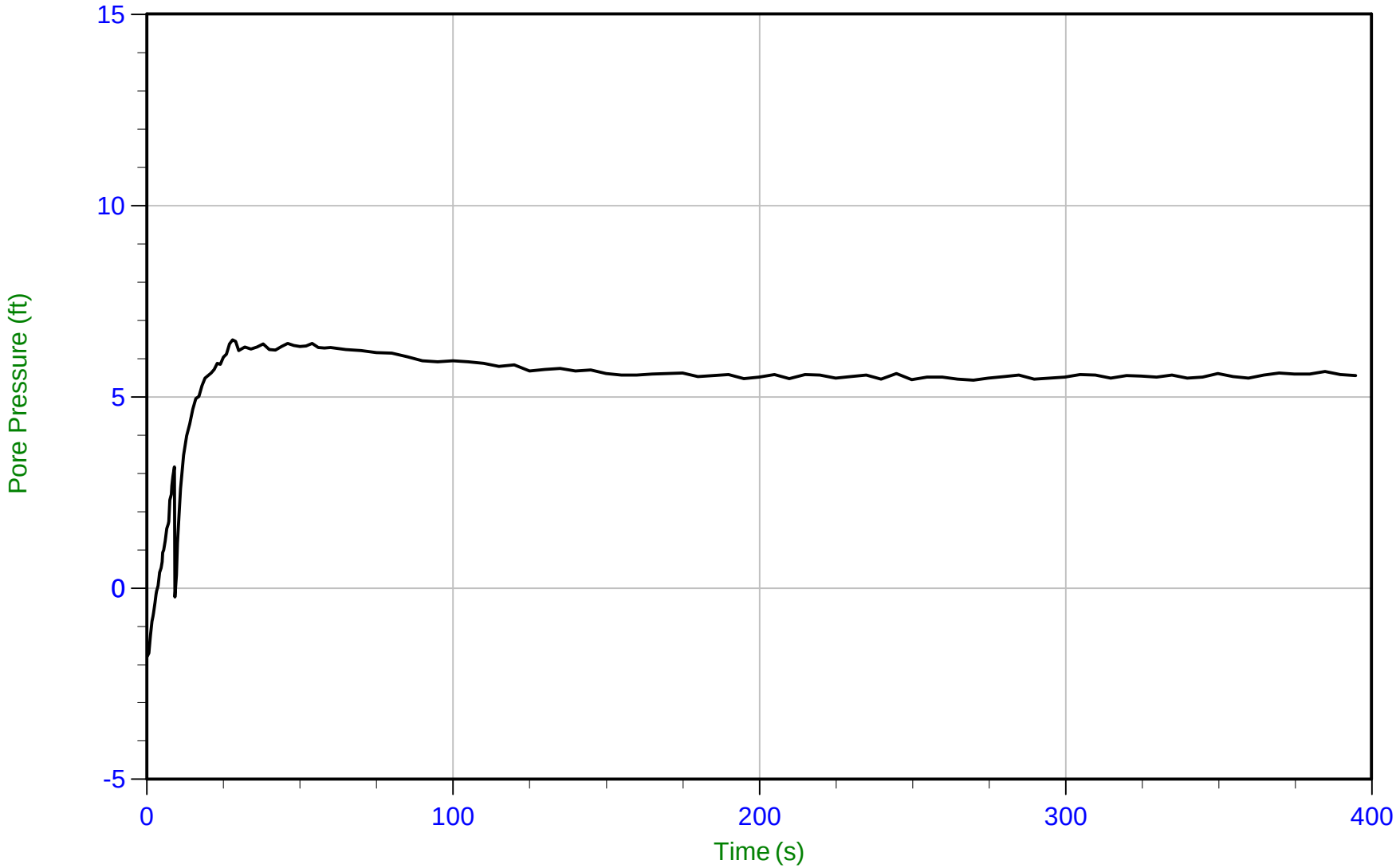
WT: 1.8 m / 5.7 ft
Ueq: 5.5 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 10:47
Site: Rex Solaire Solar Project

Sounding: GEO-09
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP09.PPF2
Depth: 4.200 m / 13.779 ft
Duration: 394.9 s

u Min: -1.8 ft
u Max: 6.5 ft
u Final: 5.5 ft

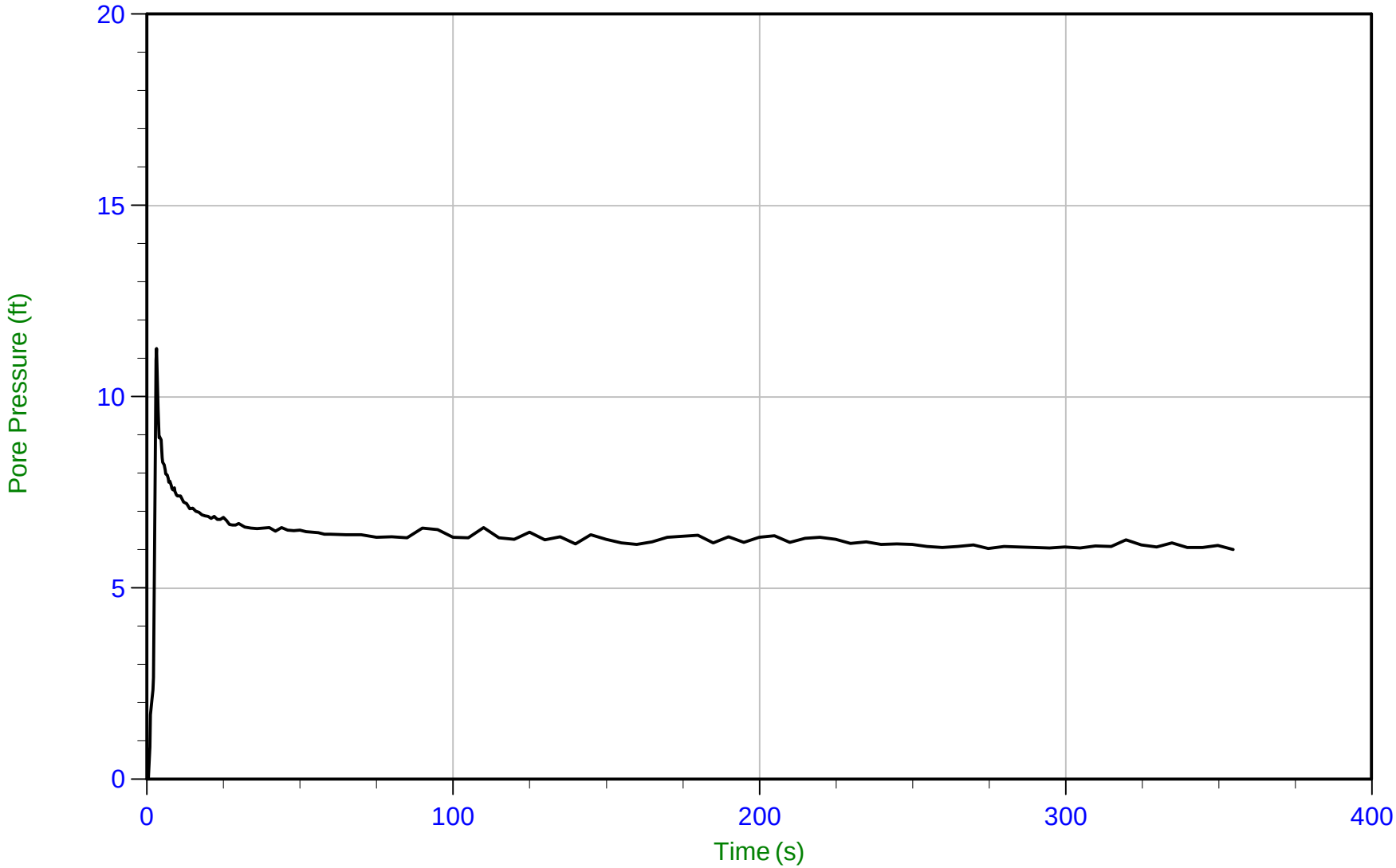
WT: 2.5 m / 8.3 ft
Ueq: 5.5 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 14:00
Site: Rex Solaire Solar Project

Sounding: GEO-10A
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP10A.PPF2
Depth: 3.125 m / 10.253 ft
Duration: 354.9 s

u Min: -0.3 ft
u Max: 11.3 ft
u Final: 6.0 ft

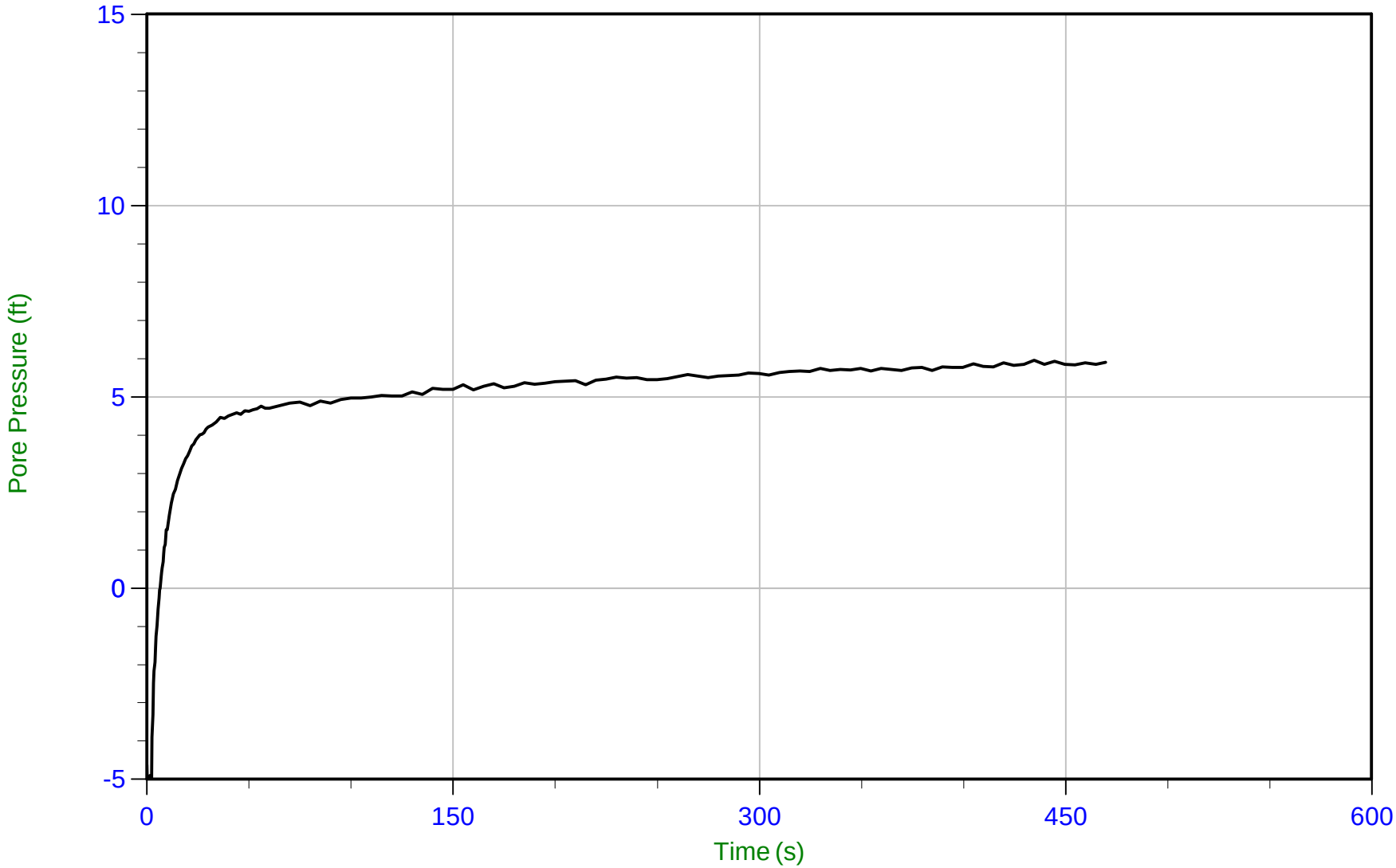
WT: 1.3 m / 4.2 ft
Ueq: 6.0 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-11 15:01
Site: Rex Solaire Solar Project

Sounding: GEO-11
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP11.PPF2
Depth: 3.450 m / 11.319 ft
Duration: 469.9 s

u Min: -5.5 ft
u Max: 5.9 ft
u Final: 5.9 ft

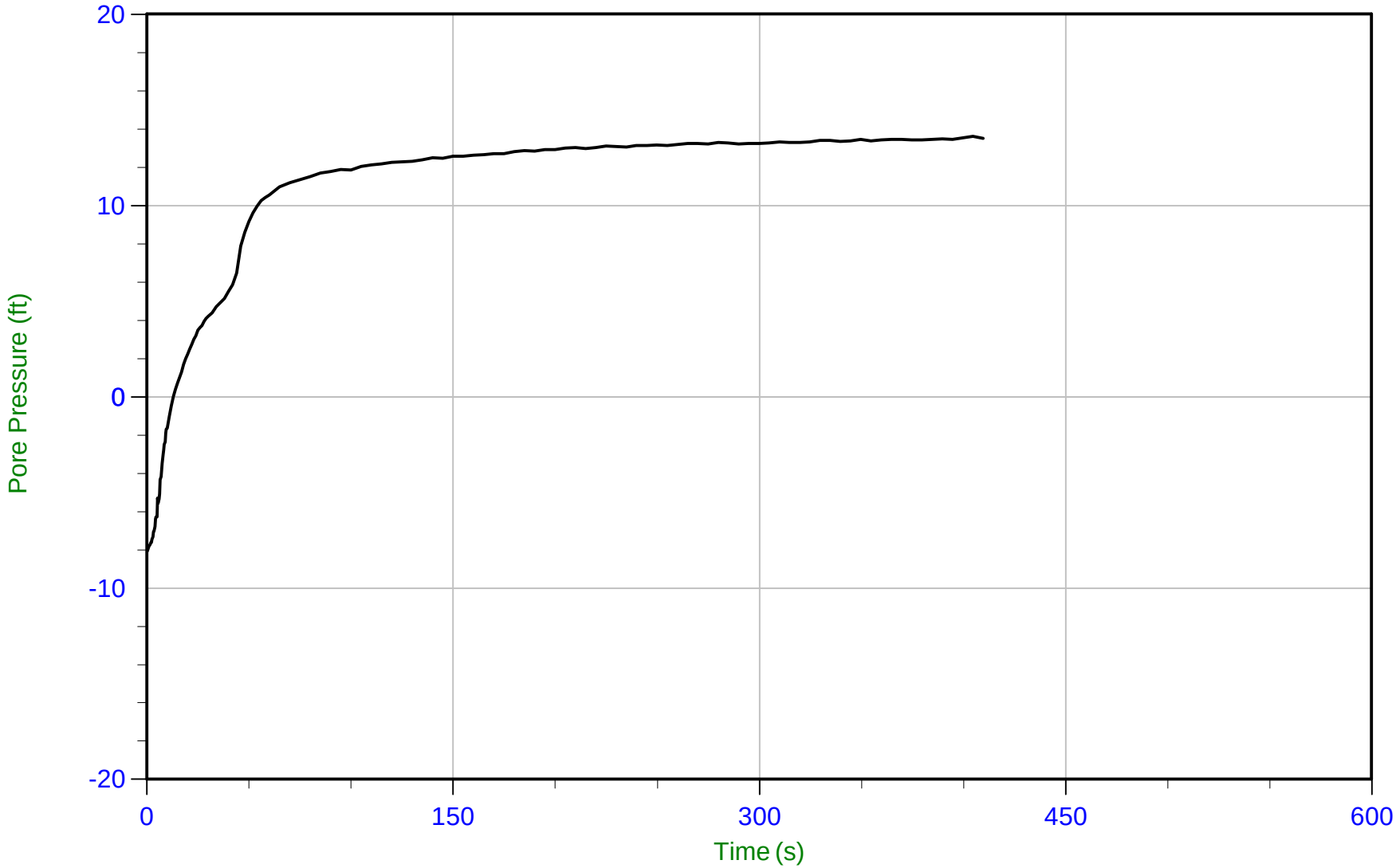
WT: 1.6 m / 5.4 ft
Ueq: 5.9 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 07:23
Site: Rex Solaire Solar Project

Sounding: GEO-12
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP12.PPF2
Depth: 5.500 m / 18.044 ft
Duration: 409.9 s

u Min: -8.1 ft
u Max: 13.6 ft
u Final: 13.5 ft

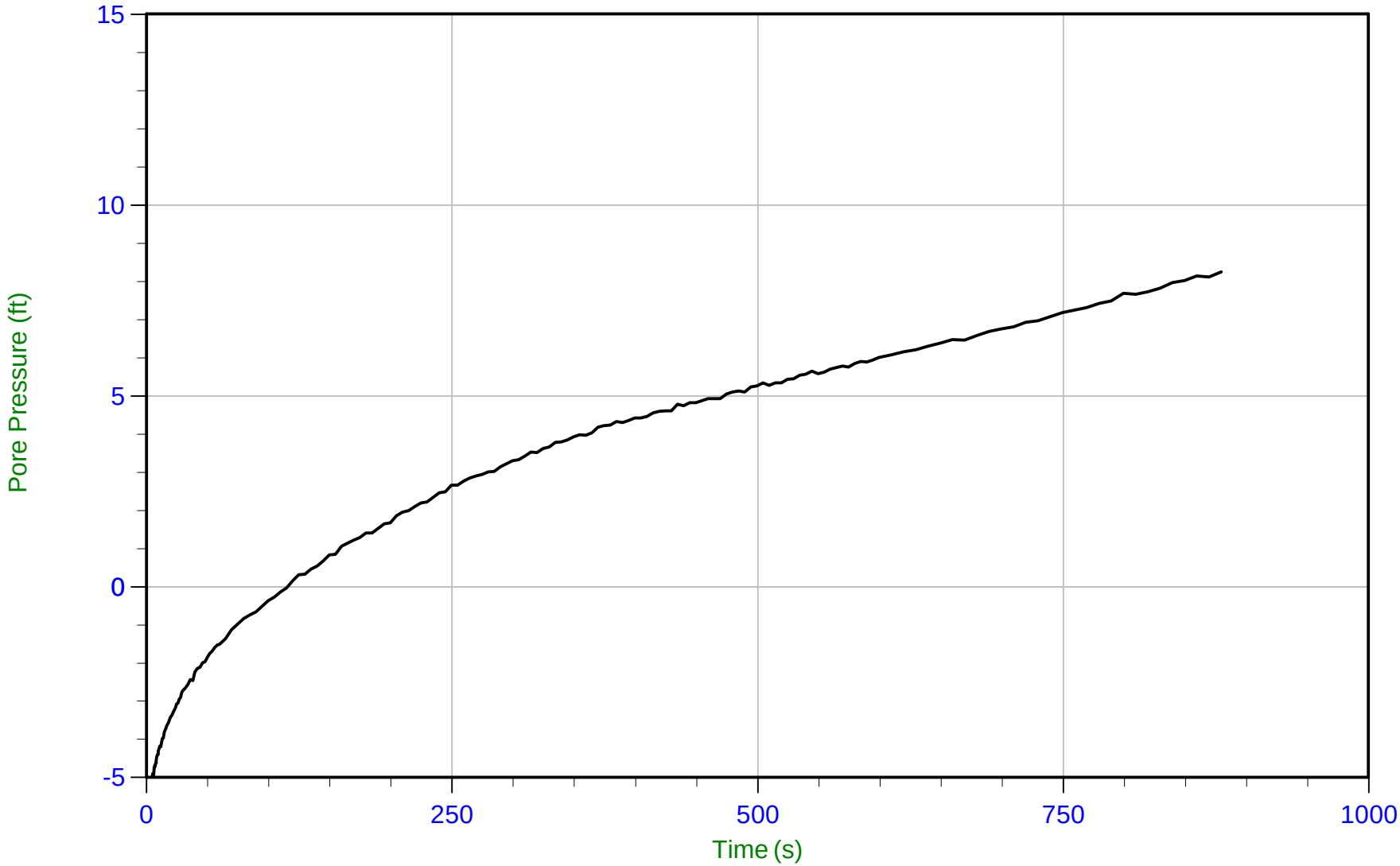
WT: 1.4 m / 4.6 ft
Ueq: 13.5 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 08:19
Site: Rex Solaire Solar Project

Sounding: GEO-13
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP13.PPF2
Depth: 5.825 m / 19.111 ft
Duration: 879.9 s

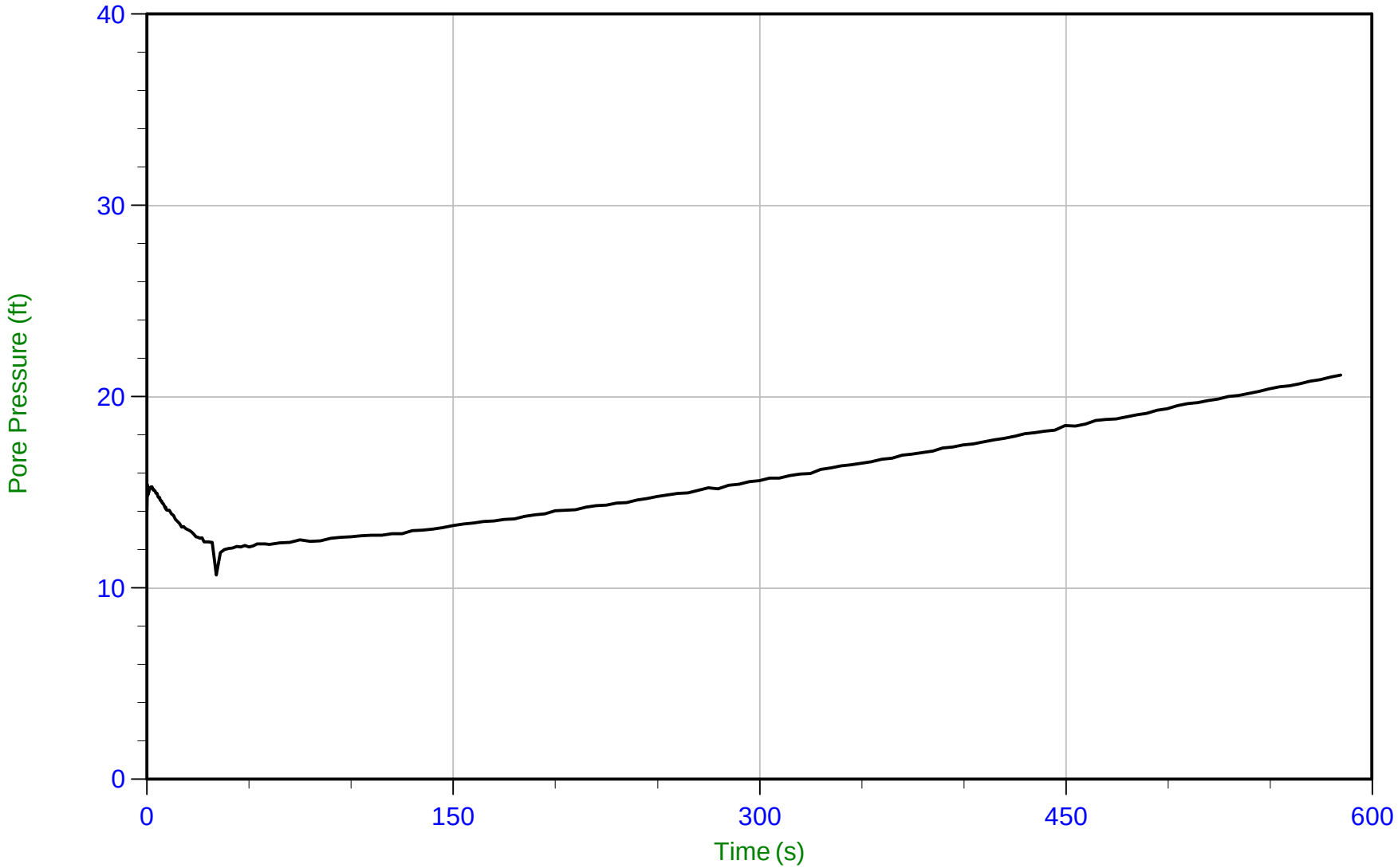
u Min: -5.2 ft
u Max: 8.3 ft
u Final: 8.3 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 09:21
Site: Rex Solaire Solar Project

Sounding: GEO-14
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP14.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 584.9 s

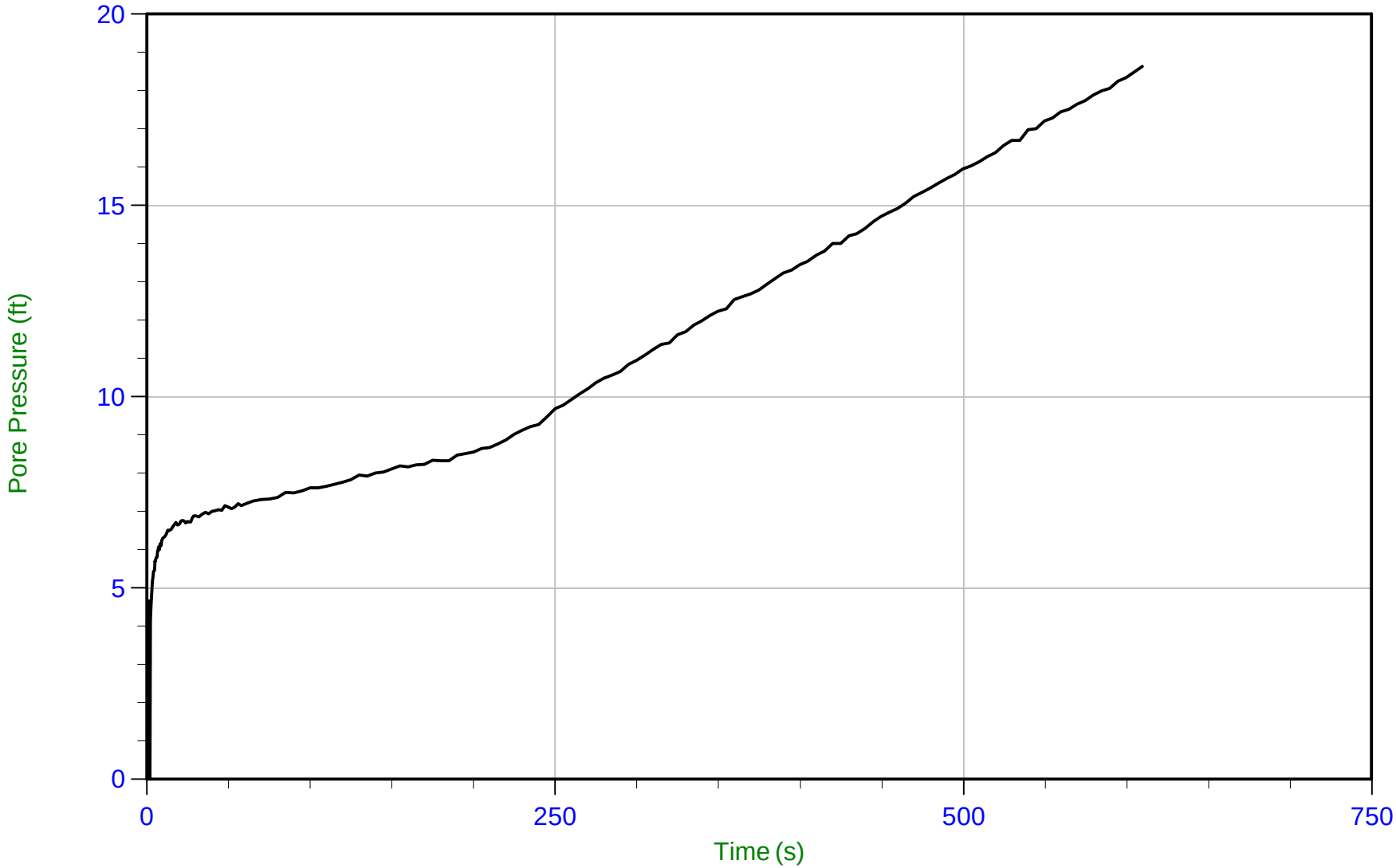
u Min: 10.7 ft
u Max: 21.1 ft
u Final: 21.1 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 11:26
Site: Rex Solaire Solar Project

Sounding: GEO-15
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP15.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 609.9 s

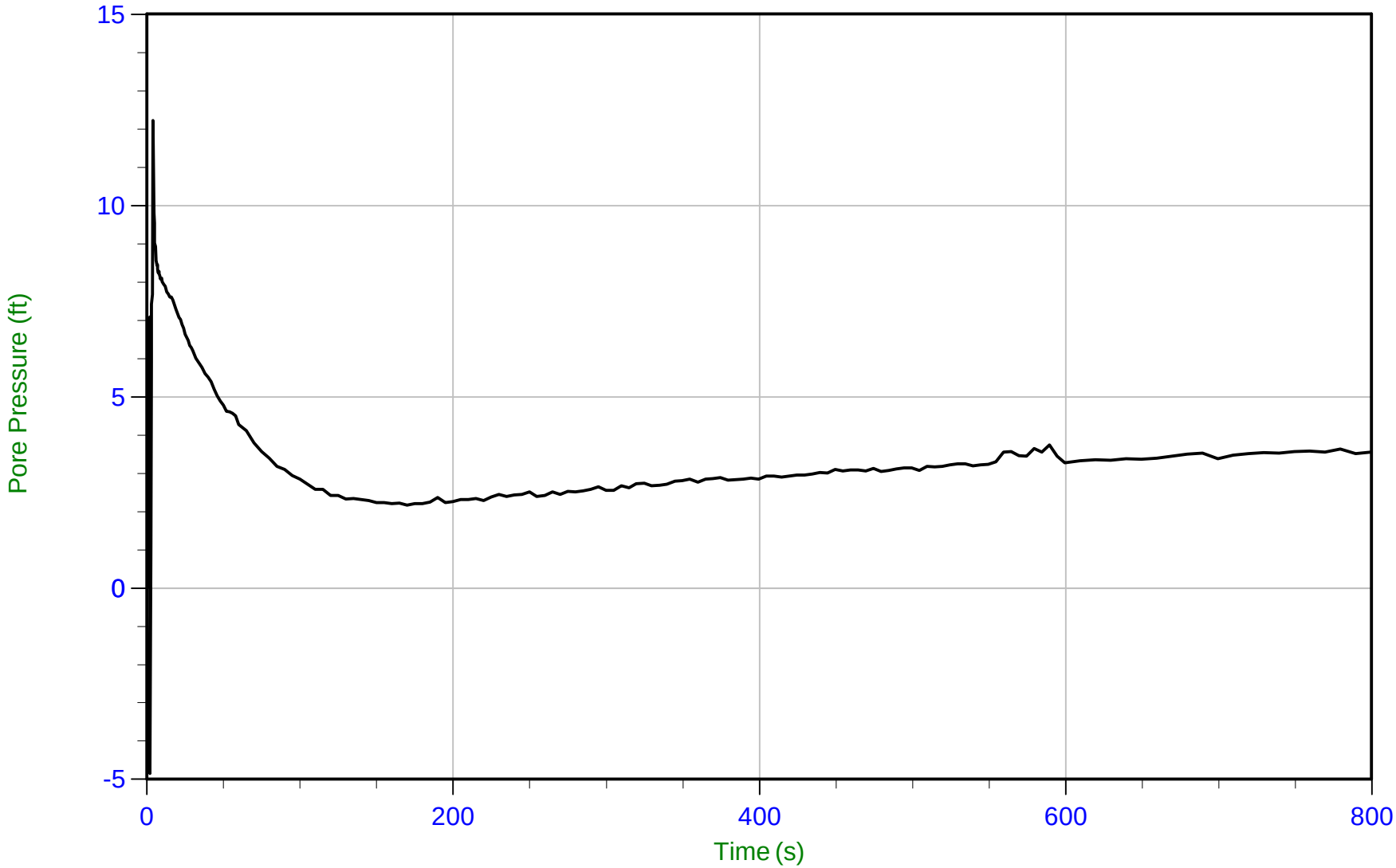
u Min: -0.2 ft
u Max: 18.6 ft
u Final: 18.6 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 14:01
Site: Rex Solaire Solar Project

Sounding: GEO-16
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_SP16.PPF2
Depth: 3.325 m / 10.909 ft
Duration: 799.9 s

u Min: -4.8 ft
u Max: 12.2 ft
u Final: 3.6 ft

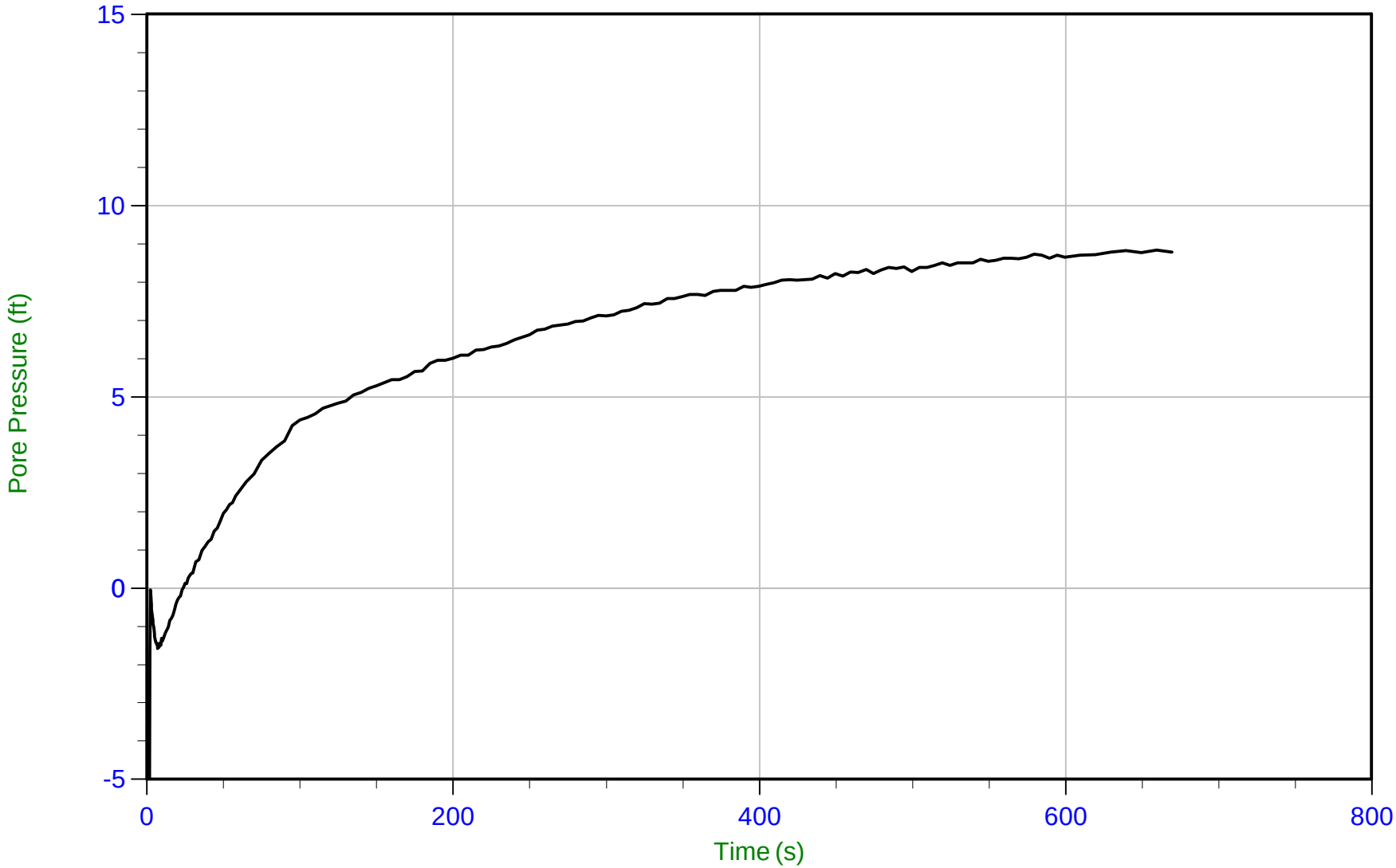
WT: 2.2 m / 7.3 ft
Ueq: 3.6 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 15:15
Site: Rex Solaire Solar Project

Sounding: GEO-17
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP17.PPF2
Depth: 4.975 m / 16.322 ft
Duration: 669.9 s

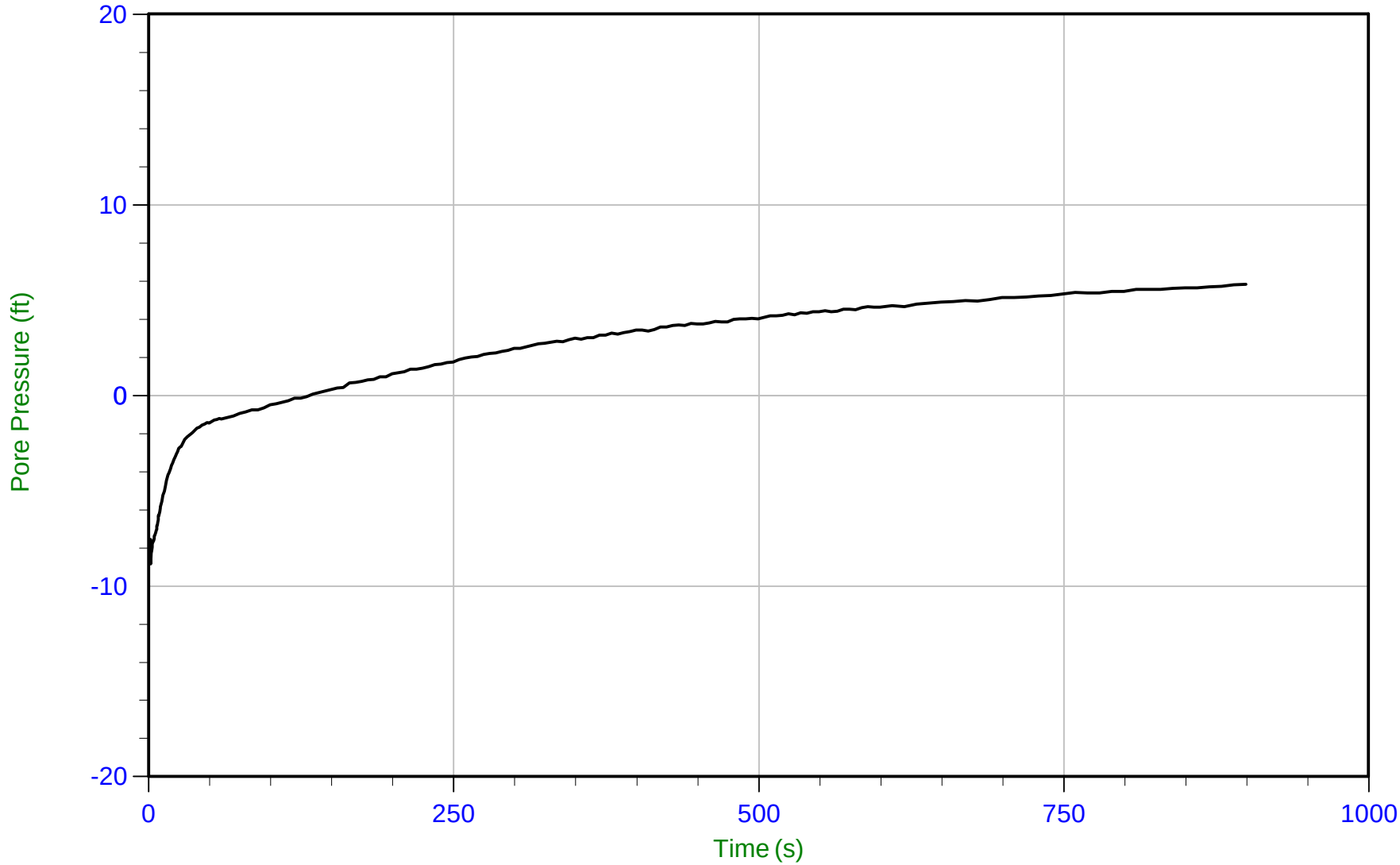
u Min: -6.5 ft
u Max: 8.8 ft
u Final: 8.8 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 12:55
Site: Rex Solaire Solar Project

Sounding: GEO-18
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP18.PPF2
Depth: 4.325 m / 14.189 ft
Duration: 899.9 s

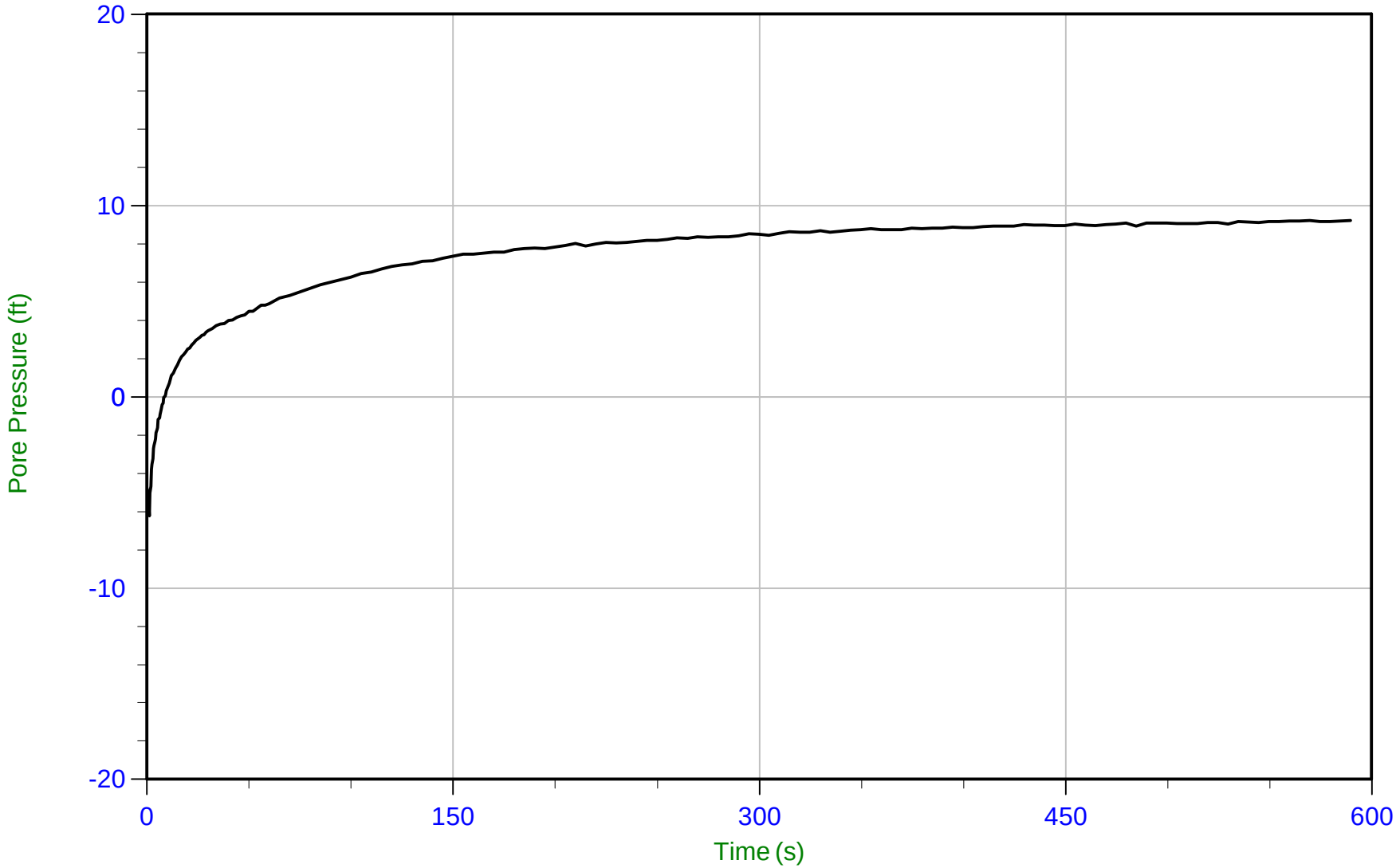
u Min: -8.8 ft
u Max: 5.8 ft
u Final: 5.8 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-12 12:55
Site: Rex Solaire Solar Project

Sounding: GEO-18
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP18.PPF2
Depth: 4.875 m / 15.994 ft
Duration: 589.9 s

u Min: -6.2 ft
u Max: 9.2 ft
u Final: 9.2 ft

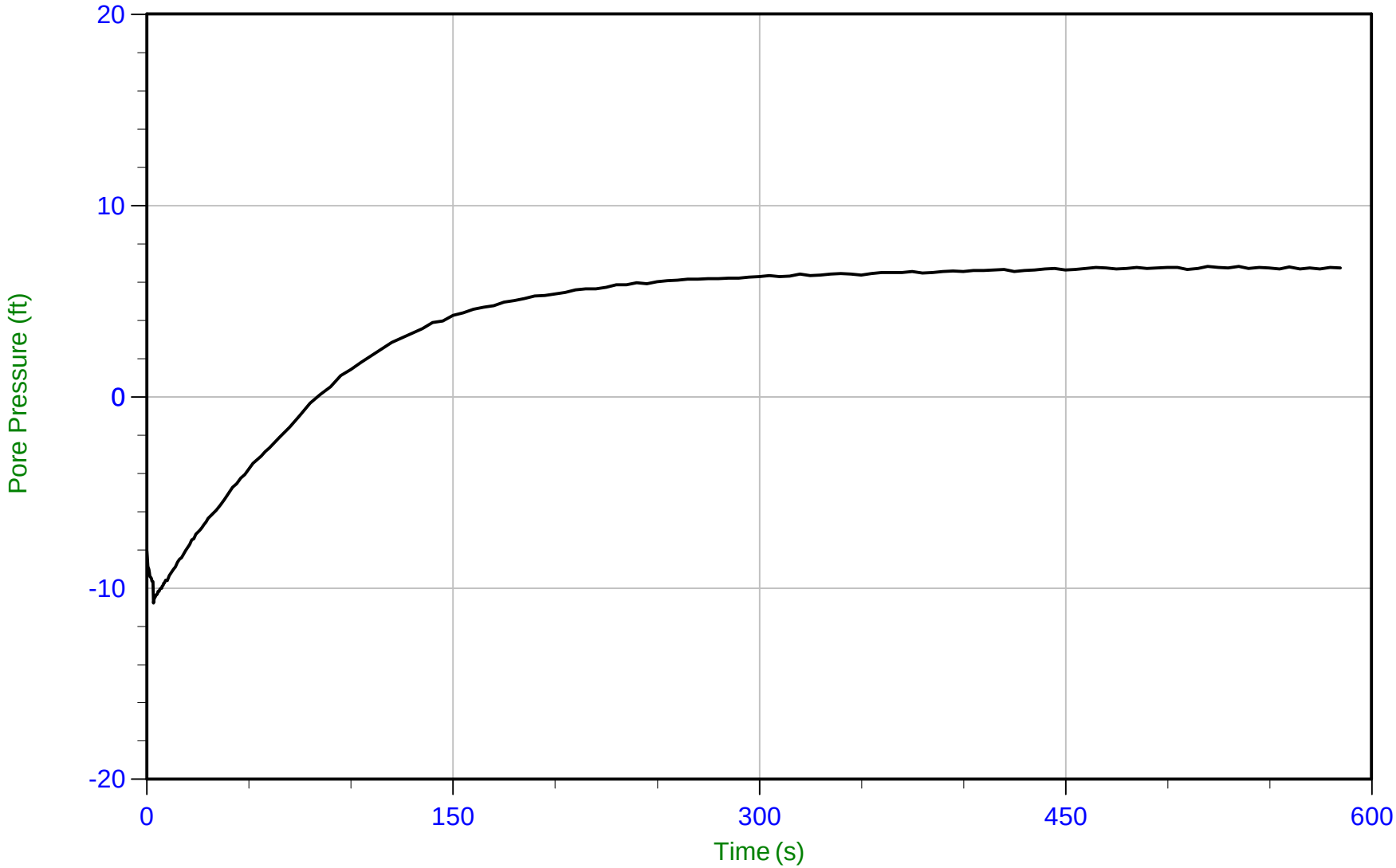
WT: 2.1 m / 6.8 ft
Ueq: 9.2 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-13 07:24
Site: Rex Solaire Solar Project

Sounding: GEO-19
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP19.PPF2
Depth: 3.825 m / 12.549 ft
Duration: 584.9 s

u Min: -10.8 ft
u Max: 6.8 ft
u Final: 6.7 ft

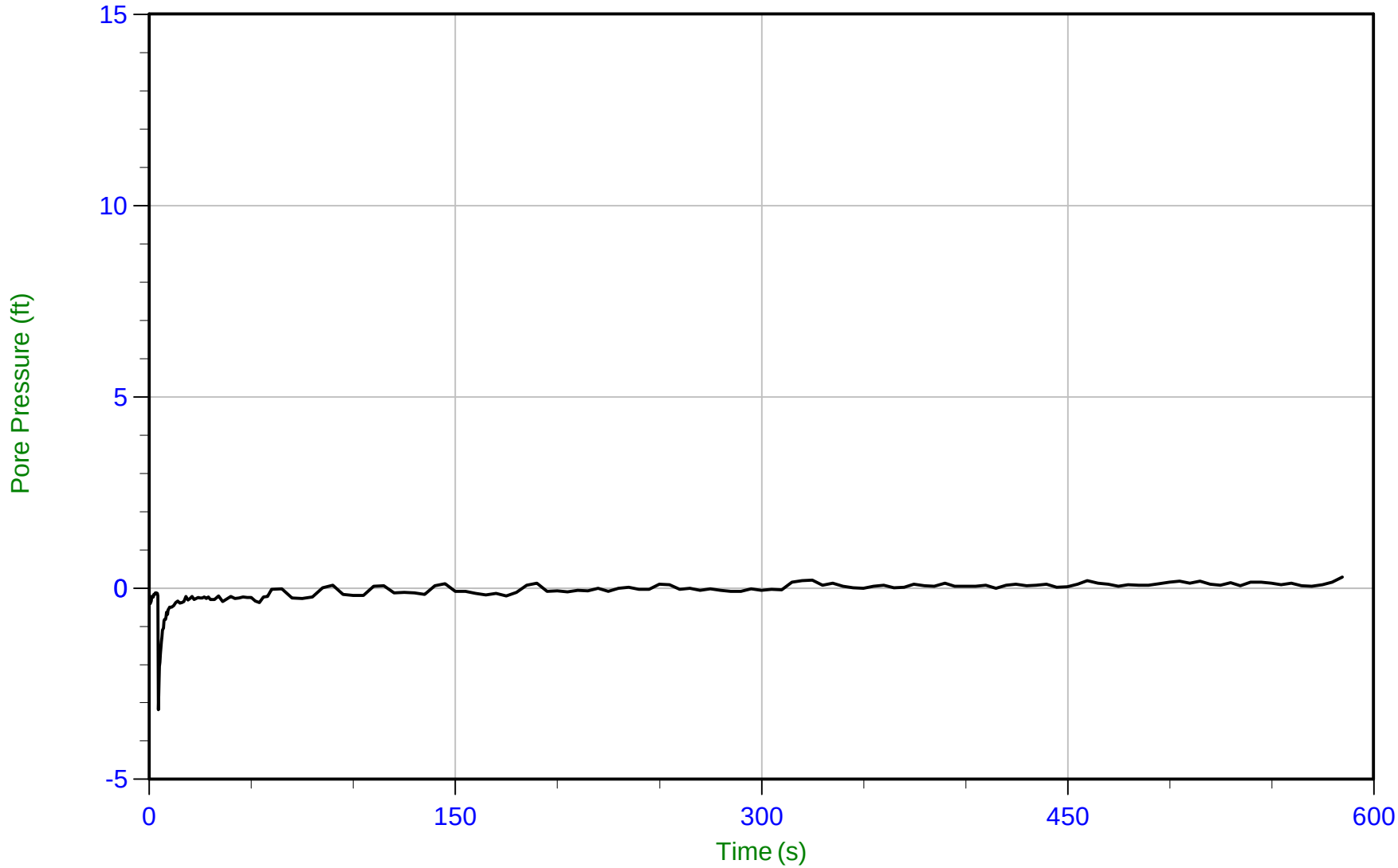
WT: 1.8 m / 5.8 ft
Ueq: 6.7 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-13 08:32
Site: Rex Solaire Solar Project

Sounding: GEO-20
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP20.PPF2
Depth: 3.550 m / 11.647 ft
Duration: 584.9 s

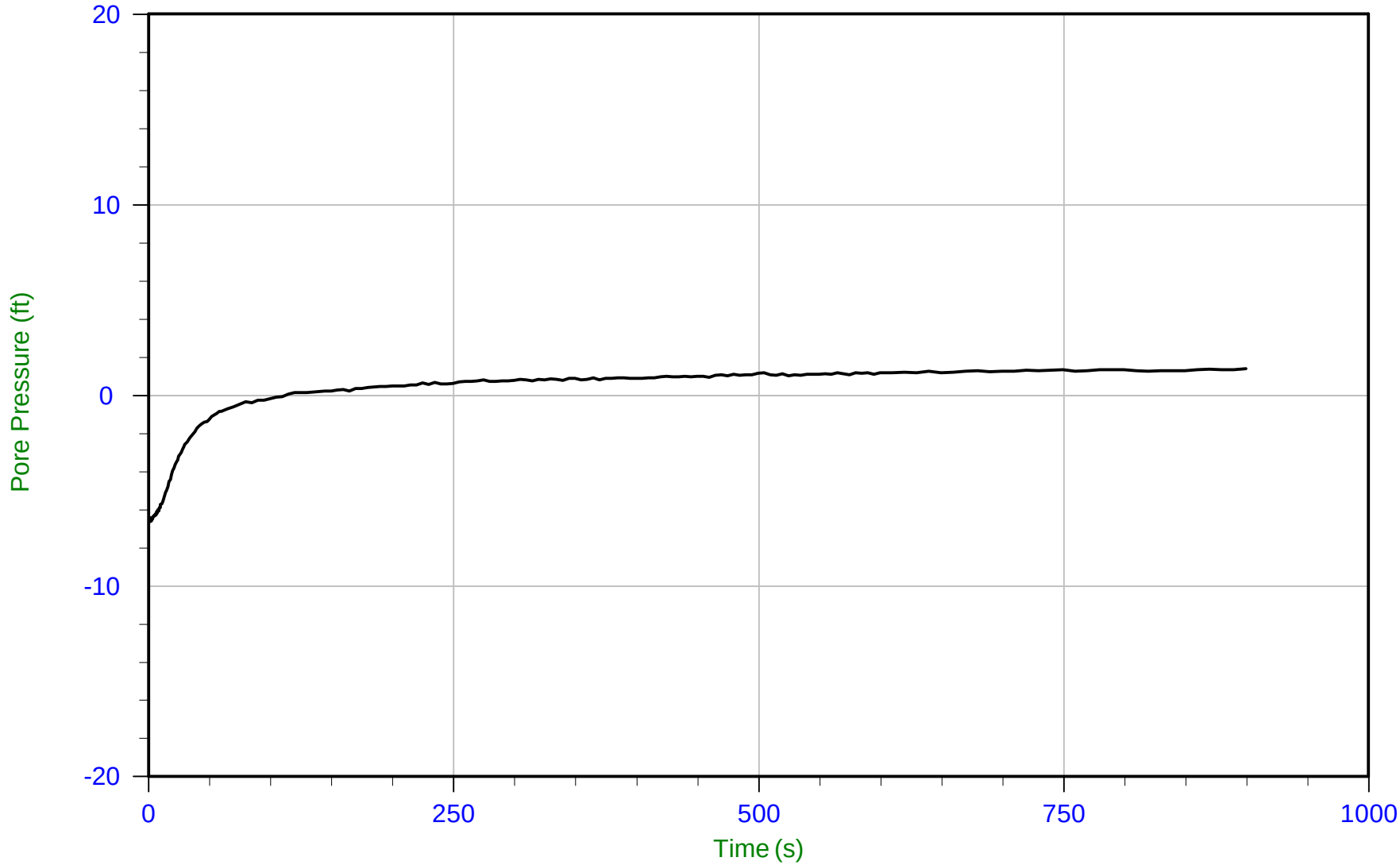
u Min: -3.2 ft
u Max: 0.3 ft
u Final: 0.3 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-13 09:31
Site: Rex Solaire Solar Project

Sounding: GEO-21
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_CP21.PPF2
Depth: 3.850 m / 12.631 ft
Duration: 899.9 s

u Min: -6.7 ft
u Max: 1.4 ft
u Final: 1.4 ft

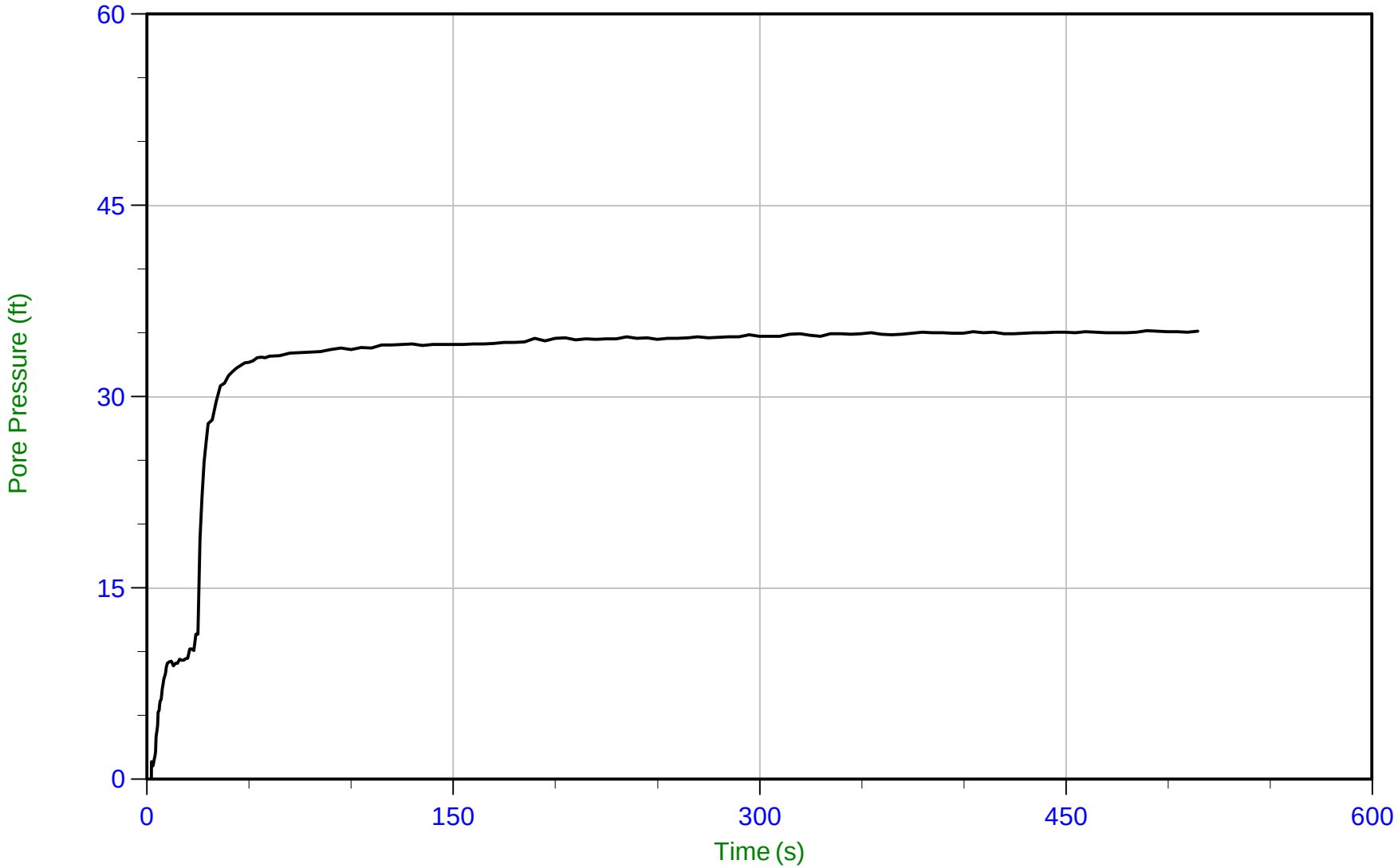
WT: 3.4 m / 11.3 ft
Ueq: 1.4 ft

CONETEC

Barr Engineering

Job No: 24-56-28295
Date: 2024-09-10 14:21
Site: Rex Solaire Solar Project

Sounding: Sub-Opt1-01
Cone: 817:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-56-28295_SP01Sub-Opt1-01.PPF2
Depth: 17.525 m / 57.496 ft
Duration: 514.9 s

u Min: -7.4 ft
u Max: 35.2 ft
u Final: 35.1 ft

WT: 6.8 m / 22.4 ft
Ueq: 35.1 ft

Appendix E

Test Pit Logs

Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-05

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.87547° Long: -119.79618°	Completion Depth:	10.0 ft
Datum:	WGS84		

Surface Elev.: Not Provided

Depth, feet	MATERIAL DESCRIPTION	Sample No.	% Recovery	SPT, N value or RQD %
0.0	TOPSOIL (SC): fine grained; gray; dry to moist; trace organics, trace white mineral inclusions. 0.5' - 1': trace shell fragments.	1	100	
			100	
1.0ft	SANDY LEAN CLAY (CL): gray; moist.	2	100	
2.5		3	100	
		4	100	
5.0				
7.5				
10.0	Bottom of Test Pit at 10.0 feet 10.0ft			

STANDARD PENETRATION TEST DATA N in blows/ft ◎ (10, 20, 30, 40)
REC%
RQD % ◆ (20, 40, 60, 80)
SHEAR STRENGTH, tsf
□ Qp/2 (0, 2.5, 5): 1.75

NATURAL DRY DENSITY (pcf) ☆ (80, 100, 120)
WATER CONTENT (%) × (PL, LL) (20, 40, 60)
5.5 (15, 31)
14 (39)
15.8
19.6
20.3

Date Test Pit Started:	9/9/24 5:21 pm	Water Levels (ft)	
Date Test Pit Completed:	9/9/24 5:55 pm	▽ At Time of Drilling	10.0
Logged By:	CJA2		
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		

Remarks:

Weather: 108F, Clear

M:\GINT\PROJECTS\05161003 REX SOLAIRE.GPJ BARRLIBRARY.GLB BOREHOLE LOG REPORT BARR TEMPLATE.GDT

Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-10A

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.90518° Long: -119.81643°	Completion Depth:	10.0 ft
Datum:	WGS84		

Surface Elev.: Not Provided

Depth, feet	MATERIAL DESCRIPTION	Sample No.	% Recovery	SPT, N value or RQD %	Natural Dry Density (pcf) / Water Content (%)
0.0	TOPSOIL (SC): fine grained; tan; dry to moist; trace organics.	1	100		10.7
	CLAYEY SAND (SC): fine grained; tan to brown; moist; trace gravel. 1.0ft	2	100		13.7
2.5		3	100		15.3
	SILTY SAND (SM): fine grained; brown; wet. 3.0ft	4	100		23.3
	4': excavation sidewalls collapsing during excavation.		100		
5.0	5': visible groundwater seeping into test pit excavation.				
7.5					
10.0	Bottom of Test Pit at 10.0 feet 10.0ft				

STANDARD PENETRATION TEST DATA N in blows/ft ◎ (10, 20, 30, 40)
REC% ▬
RQD % ◆ (20, 40, 60, 80)
SHEAR STRENGTH, tsf (0, 2.5, 5)
NATURAL DRY DENSITY (pcf) ☆ (80, 100, 120)
WATER CONTENT (%) × PL LL (20, 40, 60)

Date Test Pit Started:	9/9/24 2:55 pm	Water Levels (ft)	
Date Test Pit Completed:	9/9/24 3:15 pm	▼ At Time of Drilling	5.0
Logged By:	CJA2		
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		

Remarks:

Weather: 106F, Clear

M:\GINT\PROJECTS\05161003 REX SOLAIRE.GPJ BARRLIBRARY.GLB BOREHOLE LOG REPORT BARR TEMPLATE.GDT

Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-13

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.91957° Long: -119.80772°	Completion Depth:	10.0 ft
Datum:	WGS84		

Surface Elev.: Not Provided

Depth, feet	MATERIAL DESCRIPTION		Sample No.	% Recovery	SPT, N value or RQD %
0.0	TOPSOIL (SM): fine grained; gray to brown; dry to moist.				
	SILTY SAND (SM): fine grained; tan to brown; moist.	0.5ft	1	100	
			2	100	
	2' - 2.5': trace shell fragments.				
2.5			3	100	
	LEAN CLAY (CL): tan to brown; moist; with sand.	3.5ft	4	100	
5.0					
7.5					
10.0	Bottom of Test Pit at 10.0 feet	10.0ft			

STANDARD PENETRATION TEST DATA N in blows/ft ◎ (10, 20, 30, 40)
REC%
RQD % ◆ (20, 40, 60, 80)
SHEAR STRENGTH, tsf (0, 2.5, 5)

NATURAL DRY DENSITY (pcf) ☆ (80, 100, 120)
WATER CONTENT (%) × (PL, LL; 20, 40, 60)

2.8
9.5
11.9
21.6

Date Test Pit Started:	9/9/24 3:33 pm
Date Test Pit Completed:	9/9/24 3:55 pm
Logged By:	CJA2
Drilling Contractor:	Brown's Construction Service, Inc.
Drill Rig:	Case 580N

Water Levels (ft)
▽ At Time of Drilling
Dry

Remarks:

Weather: 108F, Clear

M:\GINT\PROJECTS\05161003 REX SOLAIRE.GPJ BARRLIBRARY.GLB BOREHOLE LOG REPORT BARR TEMPLATE.GDT

Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-15

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.88317° Long: -119.77230°	Completion Depth:	10.0 ft
Datum:	WGS84		

Elevation, feet	Depth, feet	MATERIAL DESCRIPTION	Graphic Log	Samples	Sample No.	% Recovery	SPT, N value or RQD %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ / REC% / RQD % ◆ / SHEAR STRENGTH, tsf □ Qp/2	NATURAL DRY DENSITY (pcf) ☆ / WATER CONTENT (%) × / PL / LL
		Surface Elev.: Not Provided							
	0.0	TOPSOIL (CH): gray; dry to moist; trace organics; trace sand.							
		LEAN CLAY (CL): gray; dry to moist; trace sand; trace white mineral inclusions. 0.5ft			1	100			10.4
		FAT CLAY (CH): brown; dry to moist; trace to with sand. 1.5ft			2	100			24.7
	2.5				3	100		0.75	17 54 21.5
					4	100 100		0.75	26.5 50
	5.0								
	7.5	SILTY SAND (SM): fine grained; tan to gray; moist to wet. 8.0ft							
▼		9': visible groundwater seeping into test pit excavation.							
	10.0	Bottom of Test Pit at 10.0 feet 10.0ft							

Date Test Pit Started:	9/9/24 1:39 pm	Water Levels (ft)	Remarks:
Date Test Pit Completed:	9/9/24 2:12 pm	▼ At Time of Drilling 9.0	
Logged By:	CJA2		
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		Weather: 110F, Clear

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Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-16

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.88327° Long: -119.73570°	Completion Depth:	10.3 ft
Datum:	WGS84		

Surface Elev.: Not Provided

Depth, feet	MATERIAL DESCRIPTION	Sample No.	% Recovery	SPT, N value or RQD %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ / REC% / RQD % ◆	NATURAL DRY DENSITY (pcf) ☆ / WATER CONTENT (%) ×
0.0	TOPSOIL (SM): fine grained; tan to brown; dry to moist; trace organics.	1	100			12
	LEAN CLAY (CL): tan to brown; dry to moist. 1.0ft	2	100			15.1
	SANDY SILT (ML): tan to gray; dry to moist. 2.0ft	3	100			9.7
2.5		4	100			16.5
			100			
5.0						
	7': excavation sidewalls collapsing during excavation.					
7.5						
	9.5': visible groundwater seeping into test pit excavation.					
10.0	Bottom of Test Pit at 10.3 feet 10.3ft					

SHEAR STRENGTH, tsf: 0 / 2.5 / 5

PL 21 LL 34 (20 / 40 / 60)

Date Test Pit Started:	9/9/24 11:33 am	Water Levels (ft)	
Date Test Pit Completed:	9/9/24 12:06 pm	▽ At Time of Drilling	9.5
Logged By:	CJA2		
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		

Remarks:

Weather: 99F, Clear

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Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT GEO-20

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.91281° Long: -119.73573°	Completion Depth:	10.0 ft
Datum:	WGS84		

Elevation, feet	Depth, feet	MATERIAL DESCRIPTION	Thickness	Graphic Log	Samples	Sample No.	% Recovery	SPT, N value or RQD %
	0.0	Surface Elev.: Not Provided						
		TOPSOIL (CL-SC): fine grained; brown; dry to moist; trace organics.						
		SANDY LEAN CLAY (CL-SC): fine grained; brown to gray; dry to moist; medium stiff.	0.5ft			1	100	
						2	100	
		SILTY SAND (SM): fine grained; tan to gray; dry to moist; varying fines content.	2.0ft					
	2.5					3	100	
						4	100	
		FAT CLAY (CH): brown; moist; trace sand.	4.0ft				100	
	5.0							
	7.5							
	10.0	Bottom of Test Pit at 10.0 feet	10.0ft					

STANDARD PENETRATION TEST DATA N in blows/ft ◎ (10, 20, 30, 40)
REC%
RQD % ◆ (20, 40, 60, 80)
SHEAR STRENGTH, tsf
□ Qp/2 (0, 2.5, 5)
1

NATURAL DRY DENSITY (pcf) ☆ (80, 100, 120)
WATER CONTENT (%) ×
PL LL (20, 40, 60)
16 45
12.3
16.9
3.5
4.3 14 37

Date Test Pit Started:	9/9/24 12:28 pm	Water Levels (ft)	Remarks:
Date Test Pit Completed:	9/9/24 12:59 pm	▽ At Time of Drilling	
Logged By:	CJA2	▽ Dry	
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		Weather: 102F, Clear

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Barr Engineering Co.
4300 MarketPointe Drive Suite 200
Minneapolis, MN 55435
Telephone: 952-832-2600

LOG OF TEST PIT SUB-OPT1-01

Project:	Rex Solaire Solar Project	Surface Elevation:	Not Provided
Job No.:	05161003.00	Drilling Method:	Backhoe
Location:	Kings County, California	Sampling Method:	Grab
Coordinates:	Lat: 35.85983° Long: -119.82845°	Completion Depth:	10.0 ft
Datum:	WGS84		

Surface Elev.: Not Provided

Depth, feet	MATERIAL DESCRIPTION	Sample No.	% Recovery	SPT, N value or RQD %
0.0	TOPSOIL (SC): fine grained; tan; dry to moist; trace organics.	1	100	
	SANDY LEAN CLAY (CL): tan; dry to moist. 1.0ft	2	100	
2.5		3	100	
		4	100	
	SILTY SAND (SM): fine grained; tan; dry to moist. 4.0ft		100	
5.0	5': with interbedded clayey sand.			
7.5				
10.0	Bottom of Test Pit at 10.0 feet 10.0ft			

STANDARD PENETRATION TEST DATA N in blows/ft ◎ (10, 20, 30, 40)
REC%
RQD % ◆ (20, 40, 60, 80)
SHEAR STRENGTH, tsf (0, 2.5, 5)

NATURAL DRY DENSITY (pcf) ☆ (80, 100, 120)
WATER CONTENT (%) ×
PL LL (20, 40, 60)

4.4
6.2
13
13.2
17
29

Date Test Pit Started:	9/9/24 6:13 pm	Water Levels (ft)	Remarks:
Date Test Pit Completed:	9/9/24 6:36 pm	▽ At Time of Drilling	
Logged By:	CJA2	▽ Dry	
Drilling Contractor:	Brown's Construction Service, Inc.		
Drill Rig:	Case 580N		Weather: 108F, Clear

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Appendix F

Test Pit Photographs

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-05





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-05
Location, Latitude:	35.87547°
Location, Longitude:	-119.79618°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-10A





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-10A
Location, Latitude:	35.90518°
Location, Longitude:	-119.81643°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-13





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-13
Location, Latitude:	35.91957°
Location, Longitude:	-119.80772°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-15





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-15
Location, Latitude:	35.88317°
Location, Longitude:	-119.77230°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-16





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-16
Location, Latitude:	35.88327°
Location, Longitude:	-119.73570°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.3

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: GEO-20





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	GEO-20
Location, Latitude:	35.91281°
Location, Longitude:	-119.73573°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

TEST PIT PHOTOS: SUB-OPT1-01





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Test Pit Information

Investigation ID:	SUB-OPT1-01
Location, Latitude:	35.85983°
Location, Longitude:	-119.82845°
Coordinate System:	Latitude/Longitude DD
Datum:	WGS84
Date Completed:	9/9/2024
Logged By:	CJA2
Contractor:	Brown's Construction Service, Inc.
Equipment:	Case 580N

Test Pit Depth

Termination Depth (ft):	10.0

Comments:

Appendix G

General Soil Laboratory Test Results

Water Content Test Summary (ASTM:D2216)

Project: Rex Solaire Solar Job: 15366

Client Barr Engineering Company Date: 10/9/2024

Sample Information & Classification								
Boring #	Geo-05	Geo-05	Geo-05	Geo-05	Geo-10A	Geo-10A	Geo-10A	Geo-10A
Sample #								
Depth (ft)	0.5-1	1.5-2	2.5-3	3.5-4	0.5-1	1.5-2	2.5-3	3.5-4
Type	Bag	Bag	Bag	Bag	Bag	Bag	Bag	Bag
Material Classification	Sandy Lean Clay w/shell fragments (CL)	Sandy Lean Clay (CL)	Sandy Lean Clay (CL)	Sandy Lean Clay (CL)	Silty Clayey Sand (SC-SM/SC)	Silty Clayey Sand (SC-SM/SC)	Silty Sand w/a trace of gravel (SM/SC-SM)	Clayey Sand (SC)
Water Content (%)	5.5	15.8	19.6	20.3	10.7	13.7	15.3	23.8

Sample Information & Classification								
Boring #	Geo-13	Geo-13	Geo-13	Geo-13	Geo-15	Geo-15	Geo-15	Geo-15
Sample #								
Depth (ft)	0.5-1	1.5-2	2.5-3	3.5-4	0.5-1	1.5-2	2.5-3	3.5-4
Type	Bag	Bag	Bag	Bag	Bag	Bag	Bag	Bag
Material Classification	Silty Sand (SM/SP-SM)	Silty Sand w/shell fragments (SM)	Silty Sand (SM)	Lean Clay w/sand (CL/CH)	Lean Clay w/sand (CL)	Fat Clay (CH)	Fat Clay w/sand (CH)	Fat Clay (CH)
Water Content (%)	2.8	9.5	11.9	21.6	10.4	24.7	21.5	23.5

Sample Information & Classification								
Boring #	Geo-16	Geo-16	Geo-16	Geo-16	Geo-20	Geo-20	Geo-20	Geo-20
Sample #								
Depth (ft)	0.5-1	1.5-2	2.5-3	3.5-4	0.5-1	1.5-2	2.5-3	3.5-4
Type	Bag	Bag	Bag	Bag	Bag	Bag	Bag	Bag
Material Classification	Lean Clay w/sand (CL)	Lean Clay (CL)	Sandy Silt (ML)	Silt w/sand and occasional pieces of clay (ML)	Lean Clay w/sand (CL)	Lean Clay w/sand (CL)	Sand (SP/SP-SM)	Sand w/silt (SP-SM)
Water Content (%)	12.0	15.1	9.7	16.5	12.3	16.9	3.5	4.3

Sample Information & Classification								
Boring #	Sub-Opt1-01	Sub-OPT1-1	Sub-Opt1-01	Sub-OPT1-1				
Sample #								
Depth (ft)	0.5-1	1.5-2	2.5-3	3.5-4				
Type	Bag	Bag	Bag	Bag				
Material Classification	Sandy Silty Clay (CL-ML/CL)	Sandy Lean Clay (CL)	Lean Clay (CL)	Lean Clay w/sand (CL)				
Water Content (%)	4.4	6.2	13.0	13.2				

9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Laboratory Test Summary

Project: Rex Solaire Solar Job: 15366

Client: Barr Engineering Company Date: 10/2/2024

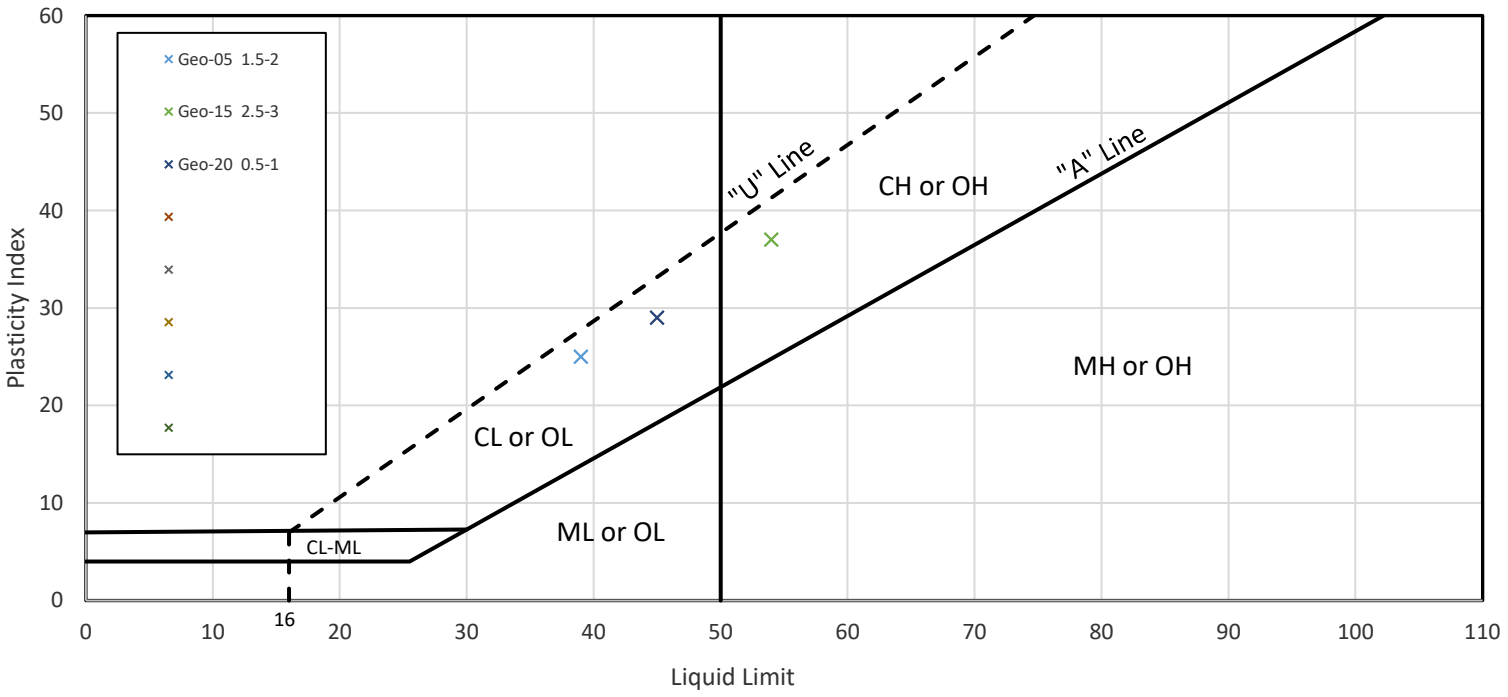
Sample Information & Classification

Boring #	Geo-05	Geo-15	Geo-20					
Sample #								
Depth (ft)	1.5-2	2.5-3	0.5-1					
Sample Type	Bag	Bag	Bag					
Material Classification	Sandy Lean Clay (CL)	Fat Clay w/sand (CH)	Lean Clay w/sand (CL)					

Atterberg Limits (ASTM:D4318)

Liquid Limit	39	54	45					
Plastic Limit	14	17	16					
Plasticity Index	25	37	29					

Plasticity Chart (ASTM:D2487)



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Laboratory Test Summary

Project: Rex Solaire Solar Job: 15366

Client: Barr Engineering Company Date: 10/9/2024

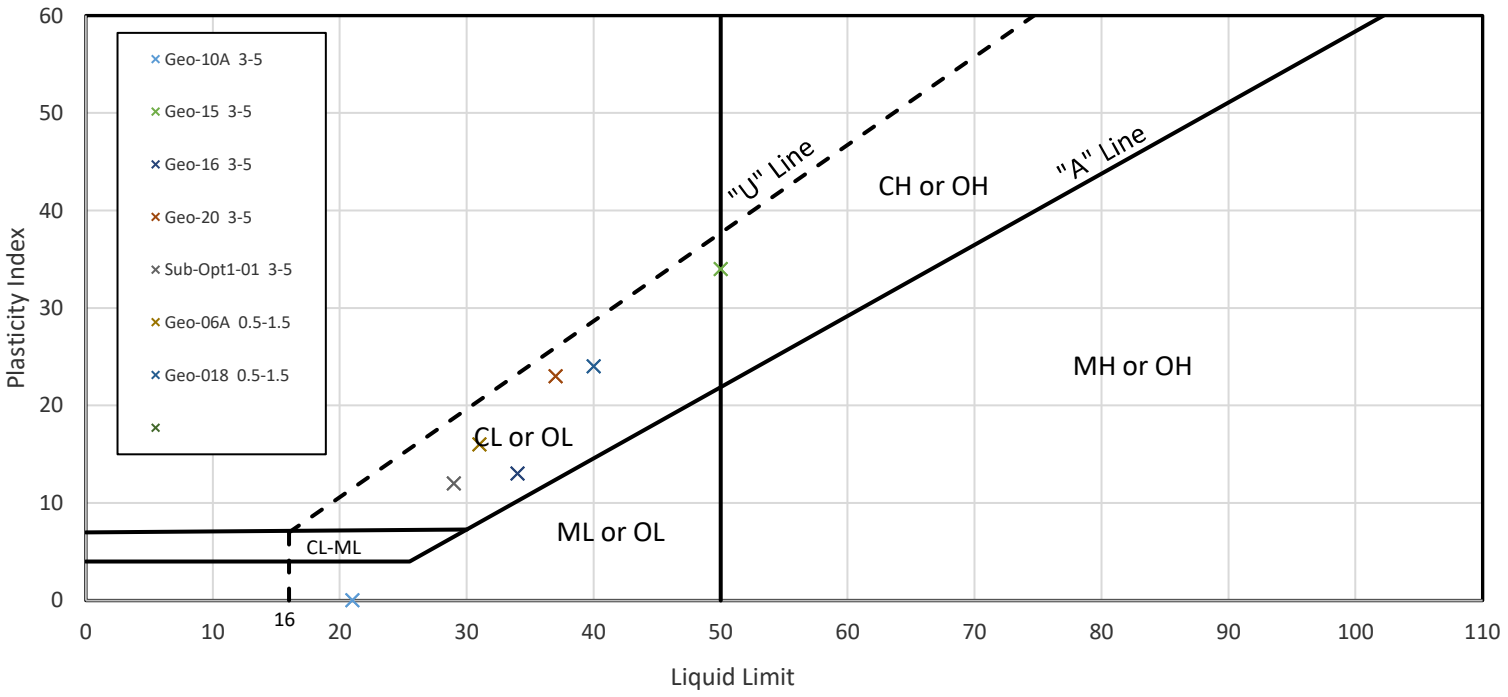
Sample Information & Classification

Boring #	Geo-10A	Geo-15	Geo-16	Geo-20	Sub-Opt1-01	Geo-06A	Geo-018	
Sample #								
Depth (ft)	3-5	3-5	3-5	3-5	3-5	0.5-1.5	0.5-1.5	
Sample Type	Bulk	Bulk	Bulk	Bulk	Bulk	Bulk	Bulk	
Material Classification	Silty Sand (SM)	Fat Clay w/sand (CH)	Sandy Lean Clay (CL)	Clayey Sand (SC)	Lean Clay w/sand (CL)	Sandy Lean Clay (CL)	Lean Clay w/sand (CL)	

Atterberg Limits (ASTM:D4318)

Liquid Limit	21	50	34	37	29	31	40	
Plastic Limit	N/A	16	21	14	17	15	16	
Plasticity Index	NP	34	13	23	12	16	24	

Plasticity Chart (ASTM:D2487)



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

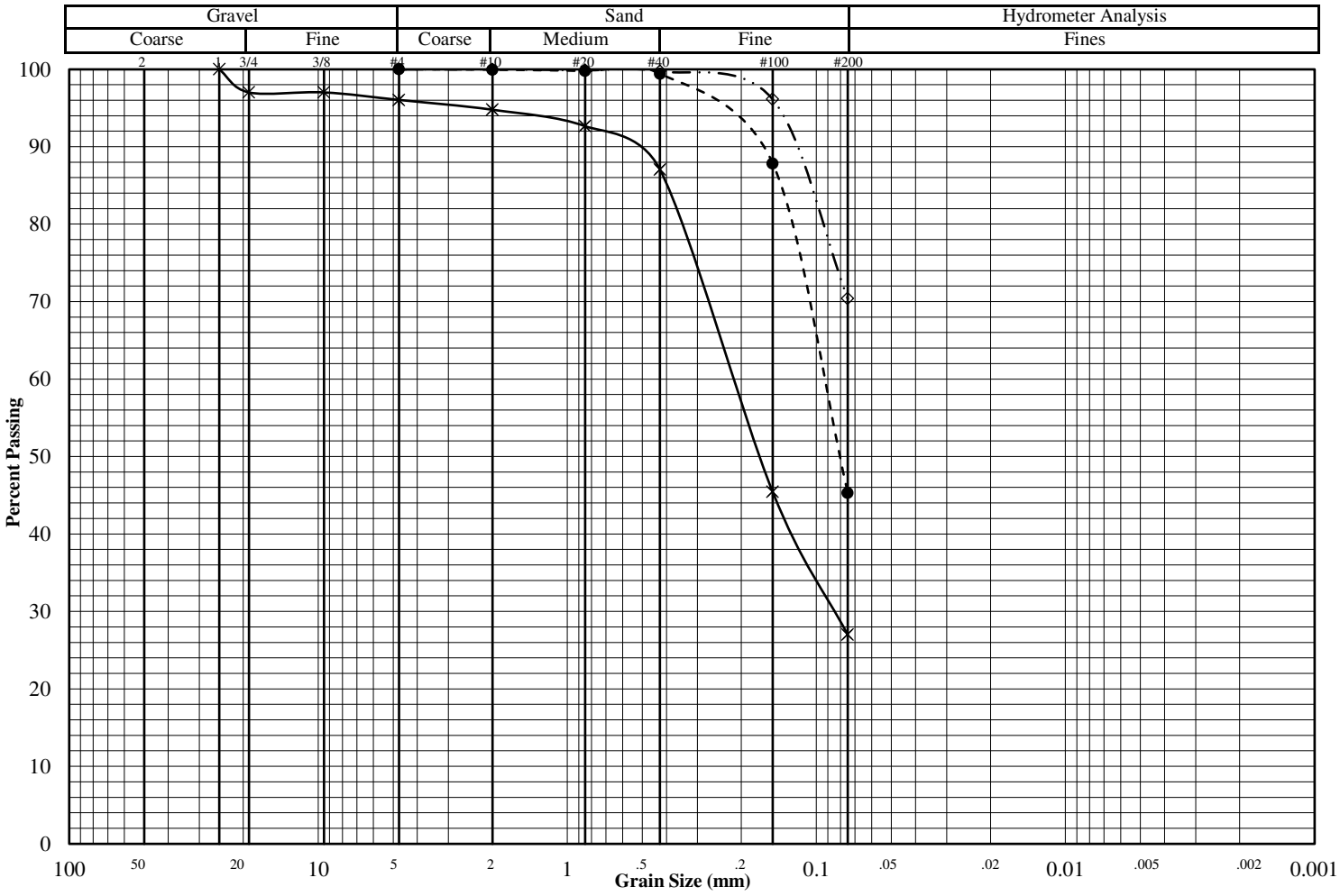
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/21/24
Reported To:	Barr Engineering Company	Report Date:	9/30/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Geo-10A		2.5-3	Bag	Silty Sand w/a trace of gravel (SM/SC-SM)
●	Geo-13		2.5-3	Bag	Silty Sand (SM)
◇	Geo-16		3.5-4	Bag	Silt w/sand and occasional pieces of clay (ML)



Additional Results	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index ASTM:D4318			
Water Content ASTM:D2216	15.3	11.9	16.5
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854			
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			

Percent Passing

	*	●	◇
Mass (g)	662.3	618.2	469.6
2"			
1.5"			
1"	100.0		
3/4"	97.0		
3/8"	97.0		
#4	96.0	100.0	100.0
#10	94.8	100.0	100.0
#20	92.7	99.8	99.9
#40	87.1	99.4	99.8
#100	45.5	87.8	96.1
#200	27.0	45.3	70.4

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

(* = assumed)

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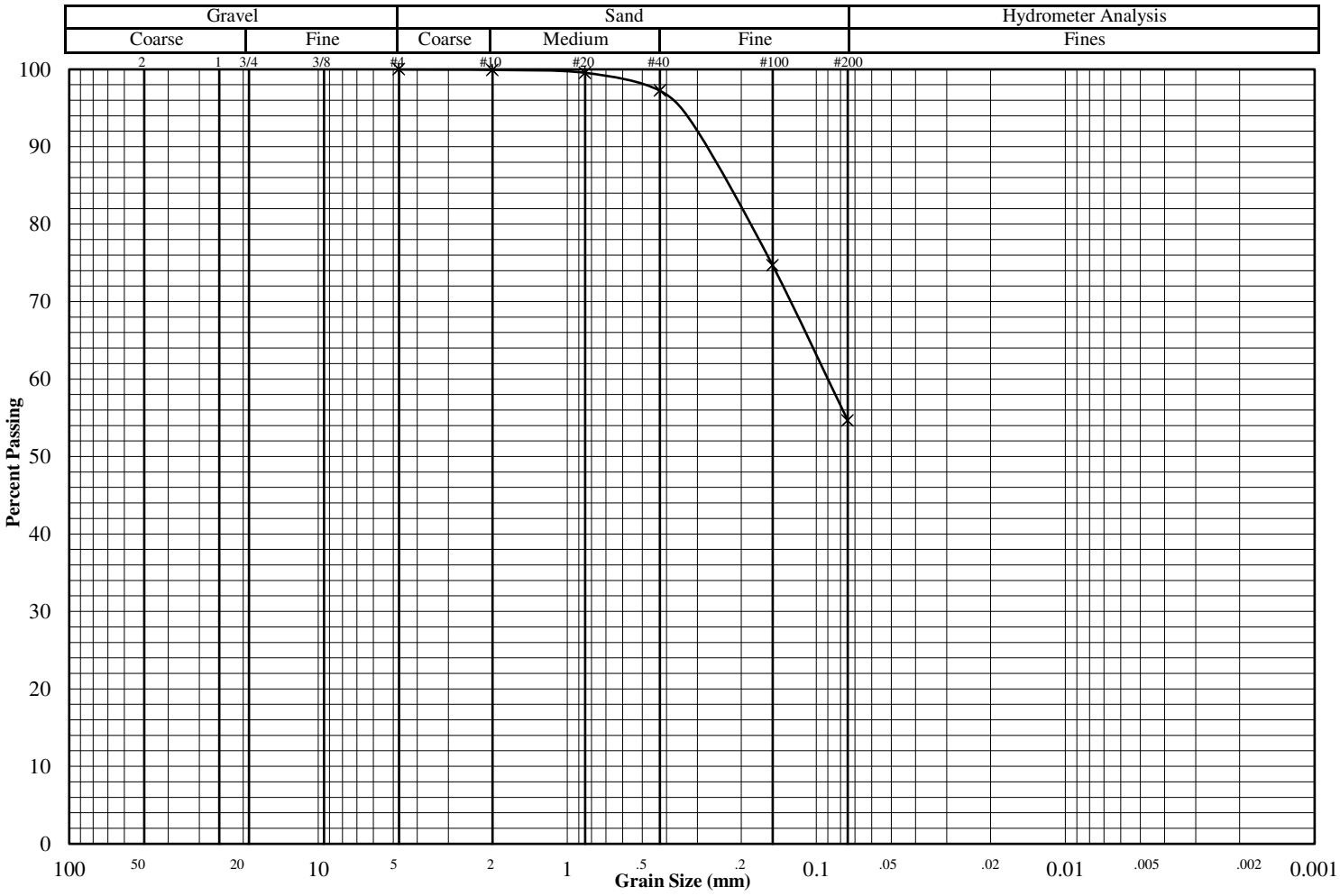
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/21/24
Reported To:	Barr Engineering Company	Report Date:	9/30/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Sub-OPT1-1		1.5-2	Bag	Sandy Lean Clay (CL)
●					
◇					



Additional Results	*	●	◇
Liquid Limit			
Plastic Limit			
Plasticity Index ASTM:D4318			
Water Content ASTM:D2216	6.2		
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854			
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			

Percent Passing

	*	●	◇
Mass (g)	613.3		
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		
#10	99.9		
#20	99.6		
#40	97.2		
#100	74.7		
#200	54.7		

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

(* = assumed)

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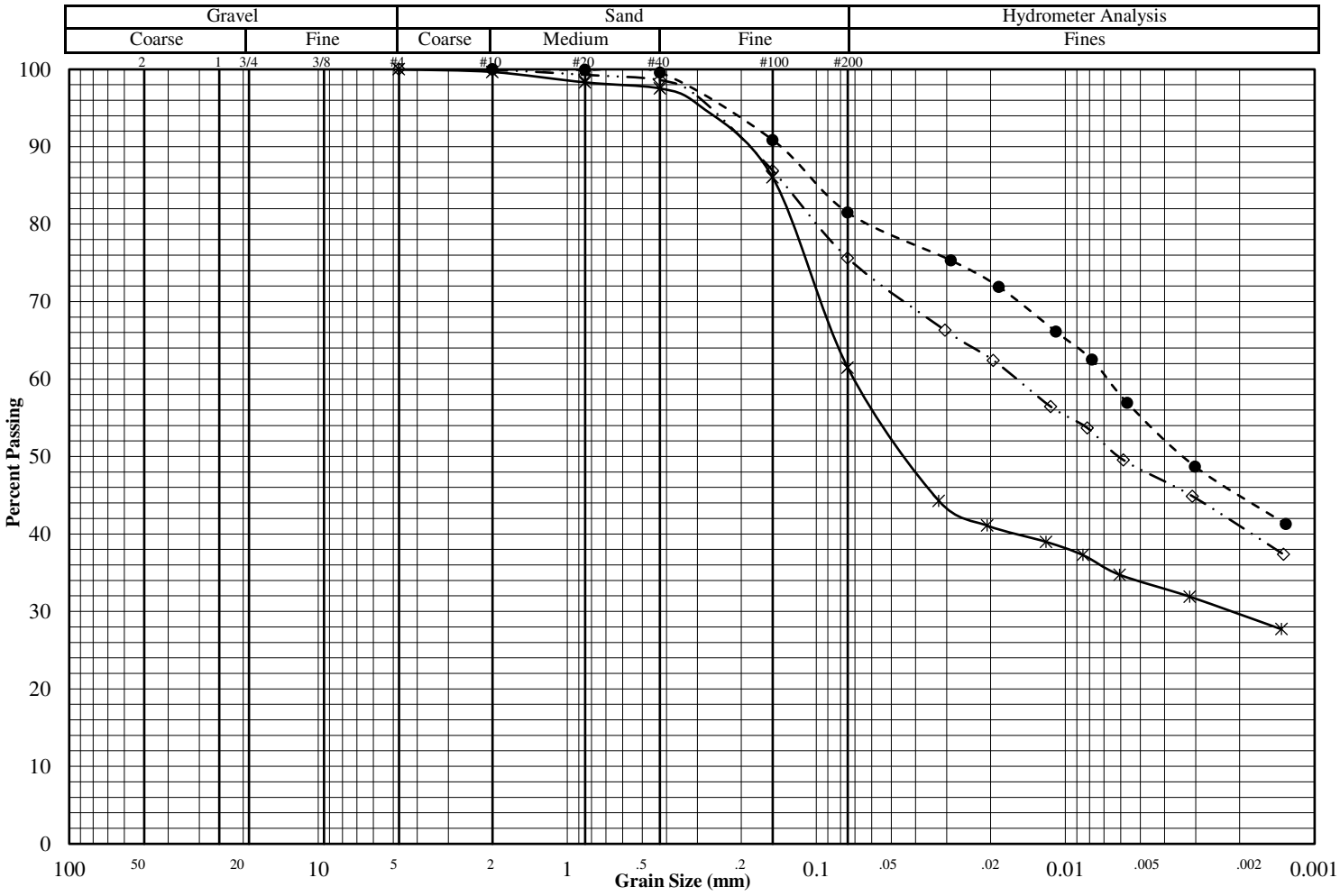
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/25/24
Reported To:	Barr Engineering Company	Report Date:	10/4/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Geo-05		1.5-2	Bag	Sandy Lean Clay (CL)
●	Geo-15		2.5-3	Bag	Fat Clay w/sand (CH)
◇	Geo-20		0.5-1	Bag	Lean Clay w/sand (CL)



Additional Results	*	●	◇
Liquid Limit	39	54	45
Plastic Limit	14	17	16
Plasticity Index ASTM:D4318	25	37	29
Water Content ASTM:D2216	15.8	21.5	12.3
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854	2.68*	2.68*	2.68*
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			

Percent Passing

	*	●	◇
Mass (g)	239.9	259.4	273.4
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0		100.0
#10	99.7	100.0	99.9
#20	98.3	99.9	99.3
#40	97.5	99.6	98.6
#100	86.0	90.9	86.9
#200	61.5	81.5	75.6

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

(* = assumed)

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Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/25/24
Reported To:	Barr Engineering Company	Report Date:	10/4/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
Spec 1	Geo-05		1.5-2	Bag	Sandy Lean Clay (CL)
Spec 2	Geo-15		2.5-3	Bag	Fat Clay w/sand (CH)
Spec 3	Geo-20		0.5-1	Bag	Lean Clay w/sand (CL)

Sieve Data

Specimen 1		Specimen 2		Specimen 3	
Sieve	% Passing	Sieve	% Passing	Sieve	% Passing
2"		2"		2"	
1.5"		1.5"		1.5"	
1"		1"		1"	
3/4"		3/4"		3/4"	
3/8"		3/8"		3/8"	
#4	100.0	#4		#4	100.0
#10	99.7	#10	100.0	#10	99.9
#20	98.3	#20	99.9	#20	99.3
#40	97.5	#40	99.6	#40	98.6
#100	86.0	#100	90.9	#100	86.9
#200	61.5	#200	81.5	#200	75.6

Hydrometer Data

Specimen 1		Specimen 2		Specimen 3	
Diameter (mm)	% Passing	Diameter	% Passing	Diameter	% Passing
0.032	44.3	0.029	75.3	0.030	66.3
0.021	41.1	0.019	71.9	0.019	62.4
0.012	39.0	0.011	66.1	0.011	56.4
0.009	37.3	0.008	62.5	0.008	53.7
0.006	34.7	0.006	56.9	0.006	49.6
0.003	31.9	0.003	48.7	0.003	44.9
0.001	27.7	0.001	41.3	0.001	37.4

Remarks

Specimen 1	Specimen 2	Specimen 3

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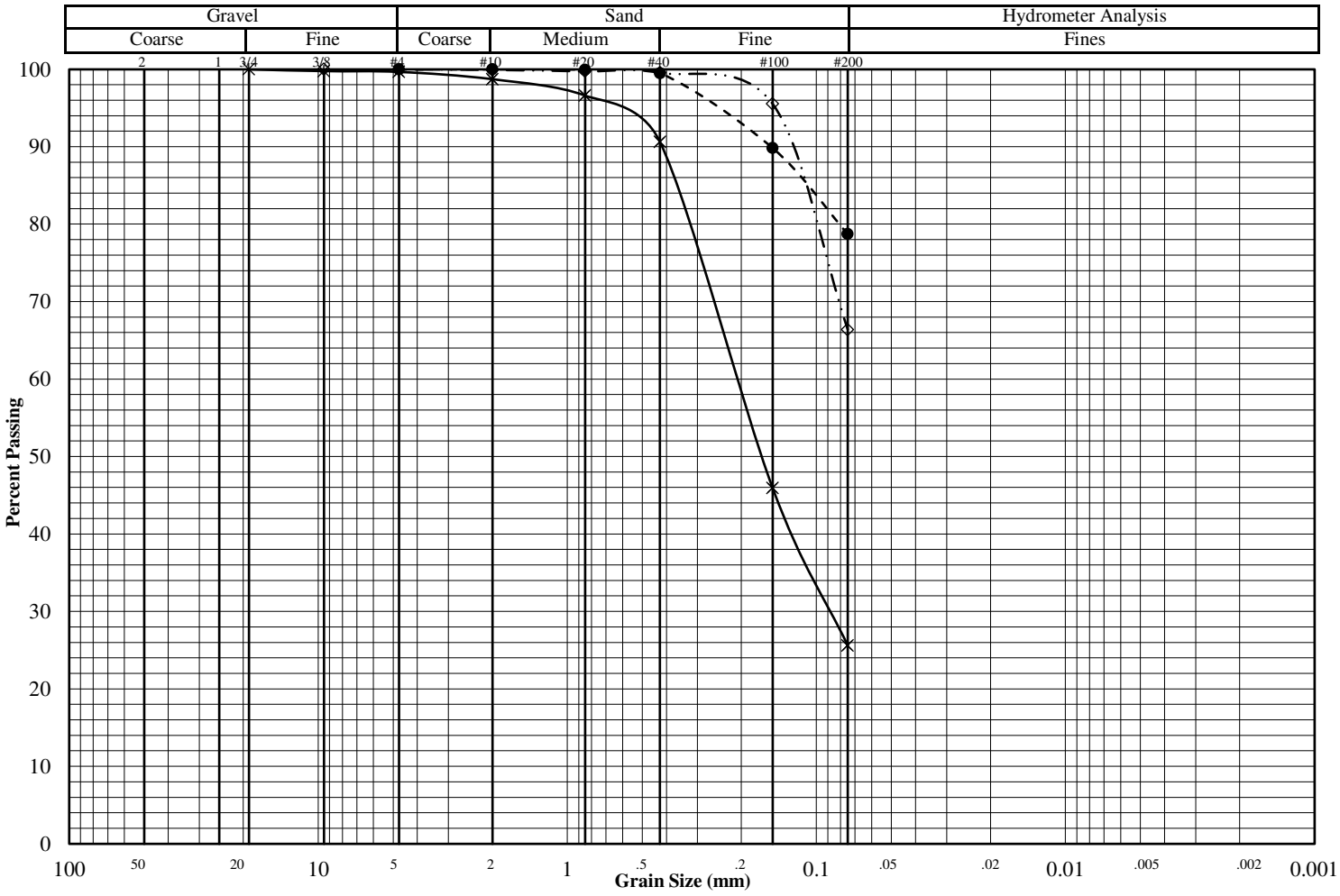
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/20/24
Reported To:	Barr Engineering Company	Report Date:	9/26/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Geo-10A		3-5	Bulk	Silty Sand (SM)
●	Geo-15		3-5	Bulk	Fat Clay w/sand (CH)
◇	Geo-16		3-5	Bulk	Sandy Lean Clay (CL)



Additional Results	*	●	◇
Liquid Limit	21	50	34
Plastic Limit	N/A	16	21
Plasticity Index ASTM:D4318	NP	34	13
Water Content ASTM:D2216			
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854			
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			

(* = assumed)

	Percent Passing		
	*	●	◇
Mass (g)	24903.0	18185.0	21265.0
2"			
1.5"			
1"			
3/4"	100.0		
3/8"	99.8		100.0
#4	99.7	100.0	100.0
#10	98.7	100.0	99.9
#20	96.6	99.9	99.7
#40	90.6	99.5	99.6
#100	45.9	89.8	95.5
#200	25.6	78.7	66.4

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

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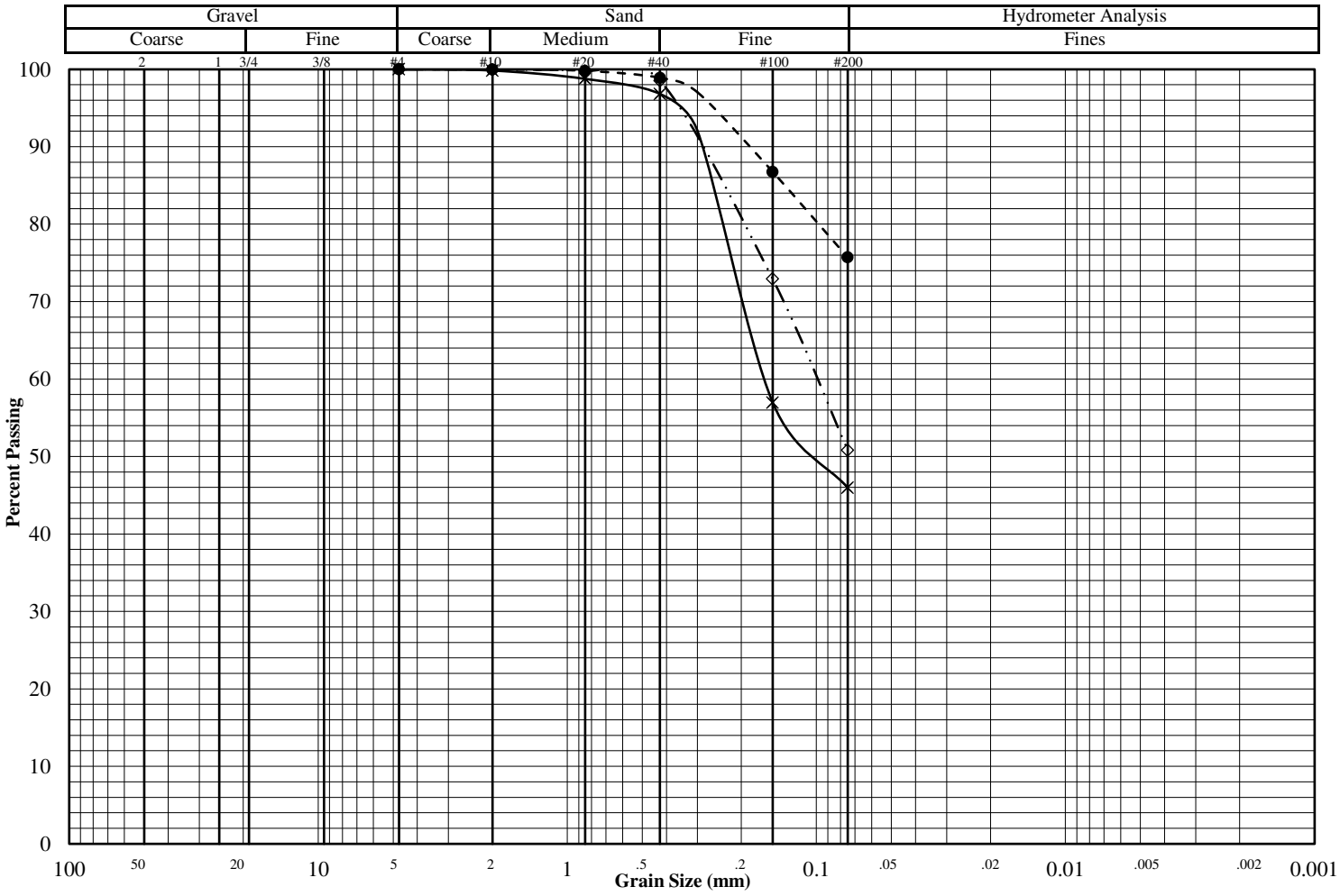
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/20/24
Reported To:	Barr Engineering Company	Report Date:	9/26/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Geo-20		3-5	Bulk	Clayey Sand (SC)
●	Sub-Opt1-01		3-5	Bulk	Lean Clay w/sand (CL)
◇	Geo-06A		0.5-1.5	Bulk	Sandy Lean Clay (CL)



Additional Results	*	●	◇
Liquid Limit	37	29	31
Plastic Limit	14	17	15
Plasticity Index ASTM:D4318	23	12	16
Water Content ASTM:D2216			
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854			
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			7.5

Percent Passing

	*	●	◇
Mass (g)	20987.0	21409.0	21847.0
2"			
1.5"			
1"			
3/4"			
3/8"			
#4	100.0	100.0	100.0
#10	99.8	100.0	100.0
#20	98.8	99.8	99.8
#40	96.8	98.9	98.5
#100	57.0	86.8	72.9
#200	46.0	75.7	50.8

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

(* = assumed)

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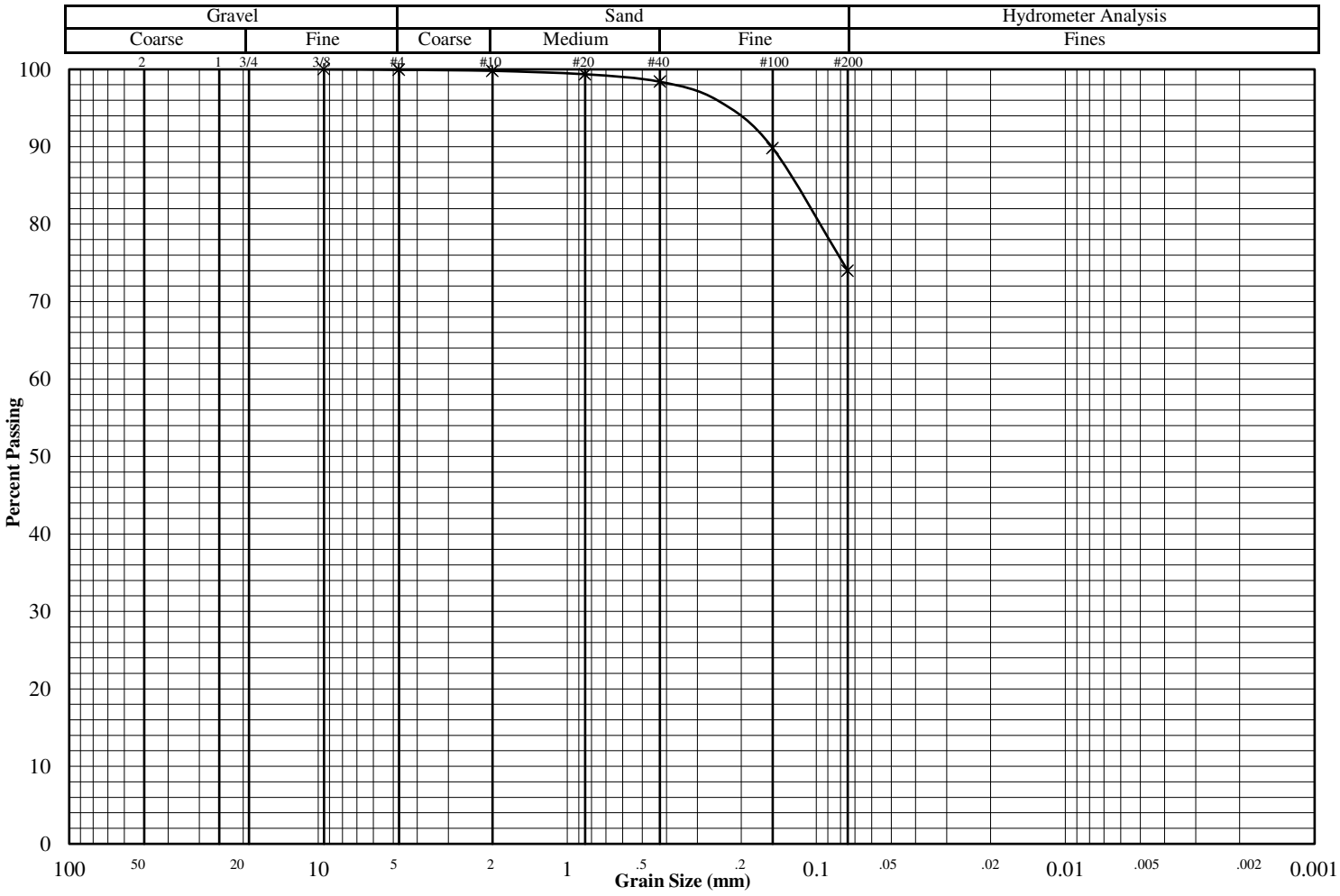
Bloomington, MN 55431

Grain Size Distribution ASTM D422-16

Job No. : **15366**

Project:	Rex Solaire Solar	Test Date:	9/20/24
Reported To:	Barr Engineering Company	Report Date:	9/26/24

	Location / Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Classification
*	Geo-018		0.5-1.5	Bulk	Lean Clay w/sand (CL)
●					
◇					



Additional Results	*	●	◇
Liquid Limit	40		
Plastic Limit	16		
Plasticity Index ASTM:D4318	24		
Water Content ASTM:D2216			
Dry Density (pcf) ASTM:D7263			
Specific Gravity ASTM:D854			
Porosity			
Organic Content ASTM:D2974			
pH ASTM:D4972 Method B			

Percent Passing

	*	●	◇
Mass (g)	21865.0		
2"			
1.5"			
1"			
3/4"			
3/8"	100.0		
#4	99.9		
#10	99.8		
#20	99.4		
#40	98.4		
#100	89.8		
#200	74.0		

	*	●	◇
D_{60}			
D_{30}			
D_{10}			
C_U			
C_C			

Remarks:

(* = assumed)

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California Bearing Ratio ASTM:D1883

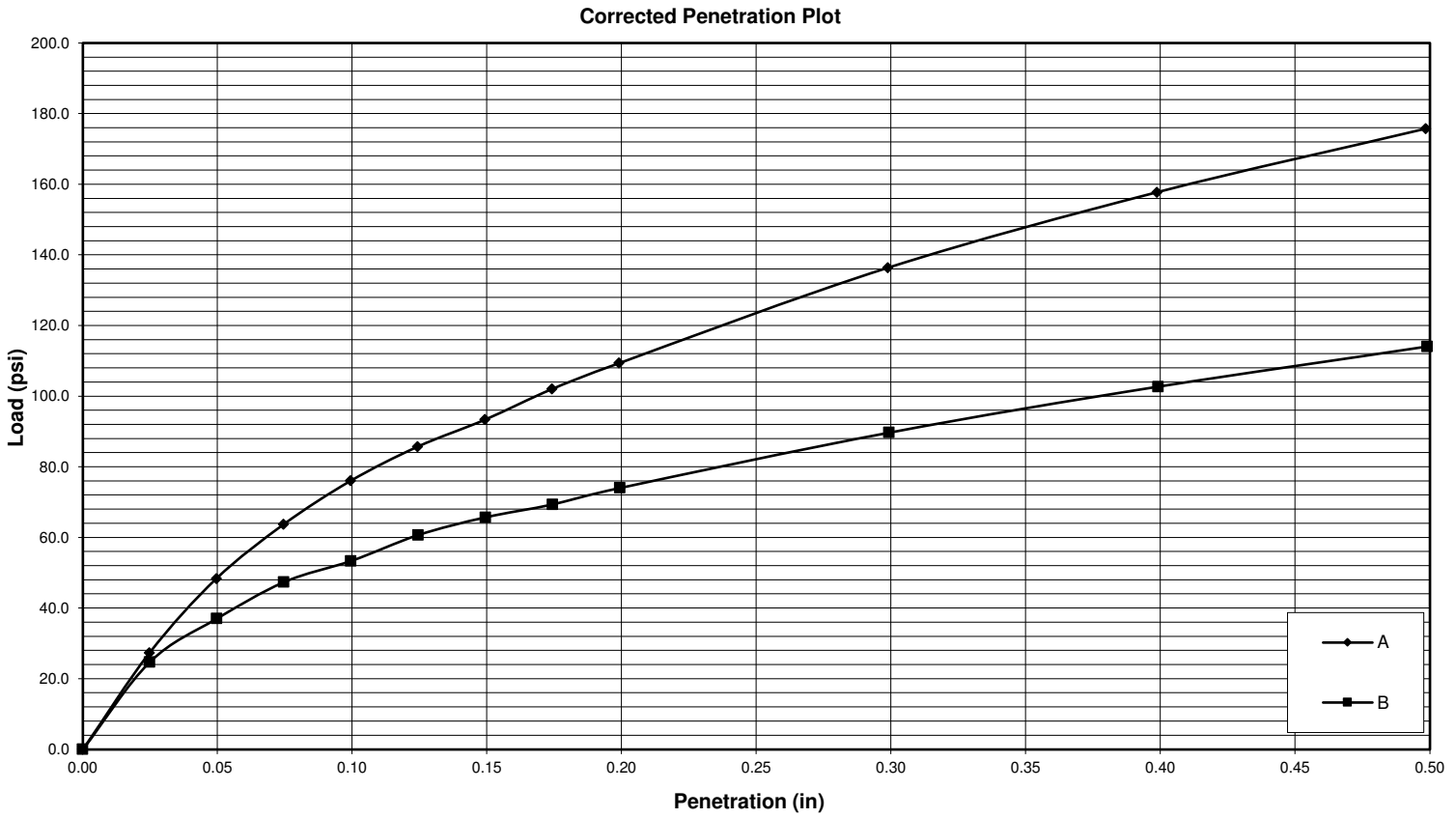
Project:	Rex Solaire Solar	Job:	**15366**
Client:	Barr Engineering Company	Date:	10/7/24

Boring #:	Geo-06A	Geo-018		Procedural Method:
Sample:				Specimens compacted to approximately 95% of maximum standard proctor density at optimum moisture content. Specimens soaked for a period of 4 days before CBR test was performed.
Depth (ft):	0.5-1.5	0.5-1.5		
Type:	Bulk	Bulk		
Classification:	Sandy Lean Clay (CL)		Lean Clay w/sand (CL)	

Laboratory Moisture-Density Values				Index Properties	
Method:	ASTM:D698 Method B			LL:	Gs:
Maximum Dry Density (PCF):	112.6	108.2		PL:	Organic Content:
Optimum Water Content:	16.1%	17.3%		PI:	pH:

Initial Molding Conditions			
Specimen	A	B	
Compaction Hammer:	5 lb	5 lb	
Number of Layers:	3	3	
Blows per Layer:	NA	NA	
Initial Moisture Content:	16.1%	17.3%	
Initial Dry Density (PCF)	107.0	102.9	
Relative Compaction	95.0%	95.1%	
Soaking Phase			
Days Soaked	4	4	
Surcharge (psf)	50	50	
Total Swell (%)	0.5%	1.4%	
Penetration Phase			
Surcharge (psf)	50	50	
Corrected CBR Values			
at 0.1 inch (%)	**7.6%**	**5.4%**	
at 0.2 inch (%)	7.3%	5.0%	
Moisture Content After Penetration			
Top 1" of Specimen:	18.1%	22.2%	
Average of specimen:	17.7%	20.5%	

Stress vs. Penetration Graph



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

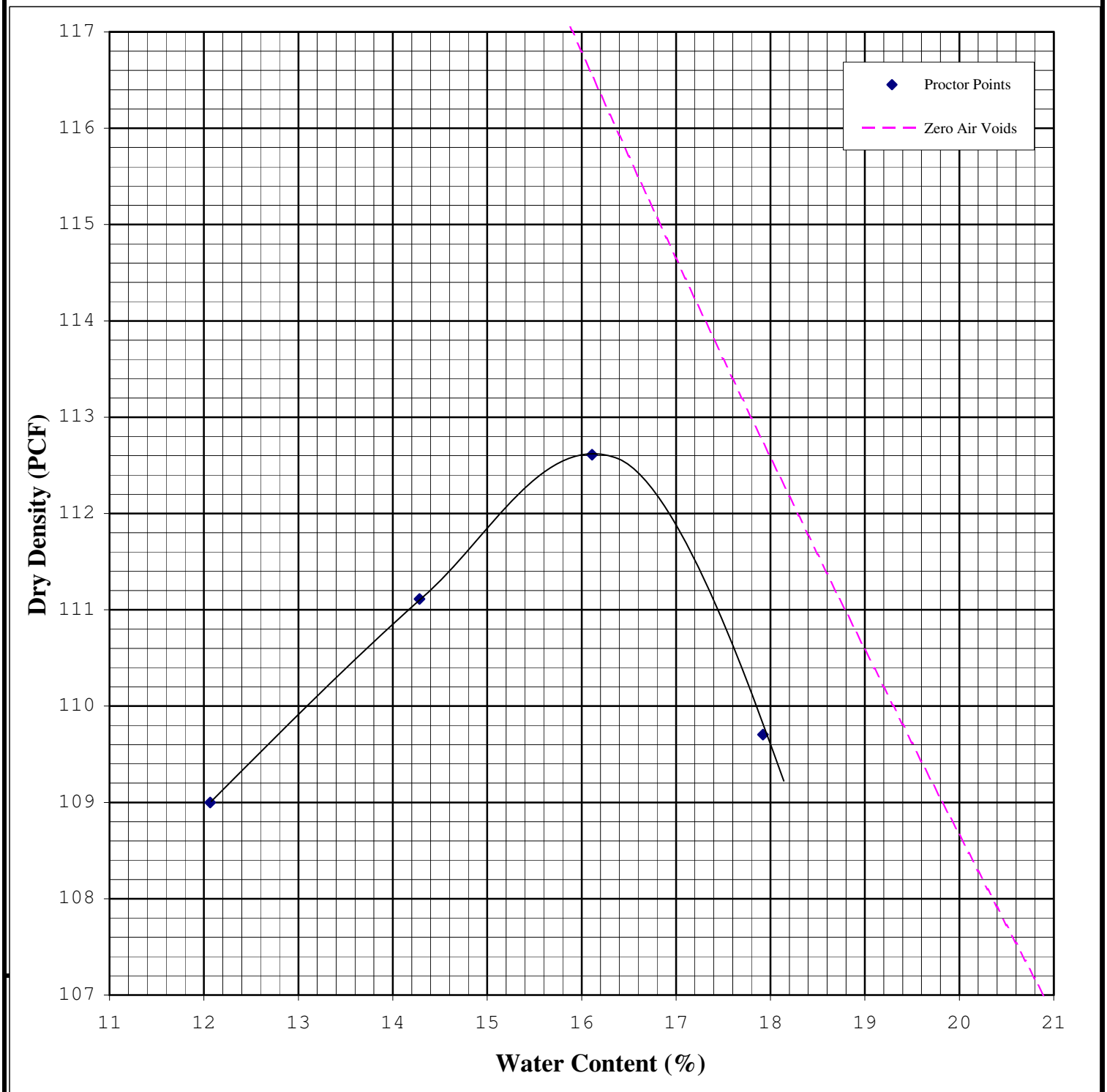
Client: **Barr Engineering Company** Job No. **15366**

Boring No.: **Geo-06A** Sample: Depth(ft): **0.5-1.5** Location:

Soil Type: **Sandy Lean Clay (CL)**

As Received W.C. (%): **7.3** LL: **31** PL: **15** PI: **16** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **112.6** Opt. Water Content (%): **16.1**



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

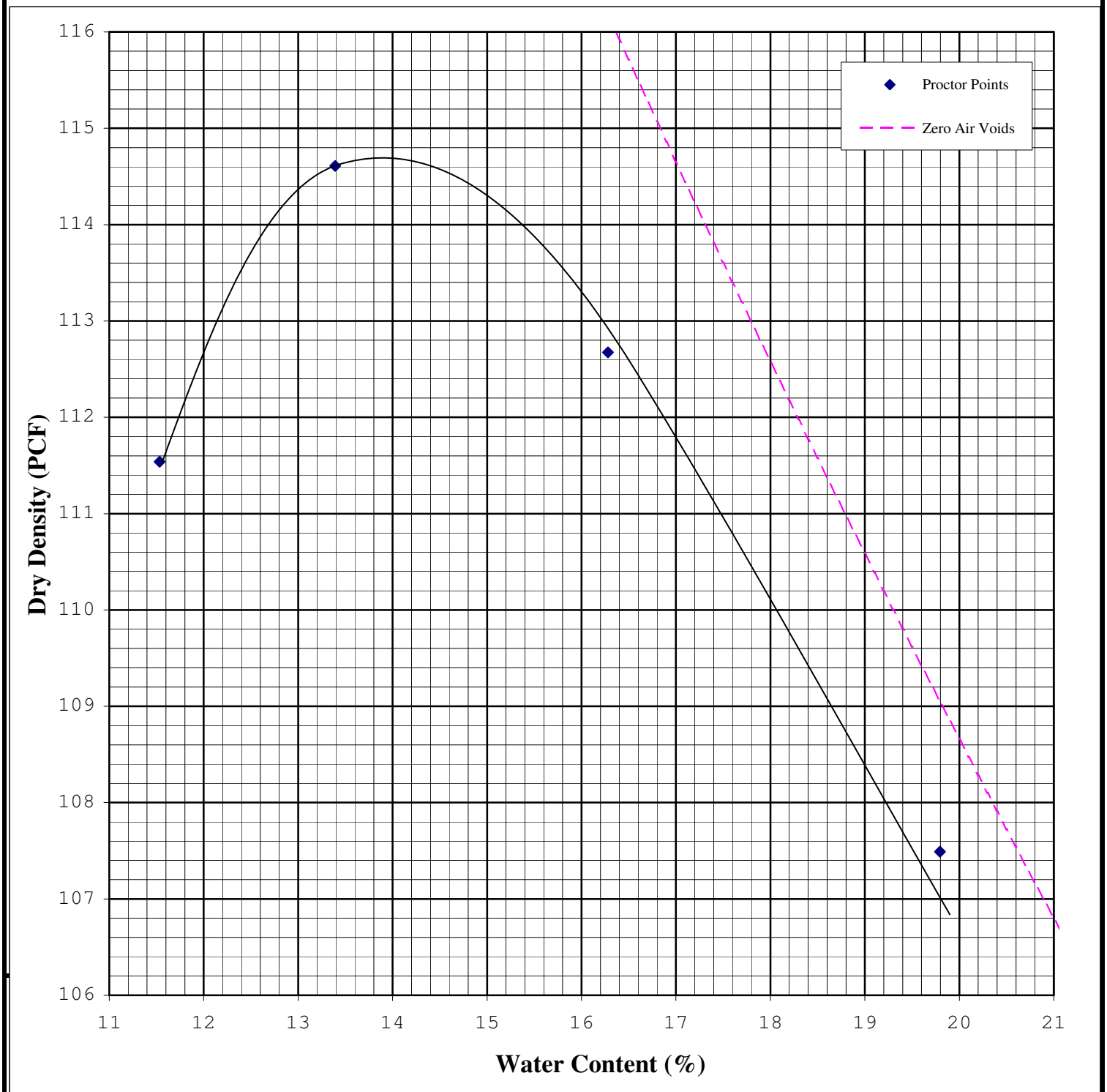
Client: **Barr Engineering Company** Job No. **15366**

Boring No.: **Geo-10A** Sample: Depth(ft): **3-5** Location:

Soil Type: **Silty Sand (SM)**

As Received W.C. (%): **13.6** LL: **21** PL: **N/A** PI: **NP** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **114.7** Opt. Water Content (%): **13.9**



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

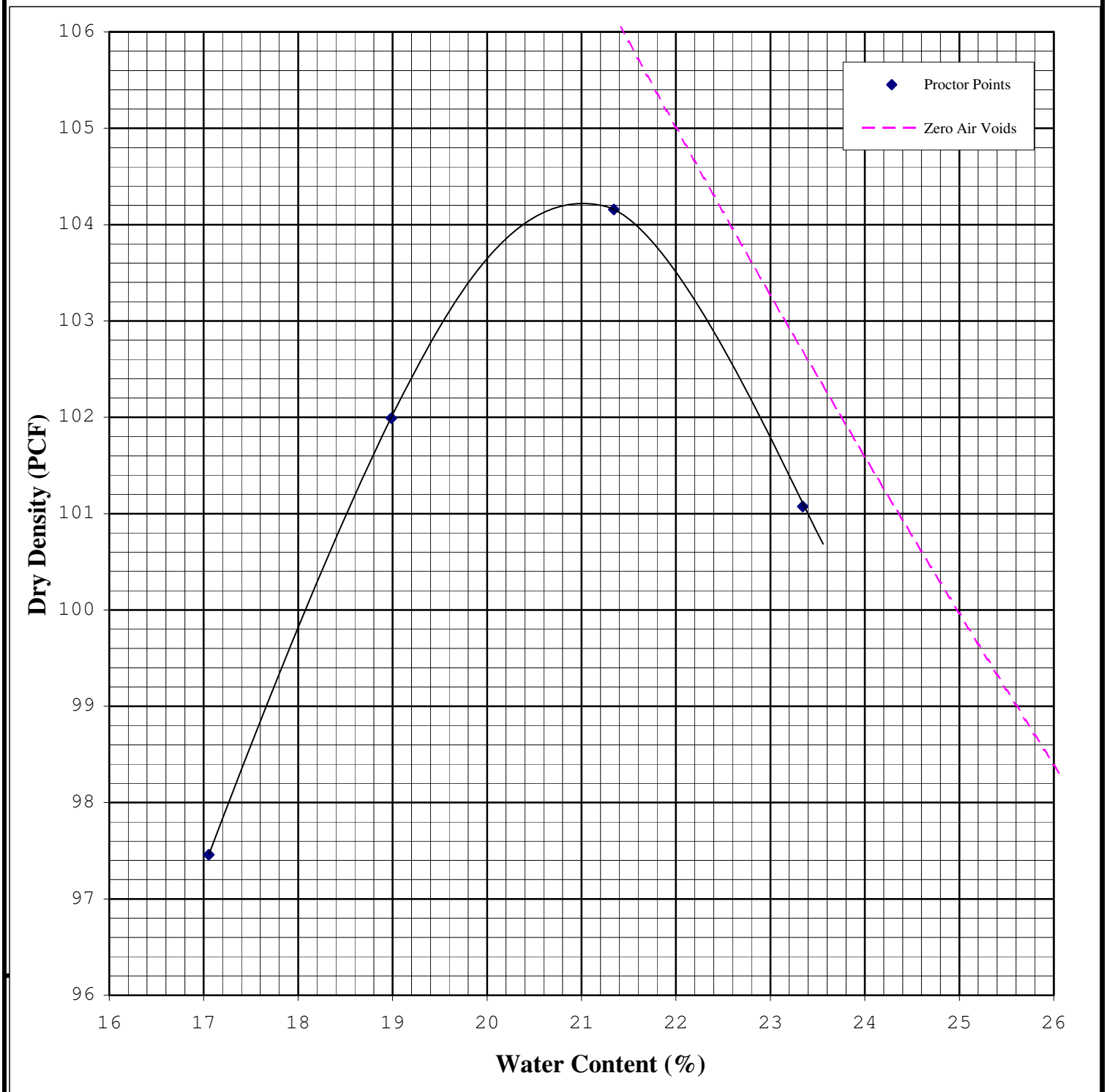
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-15** Sample: Depth(ft): **3-5** Location:

Soil Type: **Fat Clay w/sand (CH)**

As Received W.C. (%): **20.7** LL: **50** PL: **16** PI: **34** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **104.2** Opt. Water Content (%): **21.0**



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

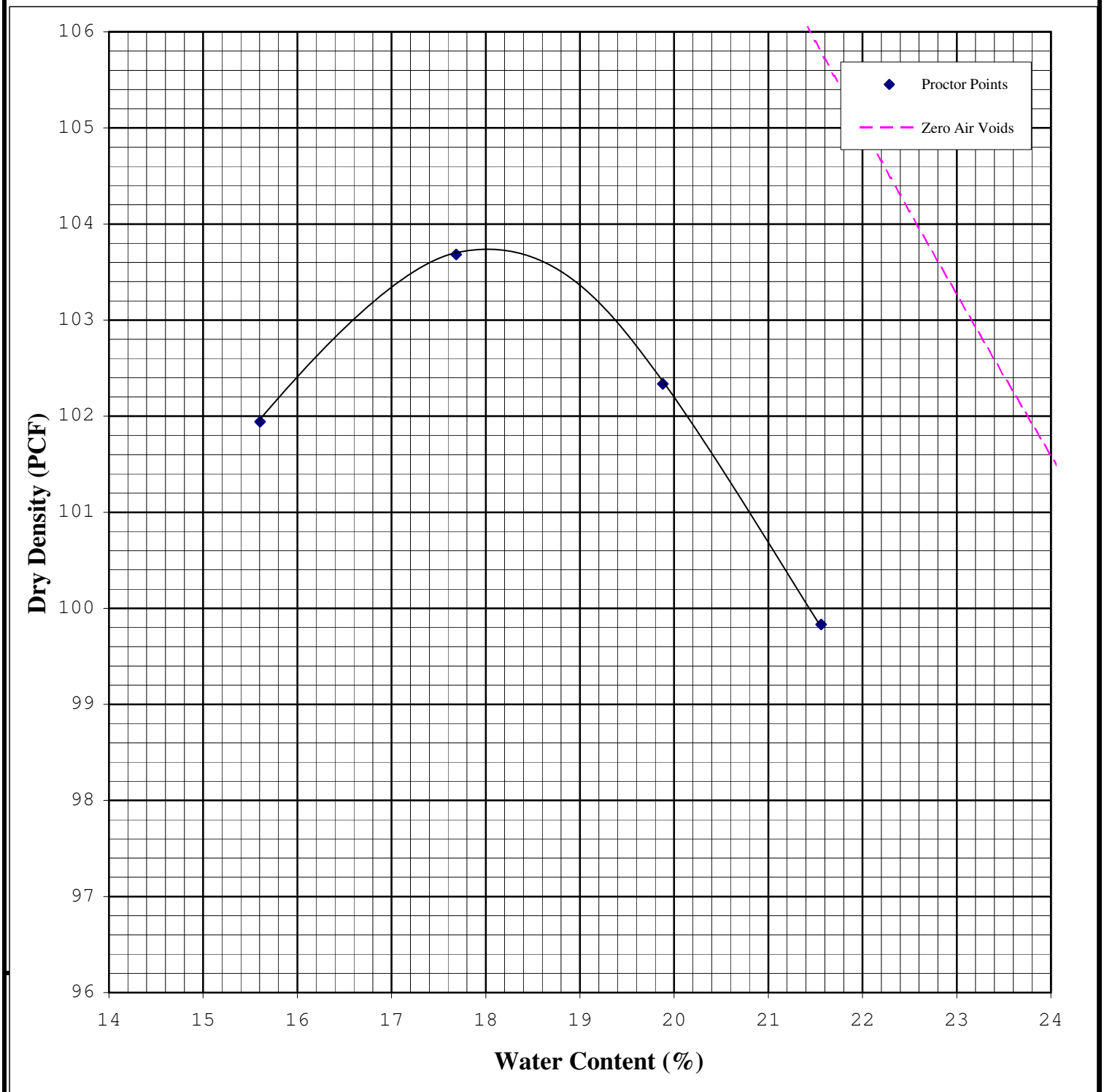
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-16** Sample: Depth(ft): **3-5** Location:

Soil Type: **Sandy Lean Clay (CL)**

As Received W.C. (%): **13.9** LL: **34** PL: **21** PI: **13** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **103.7** Opt. Water Content (%): **18.0**



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SET-R18a

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

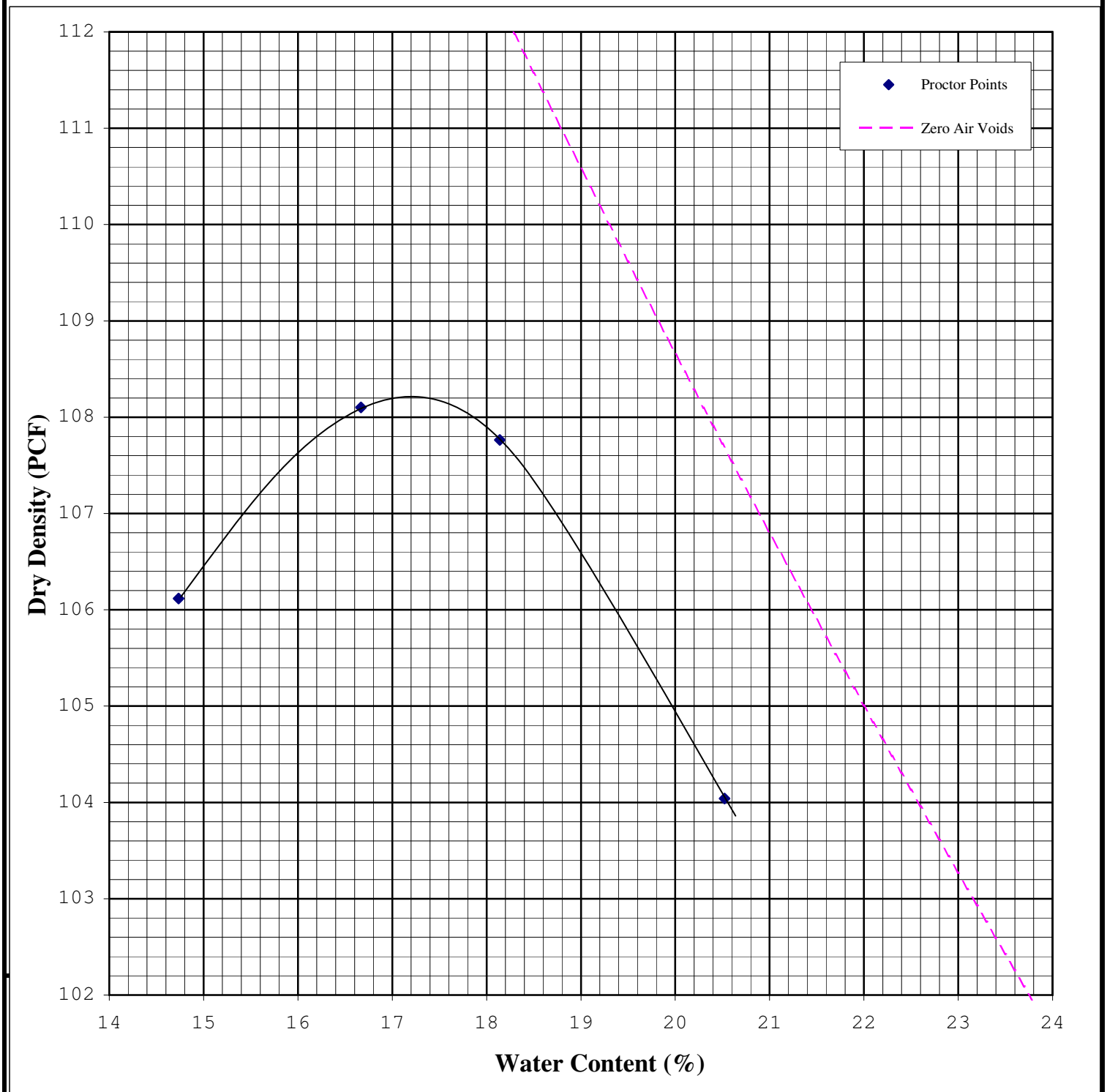
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-018** Sample: Depth(ft): **0.5-1.5** Location:

Soil Type: **Lean Clay w/sand (CL)**

As Received W.C. (%): **5.0** LL: **40** PL: **16** PI: **24** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **108.2** Opt. Water Content (%): **17.2**



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

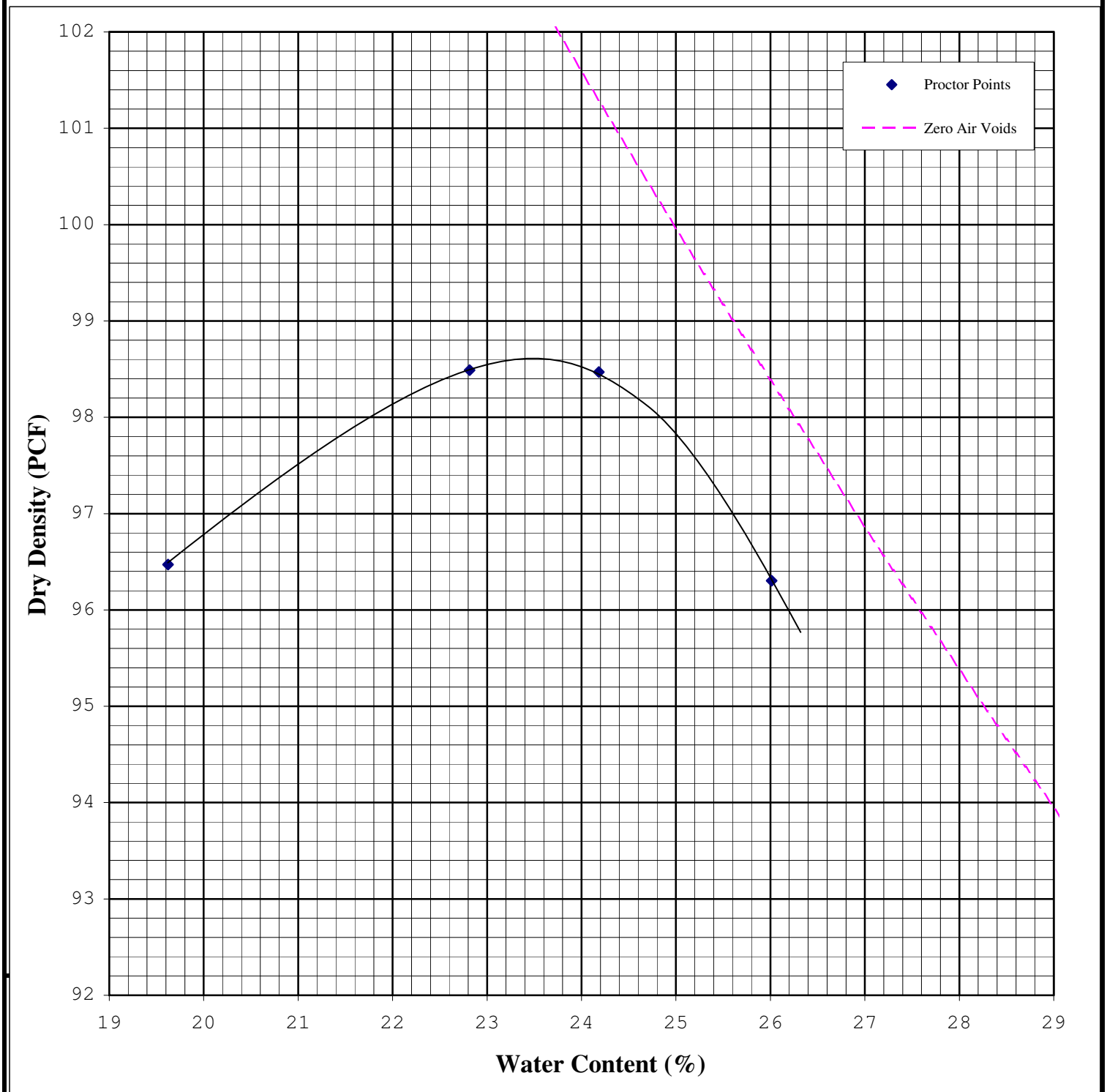
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-20** Sample: Depth(ft): **3-5** Location:

Soil Type: **Clayey Sand (SC)**

As Received W.C. (%): **22.0** LL: **37** PL: **14** PI: **23** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **98.6** Opt. Water Content (%): **23.5**



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Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

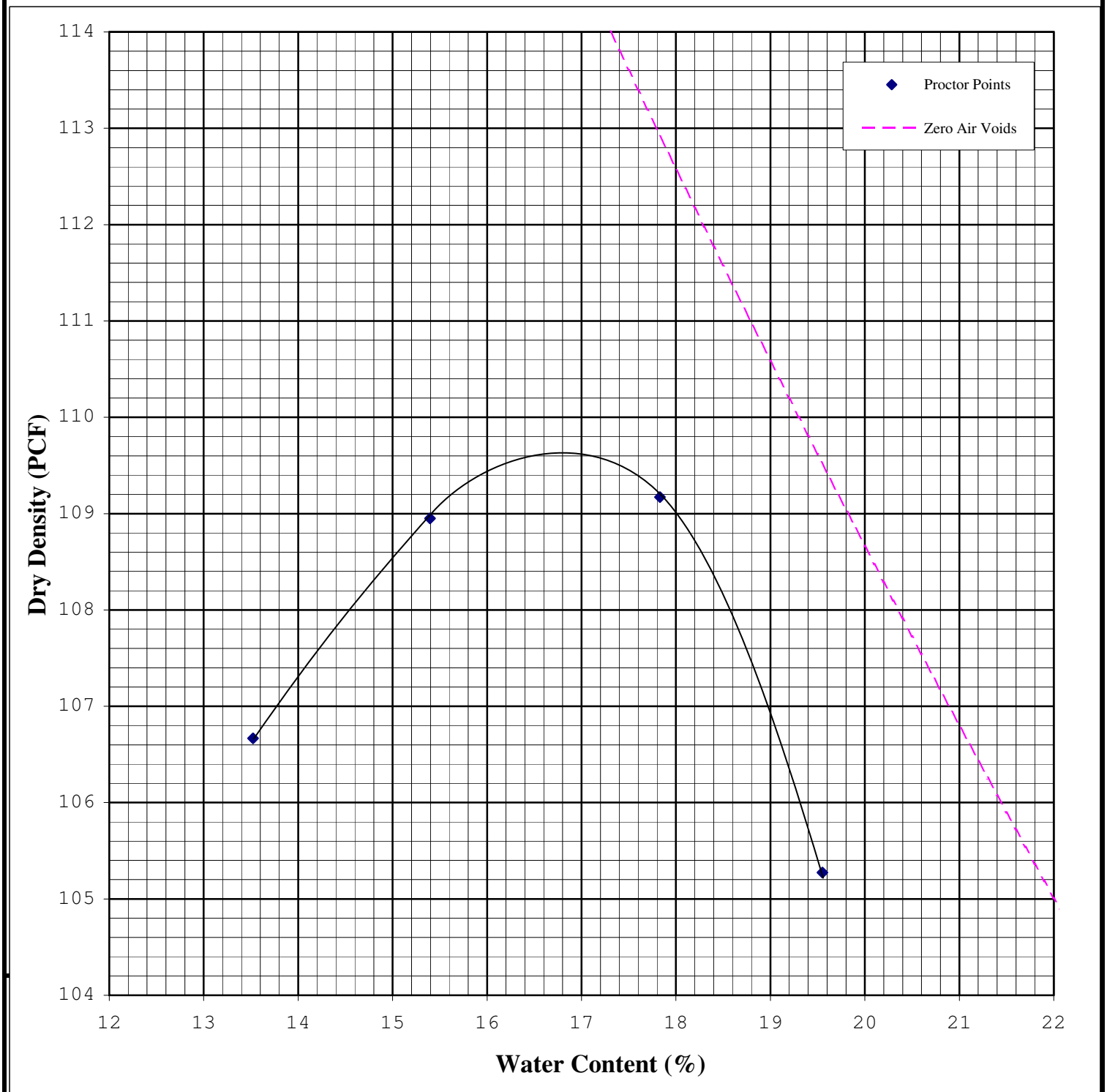
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Sub-Opt1-01** Sample: Depth(ft): **3-5** Location:

Soil Type: **Lean Clay w/sand (CL)**

As Received W.C. (%): **10.2** LL: **29** PL: **17** PI: **12** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **109.6** Opt. Water Content (%): **16.8**



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Appendix H

Analytical Laboratory Test Results

pH Testing Summary Sheet (ASTM:D4972)

Project: **Rex Solaire Solar** Job: **15366**

Client: **Barr Engineering Company** Date: **10/9/2024**

Boring / Location	Sample	Sample Type	Depth (ft)	pH	Visual Classification
Geo-05	**corrosion**	**Bag**	**1-4**	**8.9**	**Lean Clay w/sand (CL/CH)**
Geo-10A	**corrosion**	**Bag**	**1-4**	**8.0**	**Silty Sand (SM/SC-SM)**
Geo-13	**corrosion**	**Bag**	**1-4**	**9.0**	**Silty Sand (SM/SC-SM)**
Geo-15	**corrosion**	**Bag**	**1-4**	**8.1**	**Lean Clay w/sand (CL/CH)**
Geo-16	**corrosion**	**Bag**	**1-4**	**8.9**	**Silty Sand (SM)**
Geo-20	**corrosion**	**Bag**	**1-4**	**8.7**	**Silty Sand (SM)**
Sub-OPT1-1	**corrosion**	**Bag**	**3-5**	**8.1**	**Lean Clay w/sand (CL)**

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SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

pH Testing Summary Sheet (ASTM:D4972)

Project: **Rex Solaire Solar** Job: **15366**

Client: **Barr Engineering Company** Date: **10/9/2024**

Boring / Location	Sample	Sample Type	Depth (ft)	pH	Visual Classification
Geo-06A		**Bulk**	**0.5-1.5**	**7.5**	**Sandy Lean Clay (CL)**

9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Soil Resistivity Results

(ASTM G57 - Laboratory Soil Box)

Project:	**Rex Solaire Solar**	**Job #:**	**15366**
Client:	**Barr Engineering Company**	**Date:**	**9/26/24**

Sample Information & Classification

Boring	Geo-05	Geo-10A	Geo-13	Geo-15	Geo-16	Geo-20
Sample	Corrosion	Corrosion	Corrosion	Corrosion	Corrosion	Corrosion
Depth (ft)	1-4	1-4	1-4	1-4	1-4	1-4
Sample Type	Bag	Bag	Bag	Bag	Bag	Bag
Soil Classification	Lean Clay w/sand (CL/CH)	Silty Sand (SM/SC-SM)	Silty Sand (SM/SC-SM)	Lean Clay w/sand (CL/CH)	Silty Sand (SM)	Silty Sand (SM)
Water Content & Density (As Received)						
Water Content (%)	21.6	17.1	9.7	22.9	8.2	3.6
Dry Density (PCF)	99.6	98.5	88.3	95.8	79.2	81.2
Water Content & Density (Saturated)						
Water Content (%)	**30.2**	**23.6**	**26.1**	**32.3**	**27.0**	**28.1**
Dry Density (PCF)	**89.9**	**99.7**	**93.9**	**86.9**	**92.3**	**85.3**
Resistivity (ohm-cm) (ASTM G57)						
As Received Moisture Content Resistivity (ohm-cm)	**114**	**177**	**2,010**	**131**	**10,100**	**98,000**
Saturated Condition Resistivity (ohm-cm)	**98**	**134**	**980**	**114**	**1,270**	**5,800**

9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Soil Resistivity Results

(ASTM G57 - Laboratory Soil Box)

Project:	**Rex Solaire Solar**	**Job #:**	**15366**
Client:	**Barr Engineering Company**	**Date:**	**9/26/24**

Sample Information & Classification

Boring	Sub-Opt1-01	Geo-06A				
Sample	Corrosion					
Depth (ft)	3-5	0.5-1.5				
Sample Type	Bag	Bulk				
Soil Classification	Lean Clay w/sand (CL)	Sandy Lean Clay (CL)				
Water Content & Density (As Received)						
Water Content (%)	11.8	7.3				
Dry Density (PCF)	86.4	90.0				
Water Content & Density (Saturated)						
Water Content (%)	**21.4**	**23.3**				
Dry Density (PCF)	**101.8**	**98.0**				
Resistivity (ohm-cm) (ASTM G57)						
As Received Moisture Content Resistivity (ohm-cm)	**1,300**	**9,350**				
Saturated Condition Resistivity (ohm-cm)	**330**	**835**				

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218-846-1465

Virgnia
110 ½ S 15th Avenue W
Virginia, MN 55792
218-440-2043

October 07, 2024
Laboratory Report

Soil Engineering Testing, Inc
Tyler Sandoz
9530 James Ave S
Bloomington, MN 55431

RE: Rex Solaire Solar
Work Order: B014940

Enclosed are the results of analyses for samples received by the laboratory on 09/23/2024 17:40. If you have any questions concerning this report, please feel free to reach out to customer service at 888-200-5770 or the contacts listed below:

Chad Hadler	Senior Project Manager	Chad.Hadler@rmbel.com	(952) 456-8470
Justin Tweedale	Senior Project Manager	Justin.Tweedale@rmbel.com	(218) 849-8747
Kathleen Mitchell	Quality Assurance Director	Kathleen.Mitchell@rmbel.info	(785) 493-1633
Robert Borash	President | CEO	Robert.Borash@rmbel.info	(218) 849-6420

Report approved by:



Jared Dickinson
Senior Project Manager
jared.dickinson@rmbel.com

The results in this report apply only to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

Detroit Lakes (DL) Certification / Accreditation Numbers: EPA Lab ID MN00918 • Minnesota Department of Health 027-005-336 • North Dakota Department of Environmental Quality R-187
Burnsville (BL) Certification / Accreditation Numbers: EPA Lab ID MN01091 • Minnesota Department of Health 027-053-475 • North Dakota Department of Environmental Quality R-231
Hibbing (HB) Certification / Accreditation Numbers: EPA Lab ID MN01082 • Minnesota Department of Health 027-137-480 • North Dakota Department of Environmental Quality R-228

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Report Date: October 07,2024

Soil Engineering Testing, Inc
9530 James Ave S
Bloomington MN, 55431

Project: Rex Solaire Solar
Project Number: 15366

Date/Time Received
9/23/2024 5:40:00PM

ANALYTICAL REPORT FOR SAMPLES

Laboratory ID	Sample Name	Matrix	Date/Time Sampled
B014940-01	Geo-05	Solid	09/23/2024 09:00
B014940-02	Geo-10	Solid	09/23/2024 09:00
B014940-03	Geo-13	Solid	09/23/2024 09:00
B014940-04	Geo-15	Solid	09/23/2024 09:00
B014940-05	Geo-16	Solid	09/23/2024 09:00
B014940-06	Geo-20	Solid	09/23/2024 09:00
B014940-07	Sub-Opt1-1	Solid	09/23/2024 09:00
B014940-08	Geo-06	Solid	09/23/2024 09:00

Additional information:

All samples will be retained for 30 days from date sampled, unless otherwise requested.
Record retention policy is 5 years unless otherwise agreed to in writing.
All calculations are performed using the raw data results.

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Laboratory Results

October 07, 2024

Lab Number	Analyte	Sample Name	Result	Units	Sample RL	DF	Analysis Method	Analyzed	Batch	Analyte Qualifiers	Facility
Chemistry Parameters											
B014940-01	Chloride	Geo-05	**3060**	mg/Kg wet	96.4	20	EPA 9056A	09/26/24 23:00	BH09393	msu	DL
B014940-01	Sulfate as SO4	Geo-05	**3890**	mg/Kg wet	96.4	20	EPA 9056A	09/26/24 23:00	BH09393	msu	DL
B014940-01	Oxidation/Reduction Potential	Geo-05	**211**	mV		1	ASTM G200-09	10/03/24 09:54	BH09720	nc	DL
B014940-02	Chloride	Geo-10	**3590**	mg/Kg wet	238	50	EPA 9056A	09/27/24 00:44	BH09393		DL
B014940-02	Sulfate as SO4	Geo-10	**10300**	mg/Kg wet	238	50	EPA 9056A	09/27/24 00:44	BH09393		DL
B014940-02	Oxidation/Reduction Potential	Geo-10	**265**	mV		1	ASTM G200-09	10/03/24 10:00	BH09720	nc	DL
B014940-03	Chloride	Geo-13	**221**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 15:36	BH09393		DL
B014940-03	Sulfate as SO4	Geo-13	**332**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 15:36	BH09393		DL
B014940-03	Oxidation/Reduction Potential	Geo-13	**186**	mV		1	ASTM G200-09	10/03/24 10:02	BH09720	nc	DL
B014940-04	Chloride	Geo-15	**2980**	mg/Kg wet	96.2	20	EPA 9056A	09/27/24 00:59	BH09393		DL
B014940-04	Sulfate as SO4	Geo-15	**4870**	mg/Kg wet	96.2	20	EPA 9056A	09/27/24 00:59	BH09393		DL
B014940-04	Oxidation/Reduction Potential	Geo-15	**238**	mV		1	ASTM G200-09	10/03/24 10:08	BH09720	nc	DL
B014940-05	Chloride	Geo-16	**48.8**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 16:04	BH09393		DL
B014940-05	Sulfate as SO4	Geo-16	**184**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 16:04	BH09393		DL
B014940-05	Oxidation/Reduction Potential	Geo-16	**157**	mV		1	ASTM G200-09	10/03/24 10:14	BH09720	nc	DL
B014940-06	Chloride	Geo-20	**7.1**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 16:18	BH09393		DL
B014940-06	Sulfate as SO4	Geo-20	**25.5**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 16:18	BH09393		DL
B014940-06	Oxidation/Reduction Potential	Geo-20	**188**	mV		1	ASTM G200-09	10/03/24 10:28	BH09720	nc	DL
B014940-07	Chloride	Sub-Opt1-1	**839**	mg/Kg wet	49.7	10	EPA 9056A	09/27/24 01:14	BH09393		DL
B014940-07	Sulfate as SO4	Sub-Opt1-1	**2360**	mg/Kg wet	49.7	10	EPA 9056A	09/27/24 01:14	BH09393		DL
B014940-07	Oxidation/Reduction Potential	Sub-Opt1-1	**252**	mV		1	ASTM G200-09	10/03/24 10:19	BH09720	nc	DL
B014940-08	Chloride	Geo-06	**113**	mg/Kg wet	5.0	1	EPA 9056A	09/26/24 16:46	BH09393		DL
B014940-08	Sulfate as SO4	Geo-06	**581**	mg/Kg wet	10.0	2	EPA 9056A	09/27/24 10:09	BH09393		DL
B014940-08	Oxidation/Reduction Potential	Geo-06	**259**	mV		1	ASTM G200-09	10/03/24 10:22	BH09720	nc	DL

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Chemistry Parameters - Quality Control

Analyte	Result	Units	Qualifiers	Sample RL	DF	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit
Batch BH09393 - EPA 9056A											
Blank (BH09393-BLK1)											
Prepared: 09/25/2024 Analyzed: 09/26/2024											
Chloride	< 5.0	mg/Kg wet		5.0	1						
Sulfate as SO4	< 5.0	mg/Kg wet		5.0	1						
LCS (BH09393-BS1)											
Prepared: 09/25/2024 Analyzed: 09/26/2024											
Chloride	248	mg/Kg wet		5.0	1	267		93	90-110		
Sulfate as SO4	244	mg/Kg wet		5.0	1	267		92	90-110		
Matrix Spike (BH09393-MS1)											
Prepared: 09/25/2024 Analyzed: 09/27/2024											
Source: B014940-01											
Chloride	3200	mg/Kg wet	msu	96.8	20	194	3060	75	80-120		
Sulfate as SO4	4110	mg/Kg wet	msu	96.8	20	194	3890	112	80-120		
Matrix Spike Dup (BH09393-MSD1)											
Prepared: 09/25/2024 Analyzed: 09/27/2024											
Source: B014940-01											
Chloride	3450	mg/Kg wet	msu	97.2	20	194	3060	200	80-120	7	20
Sulfate as SO4	4330	mg/Kg wet	msu	97.2	20	194	3890	229	80-120	5	20
Batch BH09720 - ASTM G200-09											
LCS (BH09720-BS1)											
Prepared & Analyzed: 10/03/2024											
Oxidation/Reduction Potential	219	mV	nc		1	220		100	90-110		

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Qualifiers and Definitions

Item	Definition
RL	Reporting Limit (Corrected for dilution factor when applicable due to sample preparation variation.)
MDL	Method Detection Limit (Corrected for sample preparation variation.)
DF	Dilution Factor
DL	Indicates test performed by RMB Environmental Laboratories - Detroit Lakes
msu	Matrix spike and/or matrix spike duplicate recovery was outside of laboratory control limits due to analyte concentration at 4 times or greater than spike concentration. The QC batch was accepted based on acceptable LCS recovery.
nc	Analyte not available for accreditation under the Minnesota Environmental Laboratory Accreditation Program.

RMB Environmental Laboratories, Inc.

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CHAIN OF CUSTODY RECORD

Electronic version available at Rmbel.com/lab/chains-of-custody

B014940

Client: Soil Engineering Testing, Inc	Phone #: 952-406-8957	Fax #: N/A
Project Name: Rex Solaire Solar	Project Task Code: N/A	PO/WO #: 15366
Sampler: (print name) N/A	Sampler Phone #: N/A	
Report to: Tyler Sandoz	Bill to: Tyler Sandoz	
Report to Email: tsandoz@soilengineeringtesting.com	Bill to Email: tsandoz@soilengineeringtesting.com	

❑ "EQUIS" EDD Lab Format - MPCA Data Submittal

Preservation Code:
A = None
B = H_2SO_4
C = HNO_3
D = HCL
E = NaOH
F = Zn Acetate
G = Na_2S_2O3
H = Ascorbic Acid
I = MeOH
J = Other

Sample Number	Station ID/Sample Description	Date	Time	# of Bottles	Sample Method	Start Depth (ft)	End Depth (ft)	Sample Type	Matrix	Preserved at Collection	Preserved at Lab Receipt	Analyses Requested: Sulfate Ion	Chloride Ion	Redox	
												A	A	A	Preservation Code
												N	N	N	Field Filtered Y/N
01	GEO-05	9/23/24	9:00	1	SW-Soil	1'	4'	Sample	Soil-Sub			X	X	X	
02	GEO-10	↓	↓	1	↓	1'	4'	↓	↓			X	X	X	
03	GEO-13			1		1'	4'					X	X	X	
04	GEO-15			1		1'	4'					X	X	X	
05	GEO-16			1		1'	4'					X	X	X	
06	GEO-20			1		1'	4'					X	X	X	
07	SUB-OPT1-1			1		3'	5'					X	X	X	
08	GEO-06	↓	↓	1	↓	0.5'	1.5'	↓	↓			X	X	X	

Page 6 of 6

TS (Initials) In the event that samples are received by the lab at a temperature greater than 6° C, (10° C for micro) I hereby authorize RMB Environmental Laboratories to process the samples as received.

_____ (Initials) In the event that samples are received by the lab at a temperature greater than 6° C, (10° C for micro) please contact client at phone # _____ before processing samples.

Relinquished by: (client signature)	Print Name:	Date	Time	Shipping to Lab:
Tyler Sandoz	Tyler Sandoz	9/23/24	12:00	❑ Speedee ❑ UPS ❑ USPS ❑ FedEx ❑ Courier ❑ RMB Courier ☒ Hand Delivery

FOR LAB USE ONLY

Received by Lab: (signature)	Date	Time	Additional Fees: Mileage	Field Staff	Shipping	Courier	Equipment	Other
Allen Thompson	9/23/24	1651						

☐ DOES meet proper sample storage and transportation guidelines ☐ Received on ice ☒ Received at room temperature Received Temp: 23.5 °C Therm. ID: LTG 01

☒ Does NOT meet proper sample storage and transportation guidelines ☒ Samples received same day as collection Chlorine: No Yes N/A

Comments: NO Sign of Cooling -AbT 9/23/24

VOC vials received w/o headspace bubble < 6mm: Yes No N/A

Page ❑ of ❑

Appendix I

Field Electrical Resistivity Test Results

WENNER SOUNDING

PROJECT: Rex Solaire Solar Project

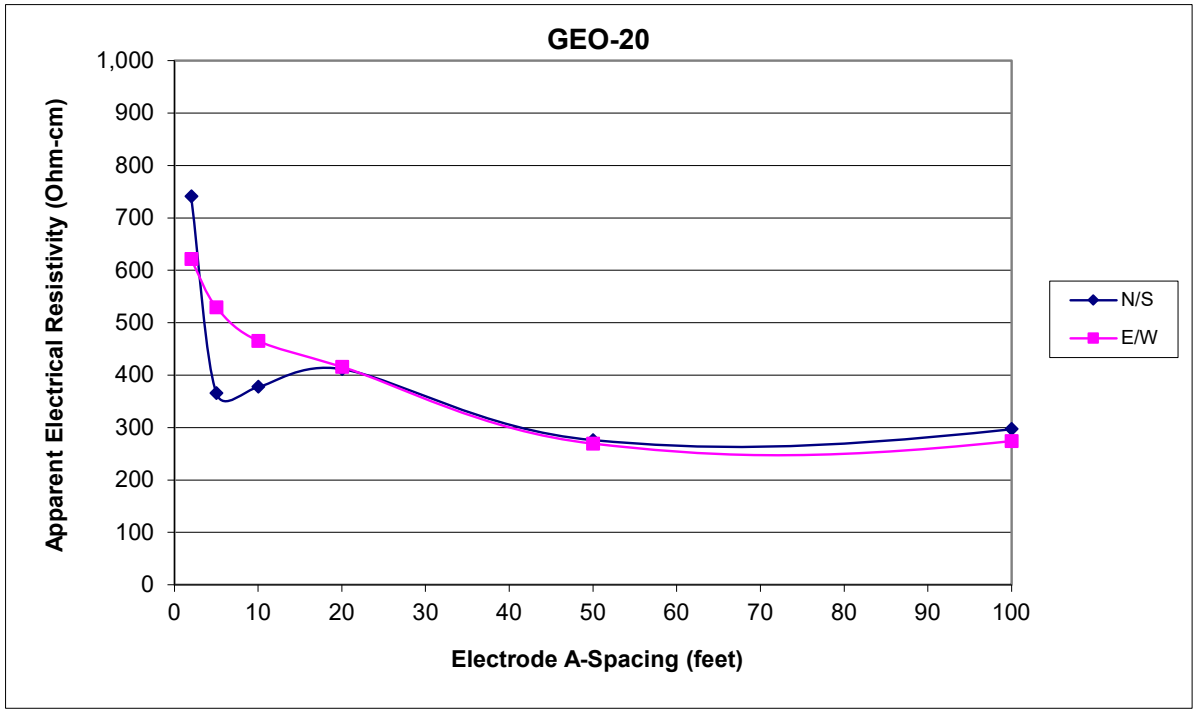
Investigation ID GEO-20 **Date** 9/10/2024

Observer CJA2

Structure ID Solar Array

Electrode Spacing "a" feet	Resistance V/I Ohms	Geometric Factor K=2πa feet	Apparent Resistivity ρ_a=K(V/I) Ohm-feet	Apparent Resistivity ρ_a=K(V/I) Ohm-cm
N/S Orientation				
2	1.9	12.57	24.29	740.5
5	0.381	31.42	11.98	365.3
10	0.197	62.83	12.38	377.4
20	0.107	125.66	13.50	411.5
50	0.029	314.16	9.05	275.8
100	0.016	628.32	9.74	296.9
200	0.014	1256.64	17.47	532.5
E/W Orientation				
2	1.6	12.57	20.37	620.9
5	0.552	31.42	17.35	529.0
10	0.243	62.83	15.25	464.9
20	0.109	125.66	13.63	415.7
50	0.028	314.16	8.83	269.1
100	0.014	628.32	8.98	273.9
200	0.014	1256.64	16.96	517.2

Cultural Features	None
Ground Cover	Plowed field, Dry
Weather	95°F, Clear
Line Location and Bearing	NS/EW

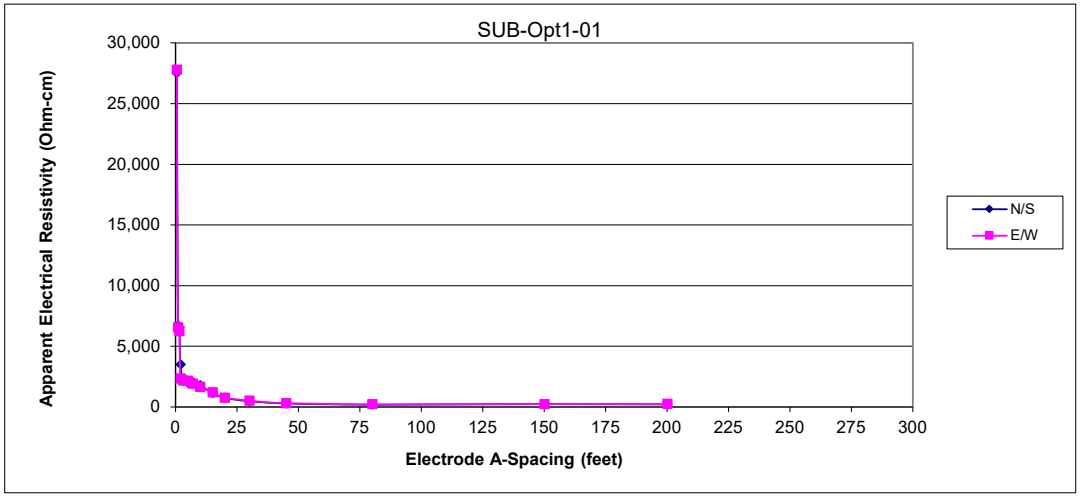


WENNER SOUNDING

PROJECT: Rex Solaire Solar Project

Investigation ID SUB-Opt1-01 **Date** 9/11/2024
Observer CJA2
Structure ID Substation

Electrode Spacing "a"	Resistance V/I	Geometric Factor K=2πa	Apparent Resistivity ρ_a=K(V/I)	Apparent Resistivity ρ_a=K(V/I)
feet	Ohms	feet	Ohm-feet	Ohm-cm
N/S Orientation				
0.5	287.3	3.14	902.58	27517.7
1	35.0	6.28	219.91	6704.6
1.5	22.8	9.42	214.89	6551.4
2	9.143	12.57	114.89	3502.9
3	3.751	18.85	70.70	2155.6
5	2.081	31.42	65.38	1993.2
7	1.498	43.98	65.89	2008.8
10	0.933	62.83	58.59	1786.3
15	0.399	94.25	37.58	1145.6
20	0.196	125.66	24.64	751.3
30	0.083	188.50	15.70	478.7
45	0.033	282.74	9.19	280.2
80	0.012	502.66	6.08	185.4
150	0.008	942.48	7.63	232.7
200	0.005	1256.64	6.66	203.1
300	N/A	1884.96	N/A	N/A
E/W Orientation				
0.5	290.0	3.14	911.06	27776.3
1	34.2	6.28	214.89	6551.4
1.5	21.7	9.42	204.52	6235.3
2	6.1	12.57	76.66	2337.0
3	3.783	18.85	71.31	2174.0
5	2.253	31.42	70.78	2157.9
7	1.431	43.98	62.93	1918.7
10	0.855	62.83	53.75	1638.6
15	0.416	94.25	39.16	1193.9
20	0.198	125.66	24.91	759.3
30	0.091	188.50	17.10	521.2
45	0.035	282.74	9.84	300.0
80	0.014	502.66	7.04	214.5
150	0.008	942.48	7.45	227.0
200	0.006	1256.64	7.92	241.4
300	N/A	1884.96	N/A	N/A



Notes	Unable to obtain accurate measurement for 300-foot "a"-spacings.
Ground Cover	Dry Grass, Dry
Weather	68°F, Clear
Line Location and Bearing	NS/EW

Appendix J

Thermal Resistivity Test Results

Thermal Resistivity Report ASTM D:5334

Project: **Rex Solaire Solar** Job #: **15366**

Client: **Barr Engineering Company** Date: **10/4/24**

Boring	Specimen Type	Depth (ft)	Type	Classification	Proctor Values		Initial Conditions			Dry
					Maximum Dry Density (PCF)	Optimum Moisture (%)	Dry Density (PCF)	WC (%)	Thermal Resistivity (ºC-cm/W)	Thermal Resistivity (ºC-cm/W)
Geo-10A	**Reconstituted**	**3-5**	**Bulk**	**Silty Sand (SM)**	**114.7**	**13.9%**	**97.5**	**13.6%**	**85**	**254**
Geo-15	**Reconstituted**	**3-5**	**Bulk**	**Fat Clay with sand (CH)**	**104.2**	**21.0%**	**88.5**	**20.9%**	**98**	**245**
Geo-16	**Reconstituted**	**3-5**	**Bulk**	**Sandy Lean Clay (CL)**	**103.7**	**18.0%**	**88.3**	**13.7%**	**111**	**369**
Geo-20	**Reconstituted**	**3-5**	**Bulk**	**Clayey Sand (SC)**	**98.6**	**23.5%**	**83.7**	**22.3%**	**108**	**389**
	Specimens reconstituted to approximately 85% of maximum standard proctor density near the as received moisture content.									

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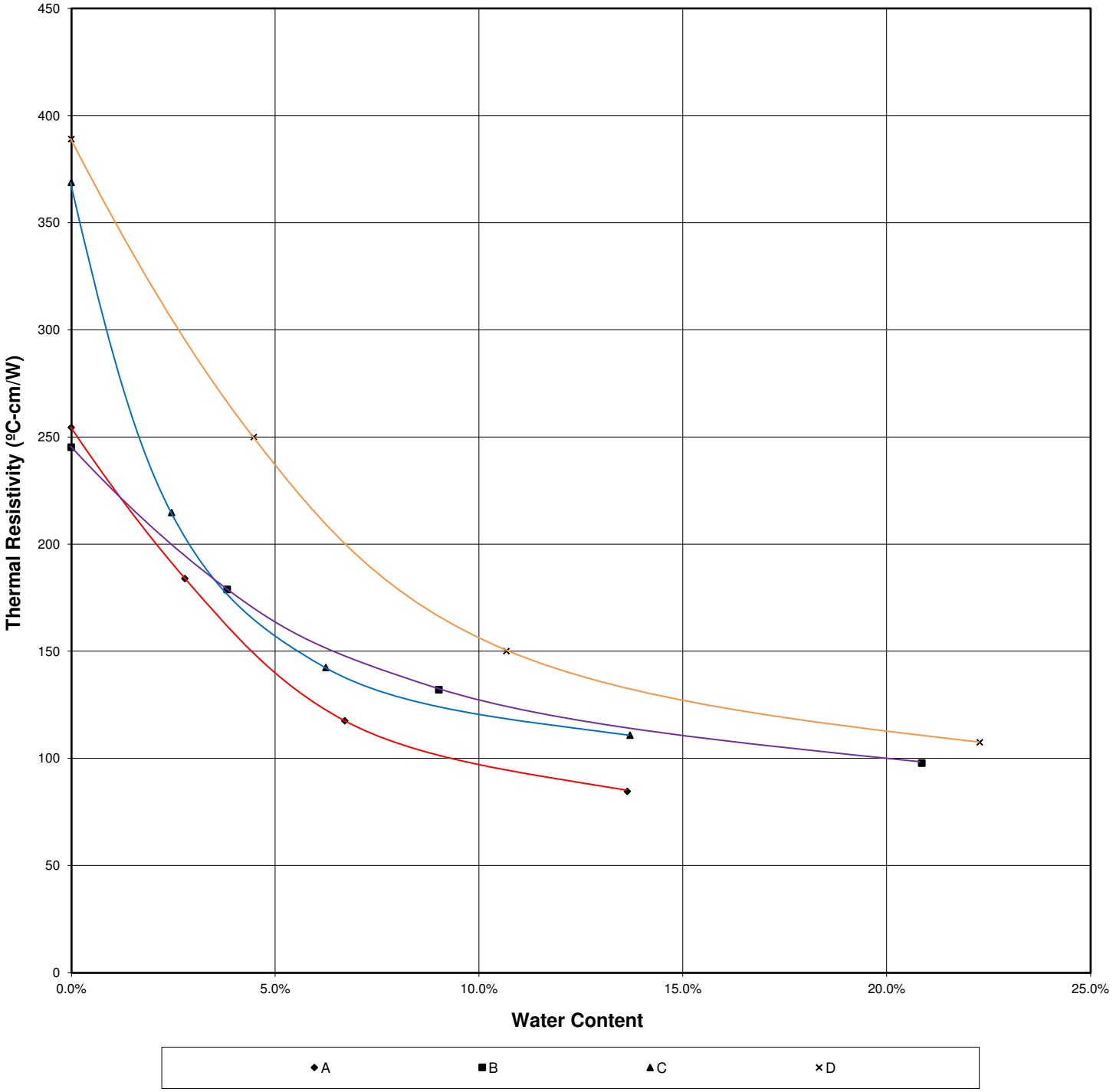
Thermal Resistivity Report ASTM D:5334

Project: Rex Solaire Solar — Job: 15366

Client: Barr Engineering Company — Date: 10/4/24

	Boring	Depth (ft)
Specimen A:	Geo-10A	3-5
Specimen B:	Geo-15	3-5
Specimen C:	Geo-16	3-5
Specimen D:	Geo-20	3-5

Thermal Dryout Curves (Water Content vs. Resistivity)



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Bloomington, MN 55431

http://www.soilengineeringtesting.com

Thermal Resistivity Report ASTM D:5334

Project: **Rex Solaire Solar** Job #: **15366**

Client: **Barr Engineering Company** Date: **10/4/24**

Boring	Sample	Specimen Type	Depth (ft)	Type	Classification	Proctor Values		Initial Conditions			Dry
						Maximum Dry Density (PCF)	Optimum Moisture (%)	Dry Density (PCF)	WC (%)	Thermal Resistivity (ºC-cm/W)	Thermal Resistivity (ºC-cm/W)
Sub-Opt1-01	**85%**	**Reconstituted**	**3-5**	**Bulk**	**Lean Clay with sand (CL)**	**109.6**	**16.8%**	**93.4**	**10.1%**	**120**	**303**
Sub-Opt1-01	**95%**	**Reconstituted**	**3-5**	**Bulk**	**Lean Clay with sand (CL)**	**109.6**	**16.8%**	**104.1**	**12.0%**	**84**	**209**
		Specimens reconstituted to approximately 85% and 95% of maximum standard proctor density near the as received moisture content*.									
		***95% specimen compacted at lowest possible moisture content to achieve target density**									

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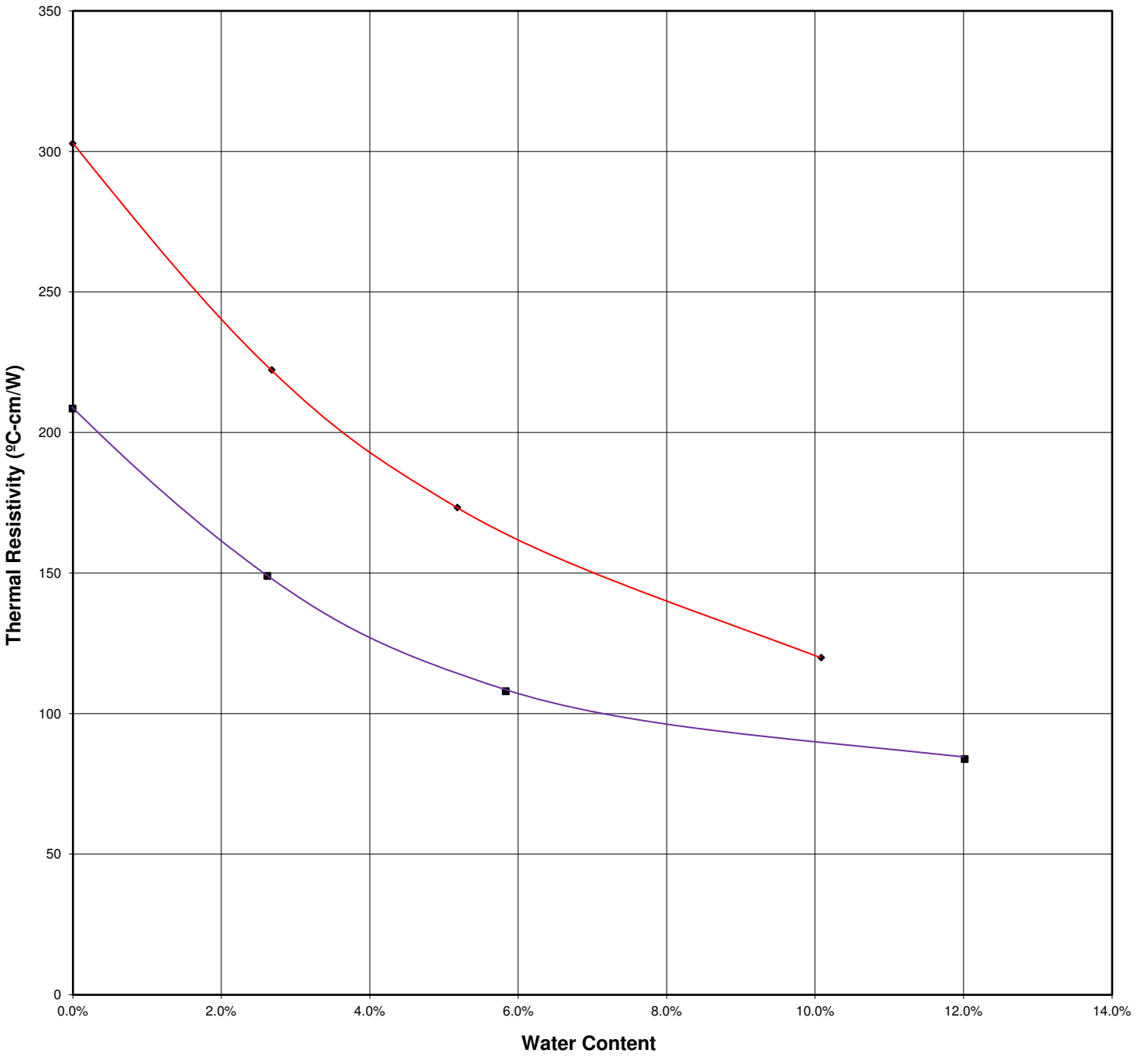
Thermal Resistivity Report ASTM D:5334

Project: Rex Solaire Solar — Job: 15366

Client: Barr Engineering Company — Date: 10/4/24

	Boring	Depth (ft)	Compaction
Specimen A:	Sub-Opt1-01	3-5	85%
Specimen B:	Sub-Opt1-01	3-5	95%

Thermal Dryout Curves (Water Content vs. Resistivity)



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

http://www.soilengineeringtesting.com

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

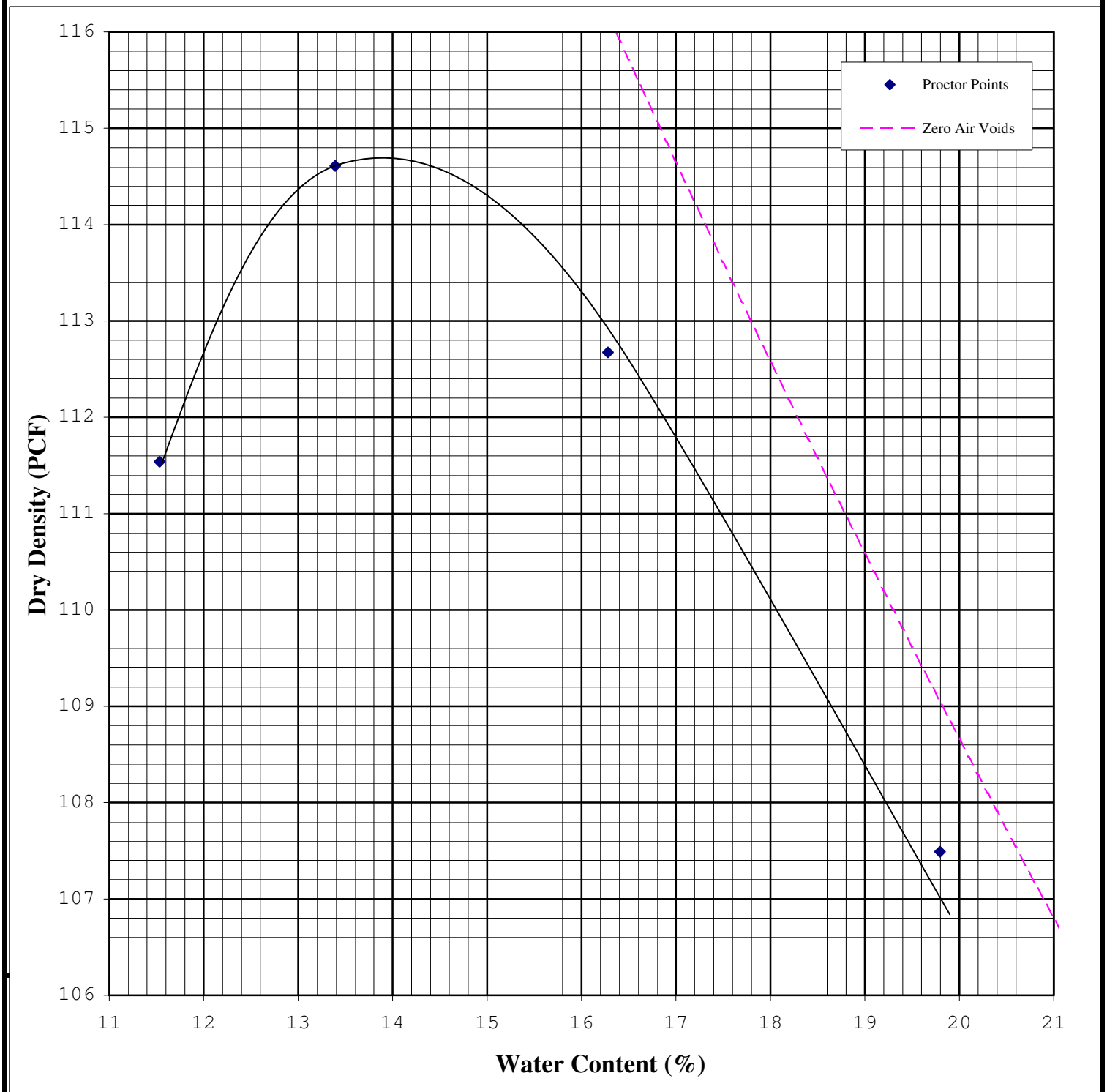
Client: **Barr Engineering Company** Job No. **15366**

Boring No.: **Geo-10A** Sample: Depth(ft): **3-5** Location:

Soil Type: **Silty Sand (SM)**

As Received W.C. (%): **13.6** LL: **21** PL: **N/A** PI: **NP** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **114.7** Opt. Water Content (%): **13.9**



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Bloomington, MN 55431

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

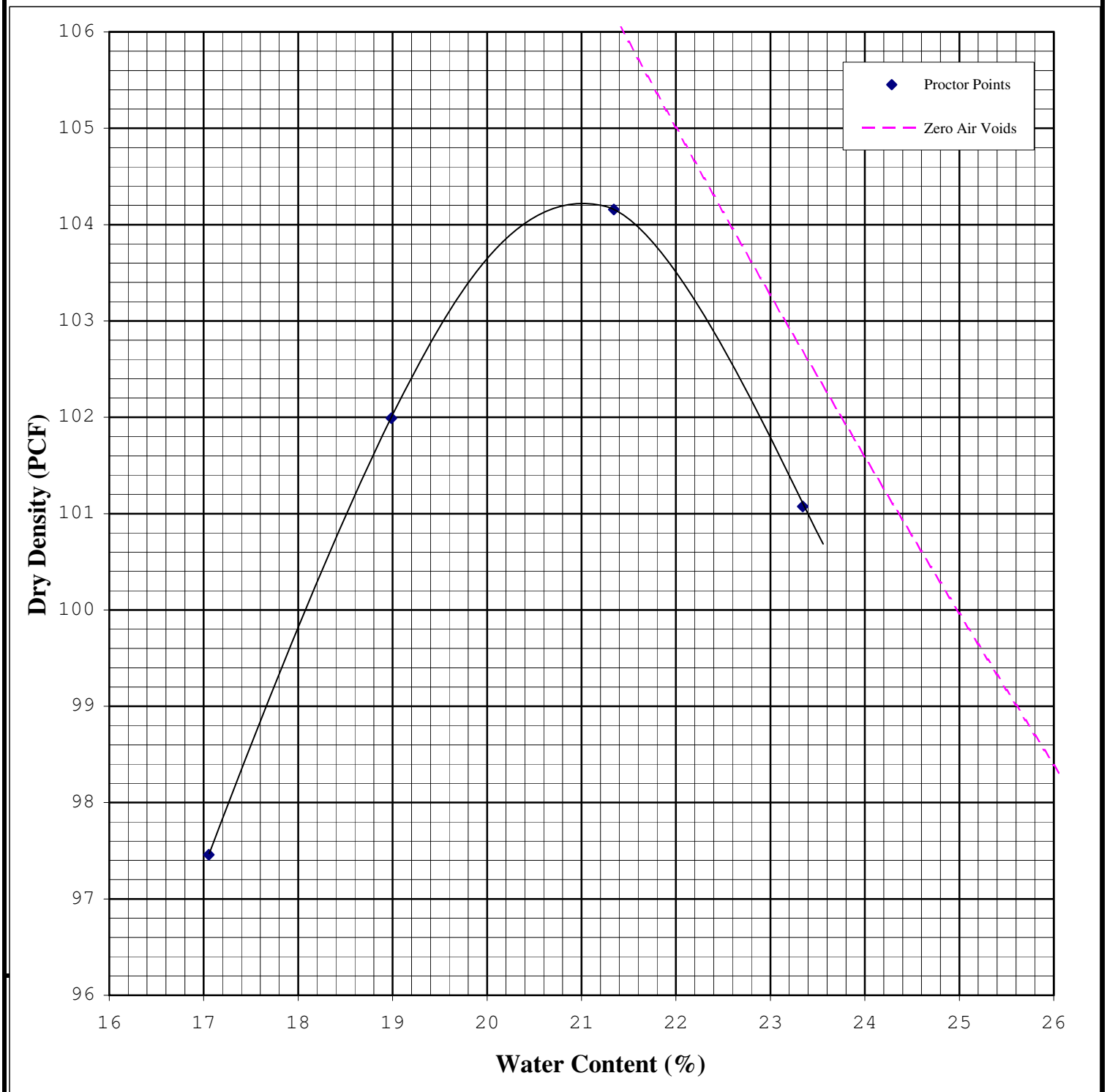
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-15** Sample: Depth(ft): **3-5** Location:

Soil Type: **Fat Clay w/sand (CH)**

As Received W.C. (%): **20.7** LL: **50** PL: **16** PI: **34** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **104.2** Opt. Water Content (%): **21.0**



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Bloomington, MN 55431

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

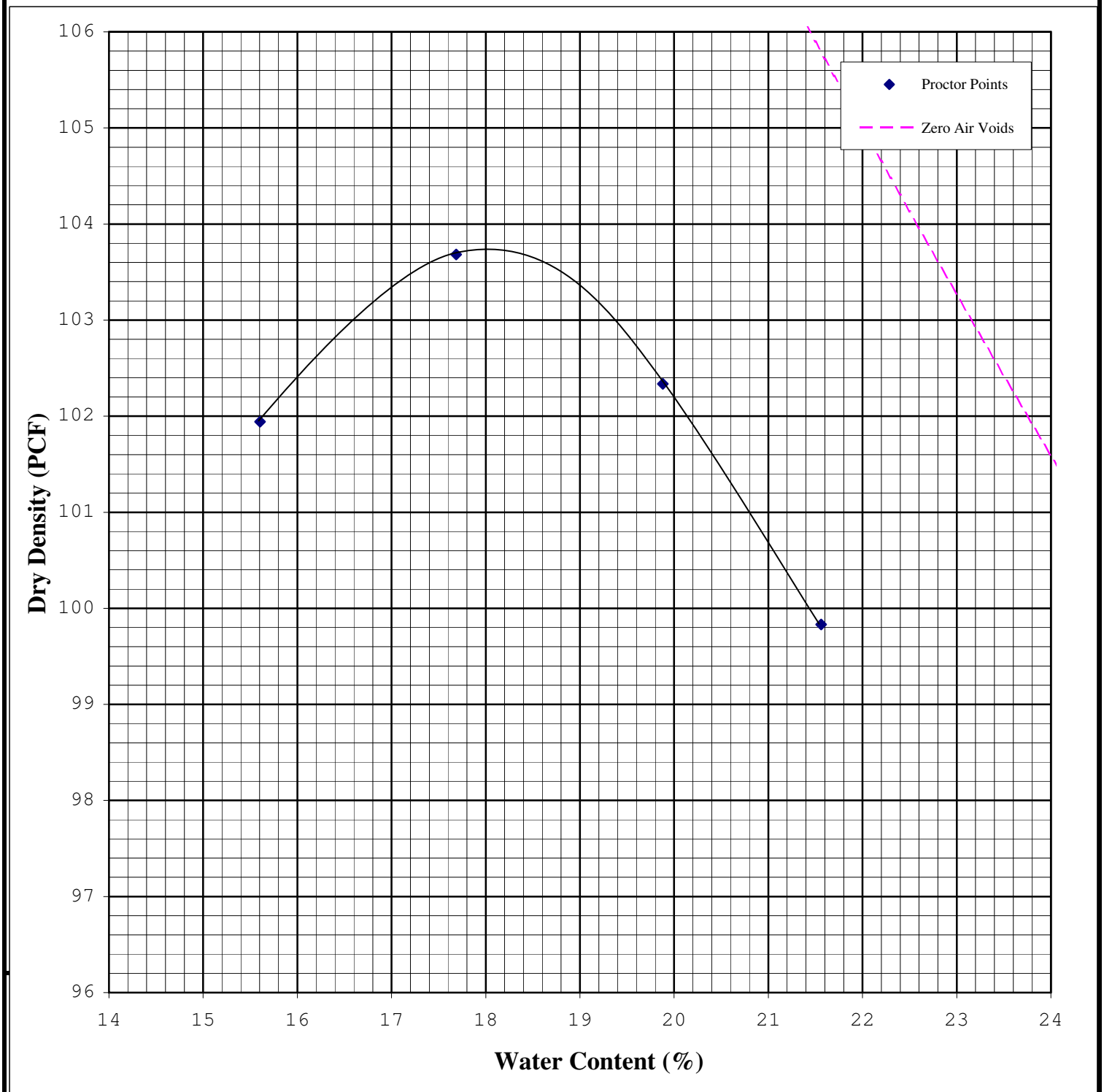
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-16** Sample: Depth(ft): **3-5** Location:

Soil Type: **Sandy Lean Clay (CL)**

As Received W.C. (%): **13.9** LL: **34** PL: **21** PI: **13** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **103.7** Opt. Water Content (%): **18.0**



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

SET-R18a

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

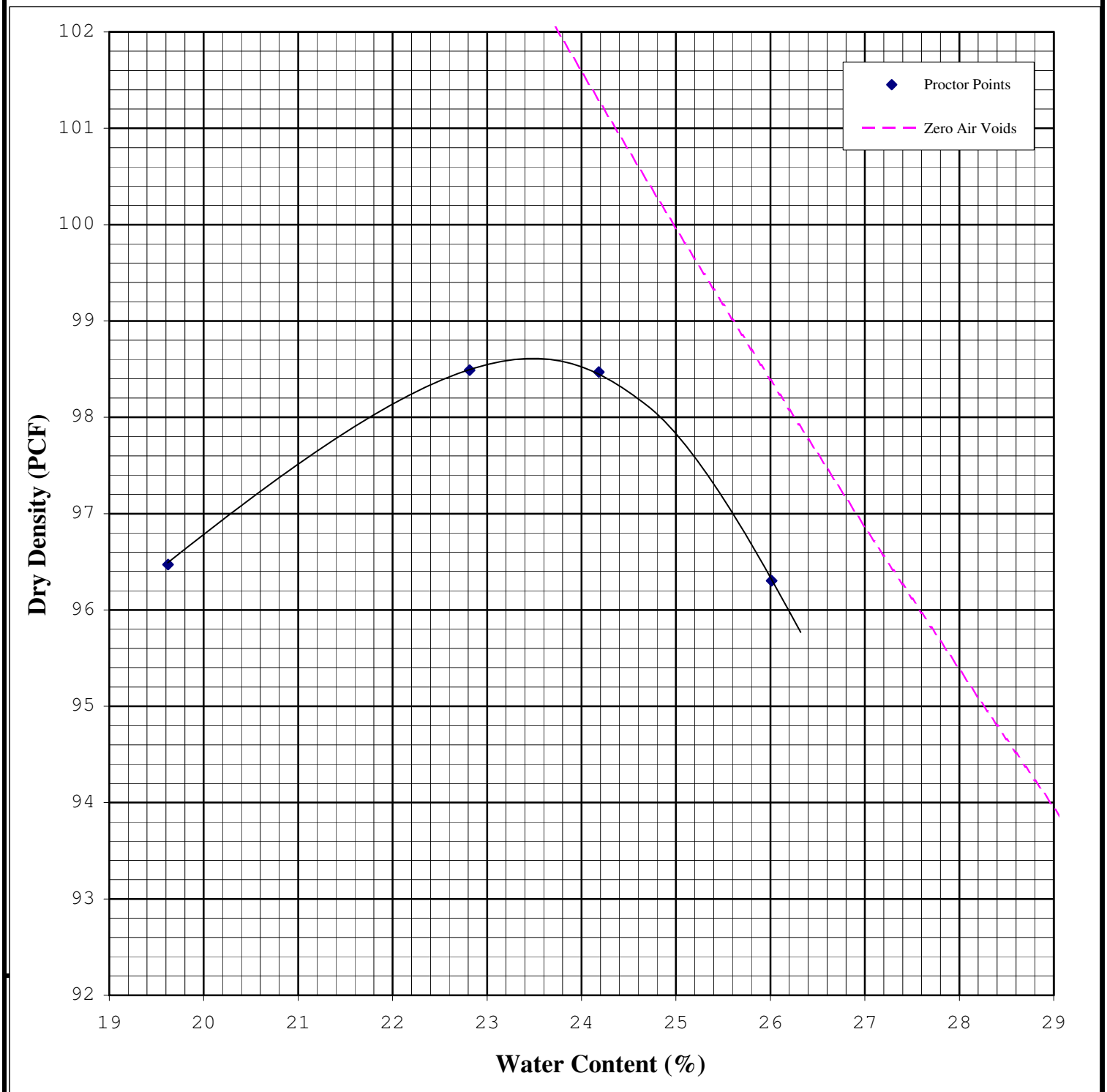
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Geo-20** Sample: Depth(ft): **3-5** Location:

Soil Type: **Clayey Sand (SC)**

As Received W.C. (%): **22.0** LL: **37** PL: **14** PI: **23** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **98.6** Opt. Water Content (%): **23.5**



9530 James Ave South

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Bloomington, MN 55431

Moisture Density Curve ASTM: D698, Method B

Project: **Rex Solaire Solar** Date: **10/1/24**

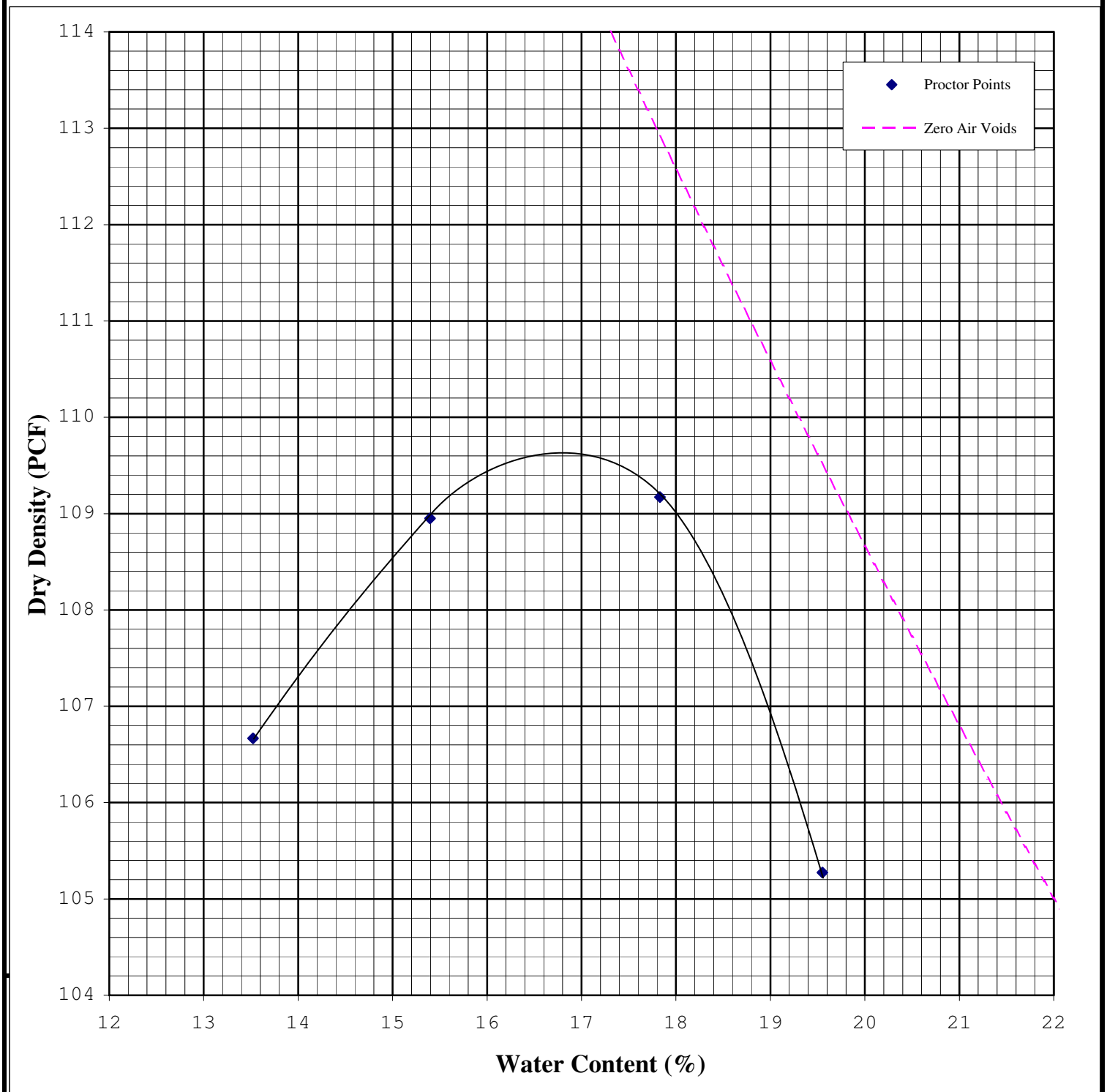
Client: **Barr Engineering Company** Job No. **15366**

Boring No. **Sub-Opt1-01** Sample: Depth(ft): **3-5** Location:

Soil Type: **Lean Clay w/sand (CL)**

As Received W.C. (%): **10.2** LL: **29** PL: **17** PI: **12** Specific Gravity: **2.67** *Assumed

Maximum Dry Density (pcf): **109.6** Opt. Water Content (%): **16.8**



9530 James Ave South

SOIL ENGINEERING TESTING, INC.

Bloomington, MN 55431

Appendix K

General Site Photographs

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-01.



Notes:
Site conditions near investigation location GEO-02.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs





Project Information

Project Name:	Rex Solaire Solar Project
Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-04A.

Notes:
Standing water observed approximately 750 to 800 feet west of investigation location GEO-08A.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-08A.



Notes:
Site conditions near investigation location GEO-09.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-10A.



Notes:
Site conditions near investigation location GEO-11.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

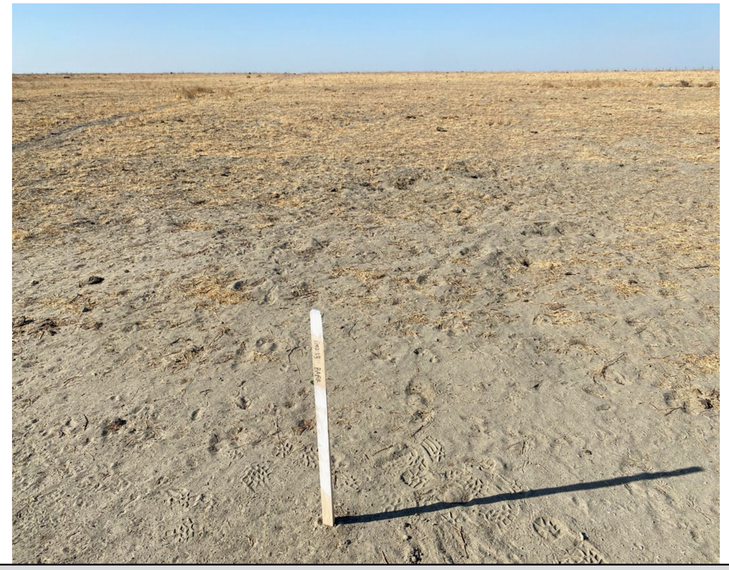
General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-12.



Notes:
Site conditions near investigation location GEO-13.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-14.



Notes:
Site conditions near investigation location GEO-16.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-17.



Notes:
Site conditions near investigation location GEO-18.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs



Project Information

Project Name: Rex Solaire Solar Project
Project Number: 05161003.00
Project Location: Kings County, California
Project Client: Terra-Gen Development Company, LLC

Notes:
Site conditions near investigation location GEO-21.



Notes:
Site conditions near investigation location SUB-Opt1-01.

Barr Engineering Co.

Rex Solaire Solar Project
Kings County, California
Prepared for Terra-Gen Development Company, LLC

General Site Photographs





Project Information

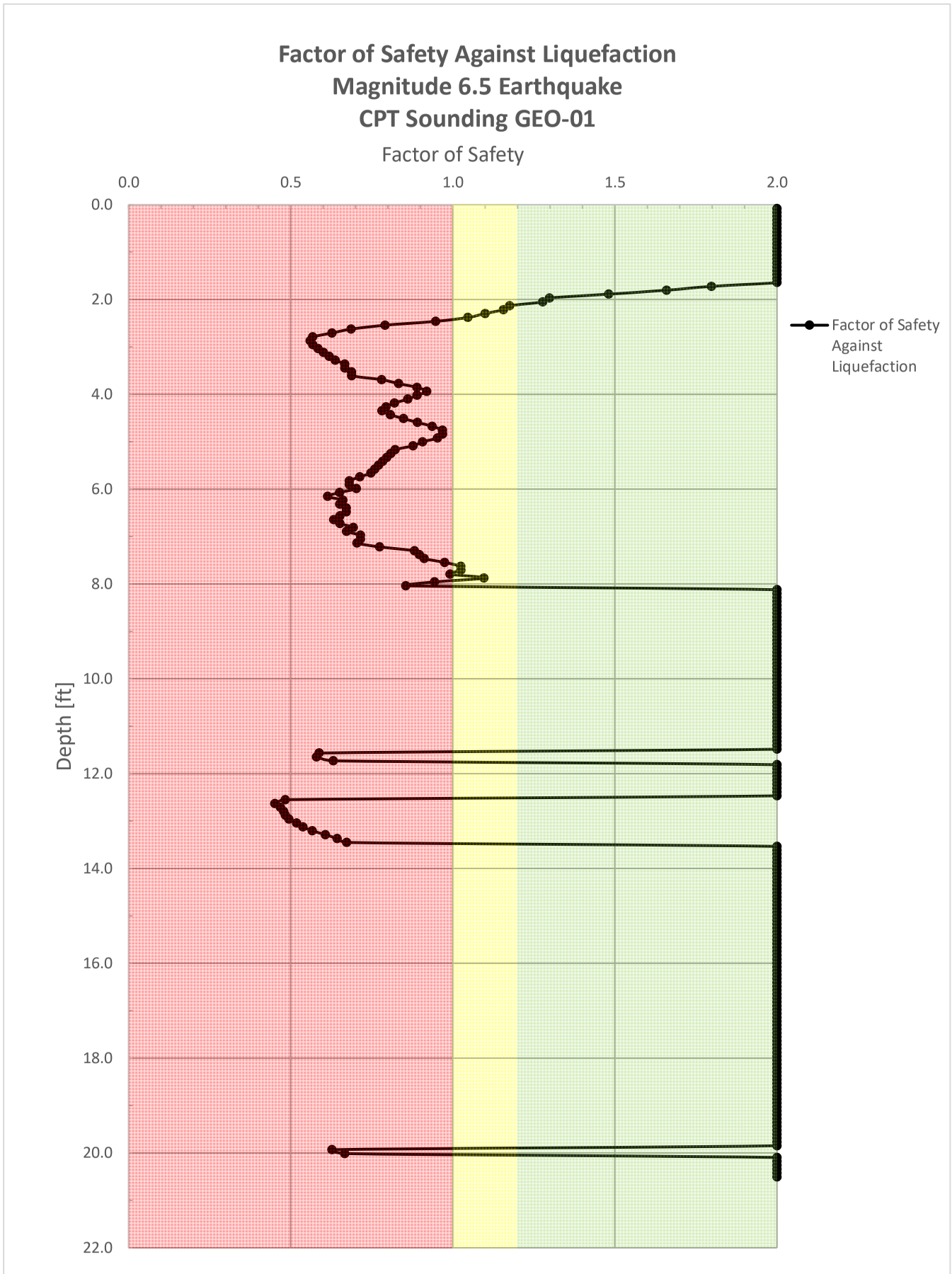
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Project Number:	05161003.00
Project Location:	Kings County, California
Project Client:	Terra-Gen Development Company, LLC

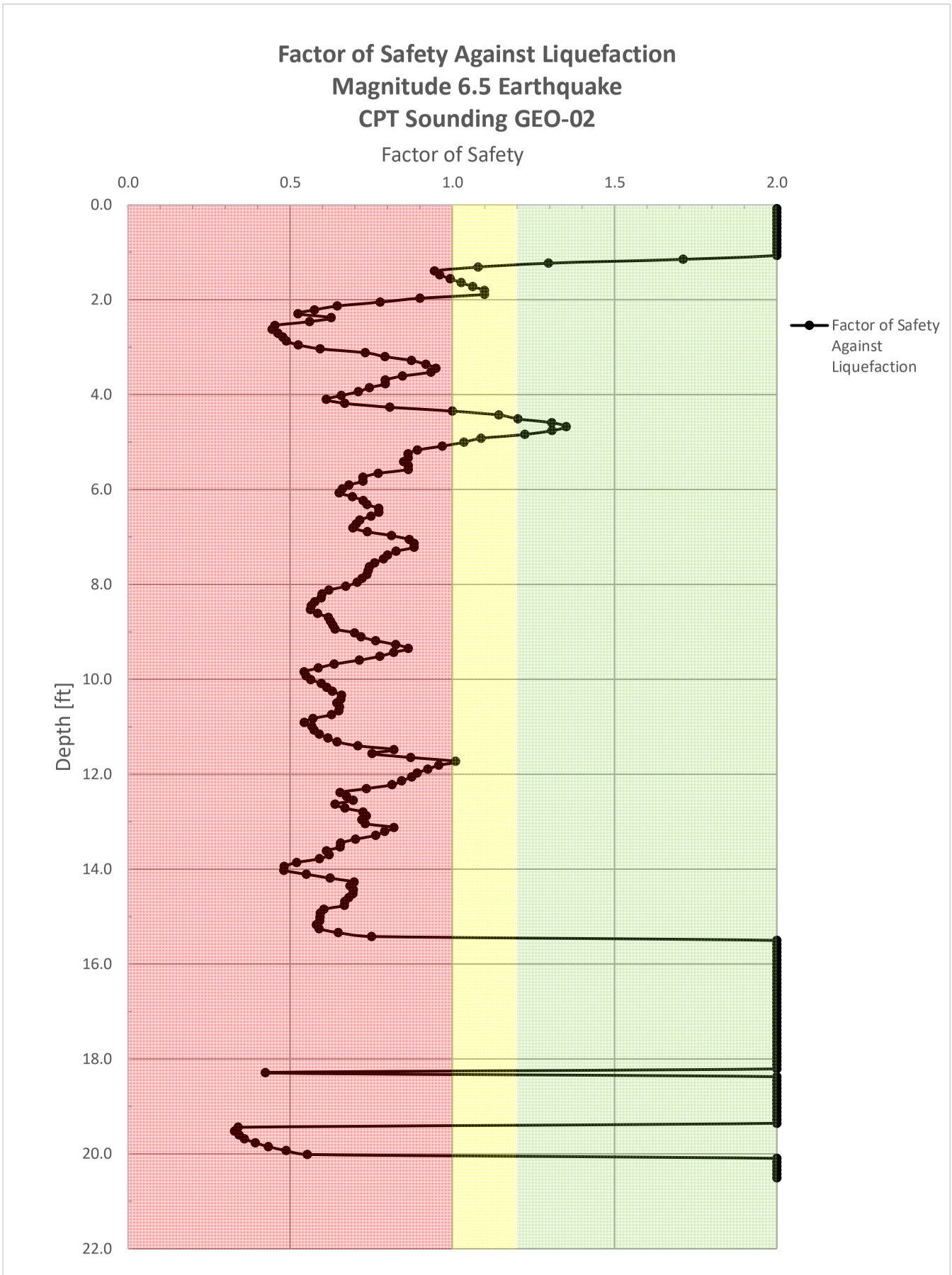
Notes:
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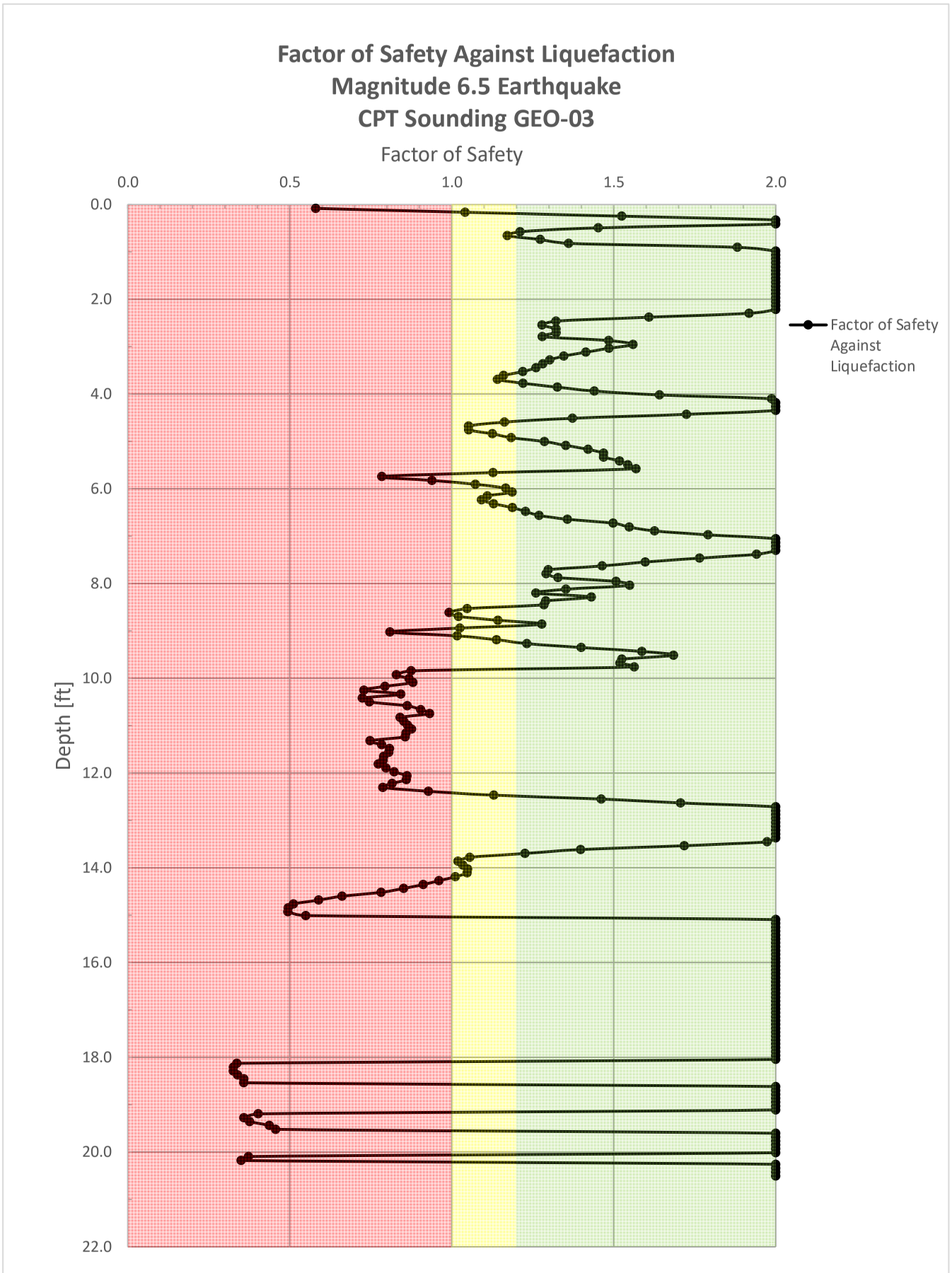
Notes:
Site conditions near investigation location SUB-Opt1-01.

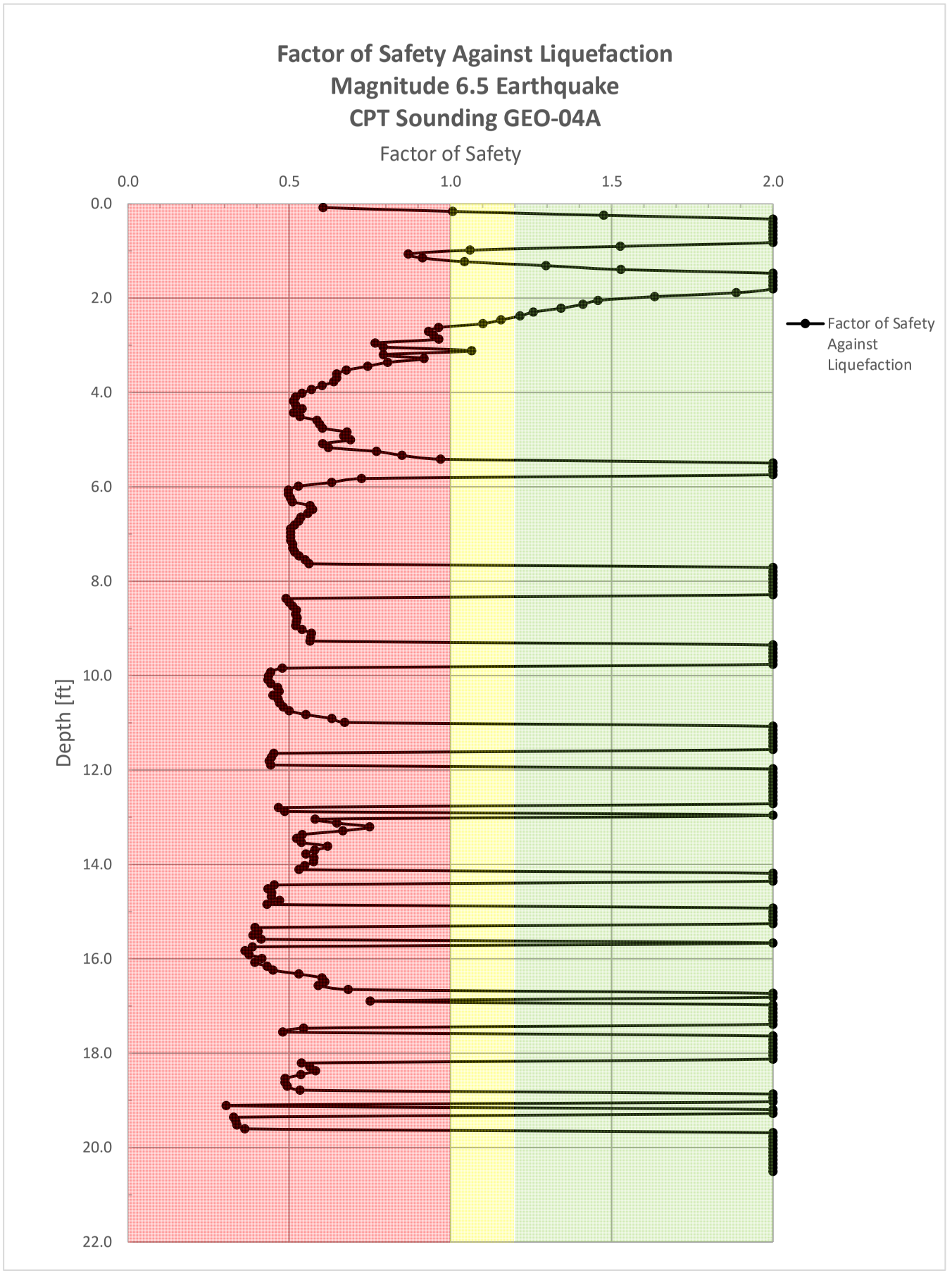
Appendix L

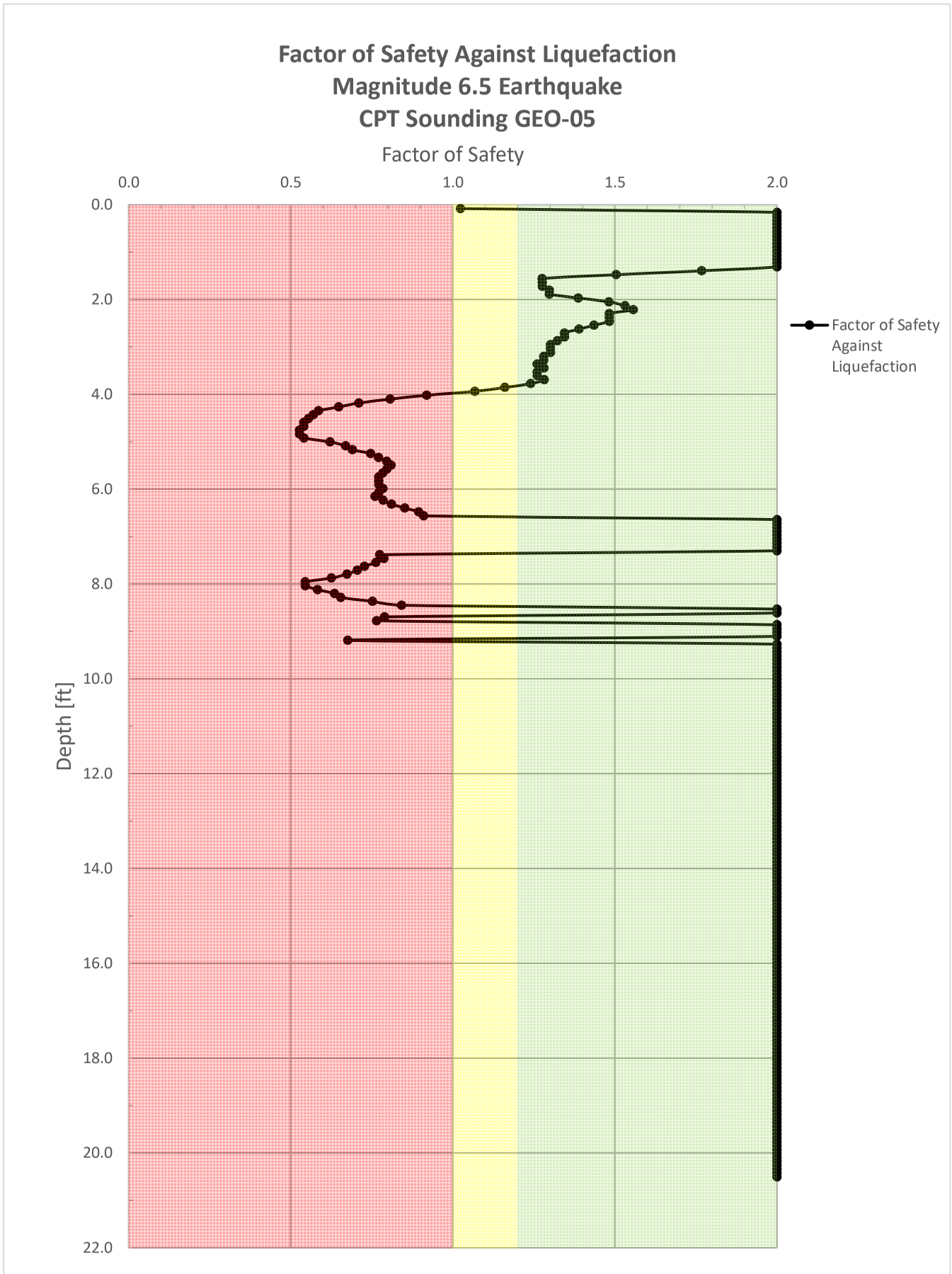
Preliminary Liquefaction Susceptibility Analysis

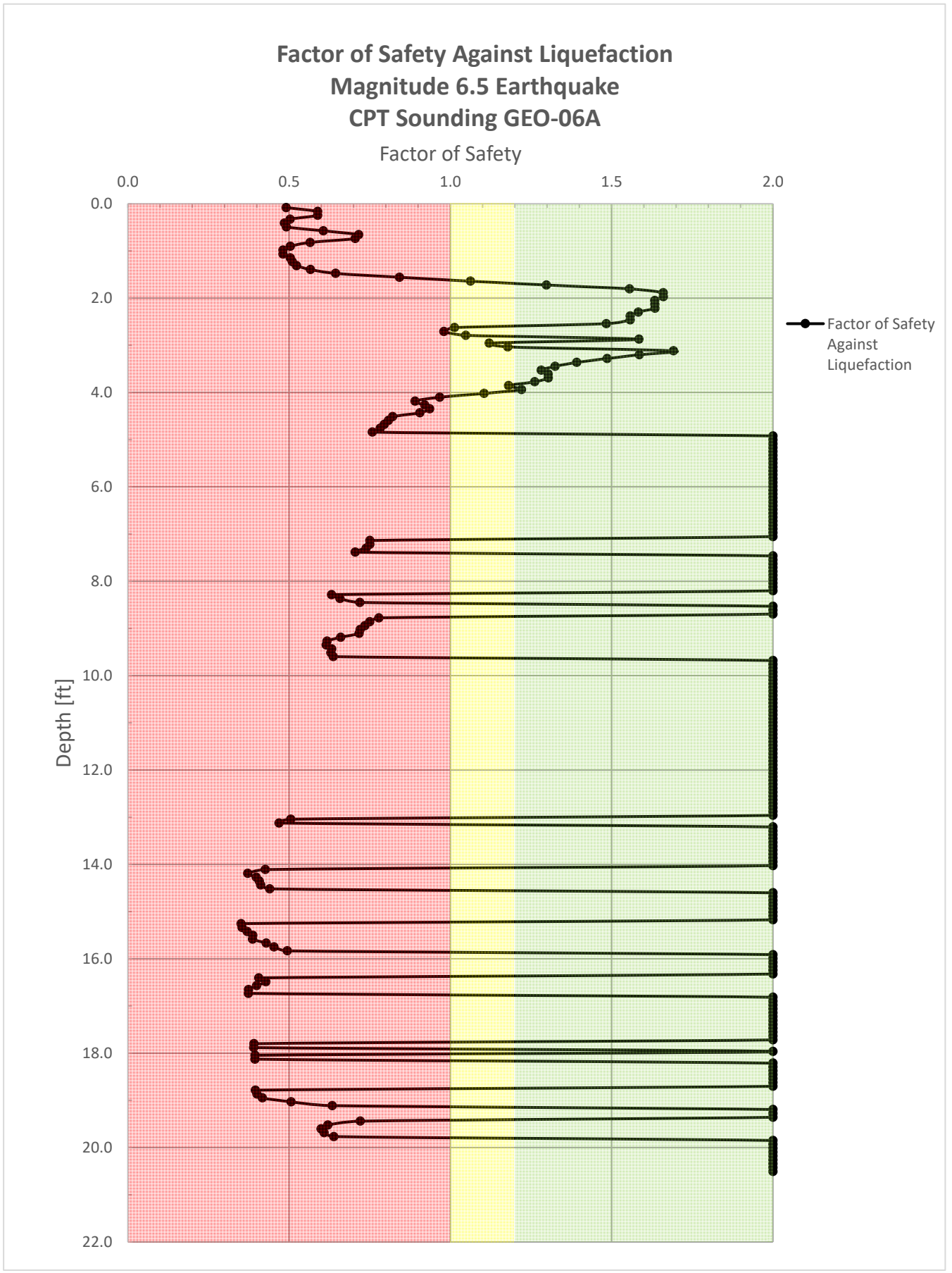

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-01
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

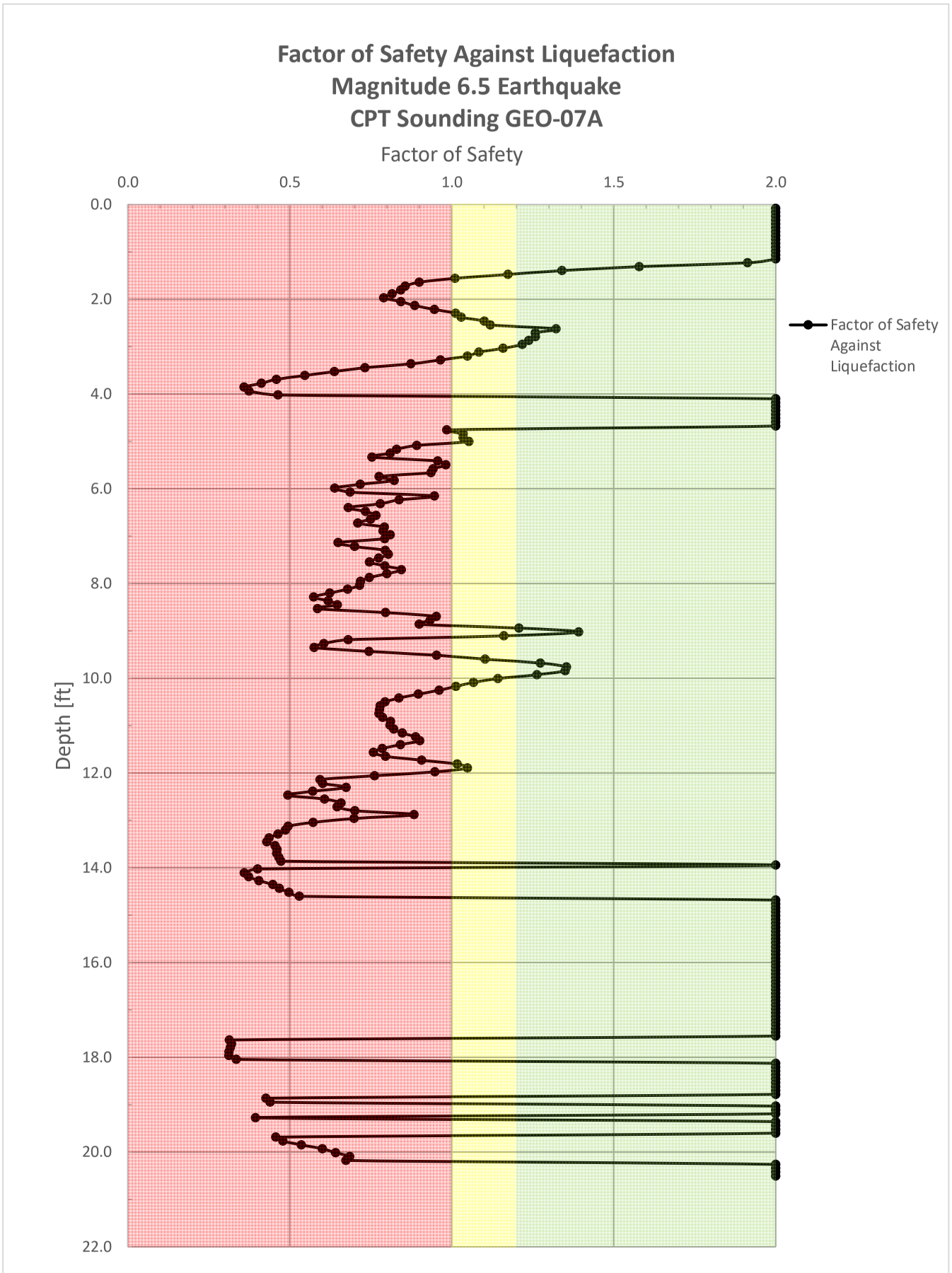

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-02
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

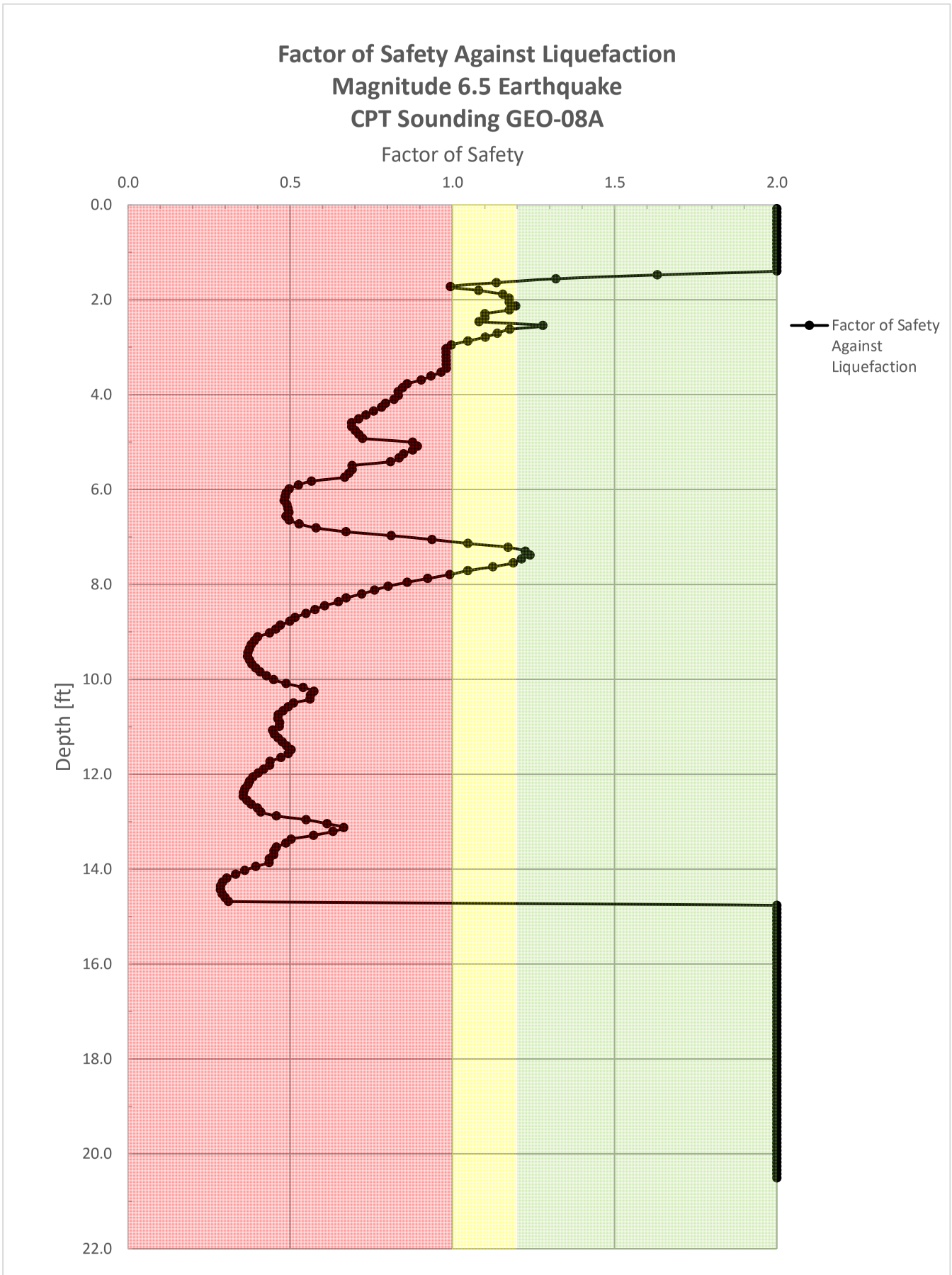

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-03
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

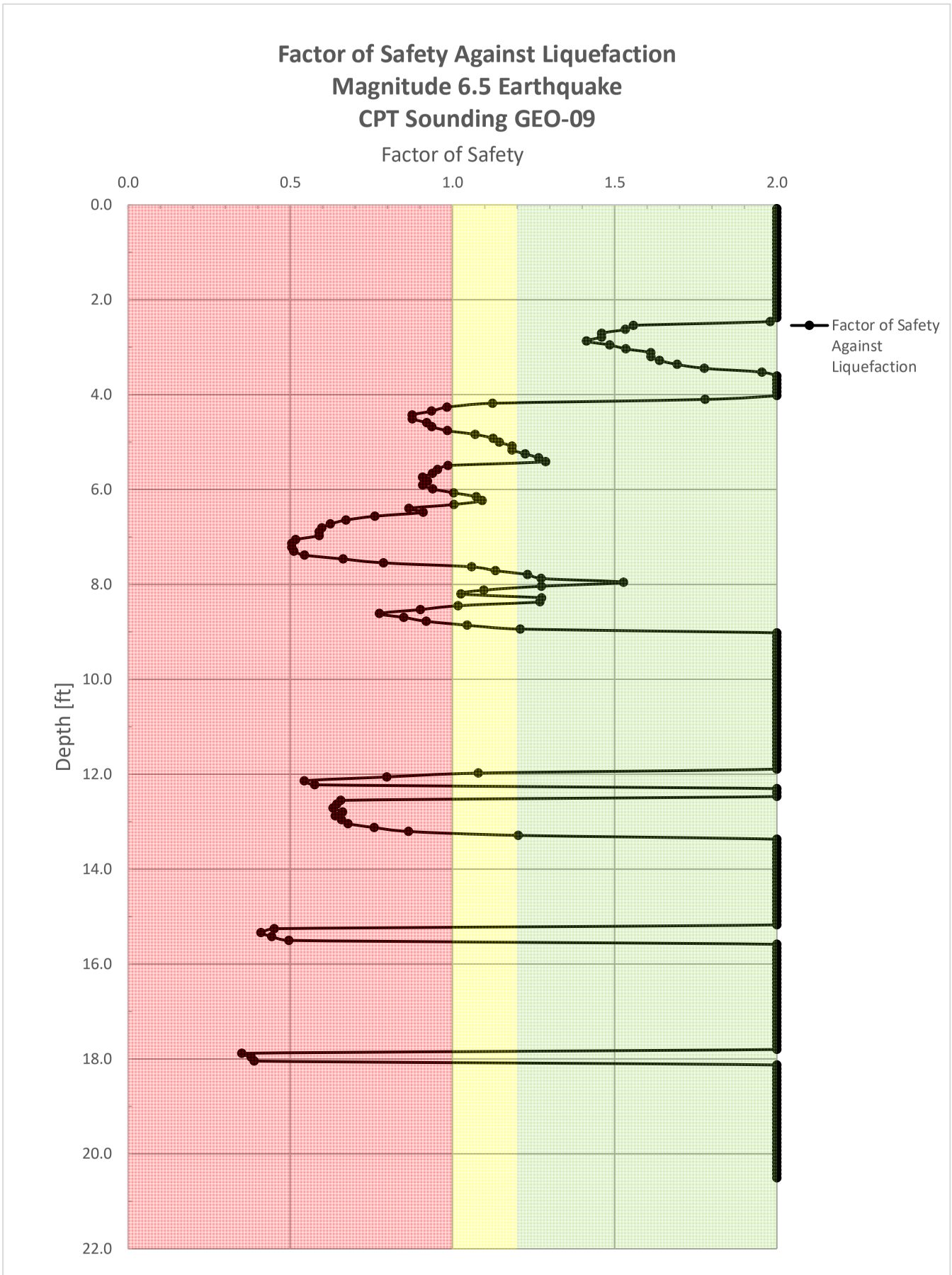

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-04A
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

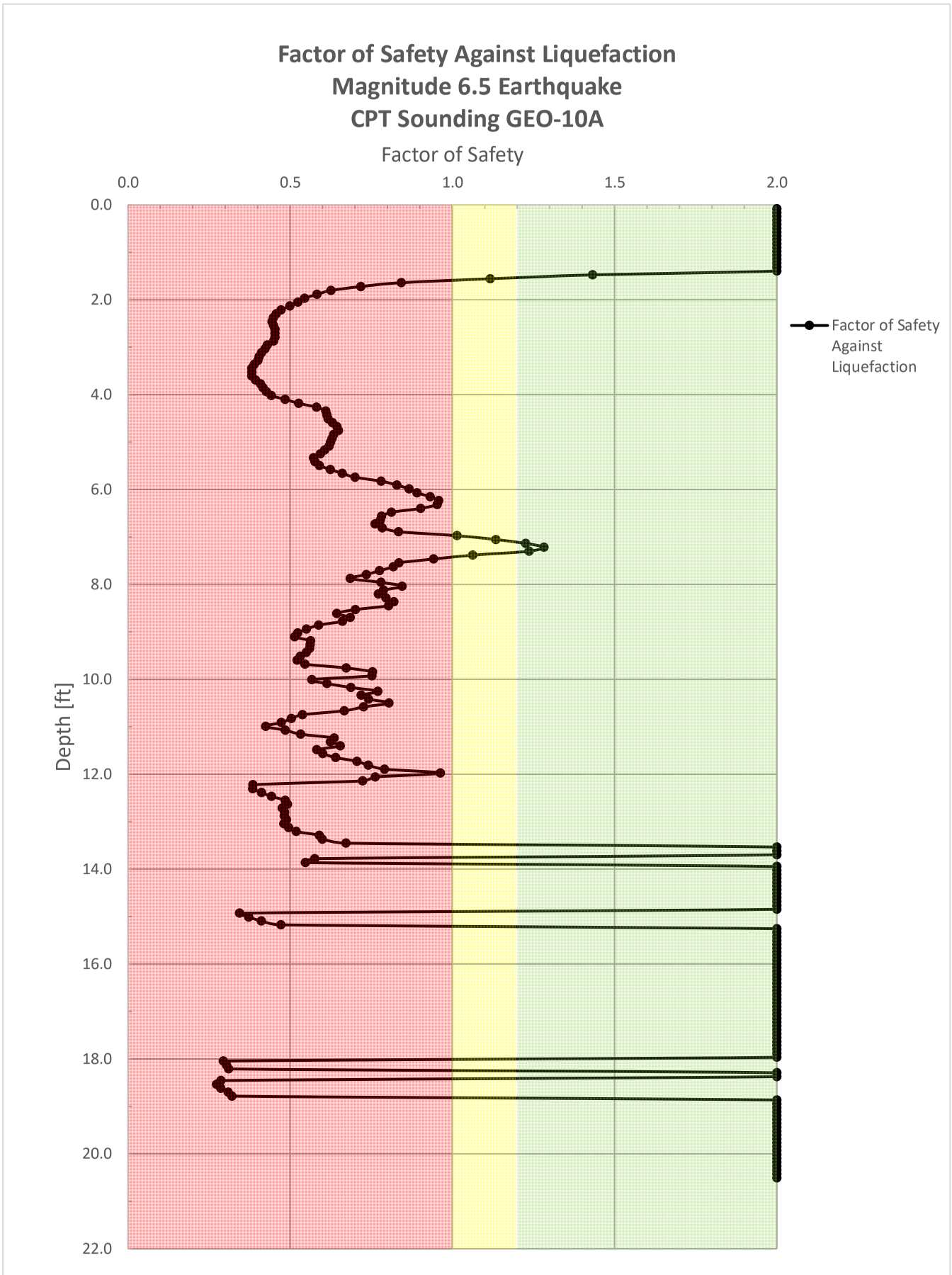

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-05
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

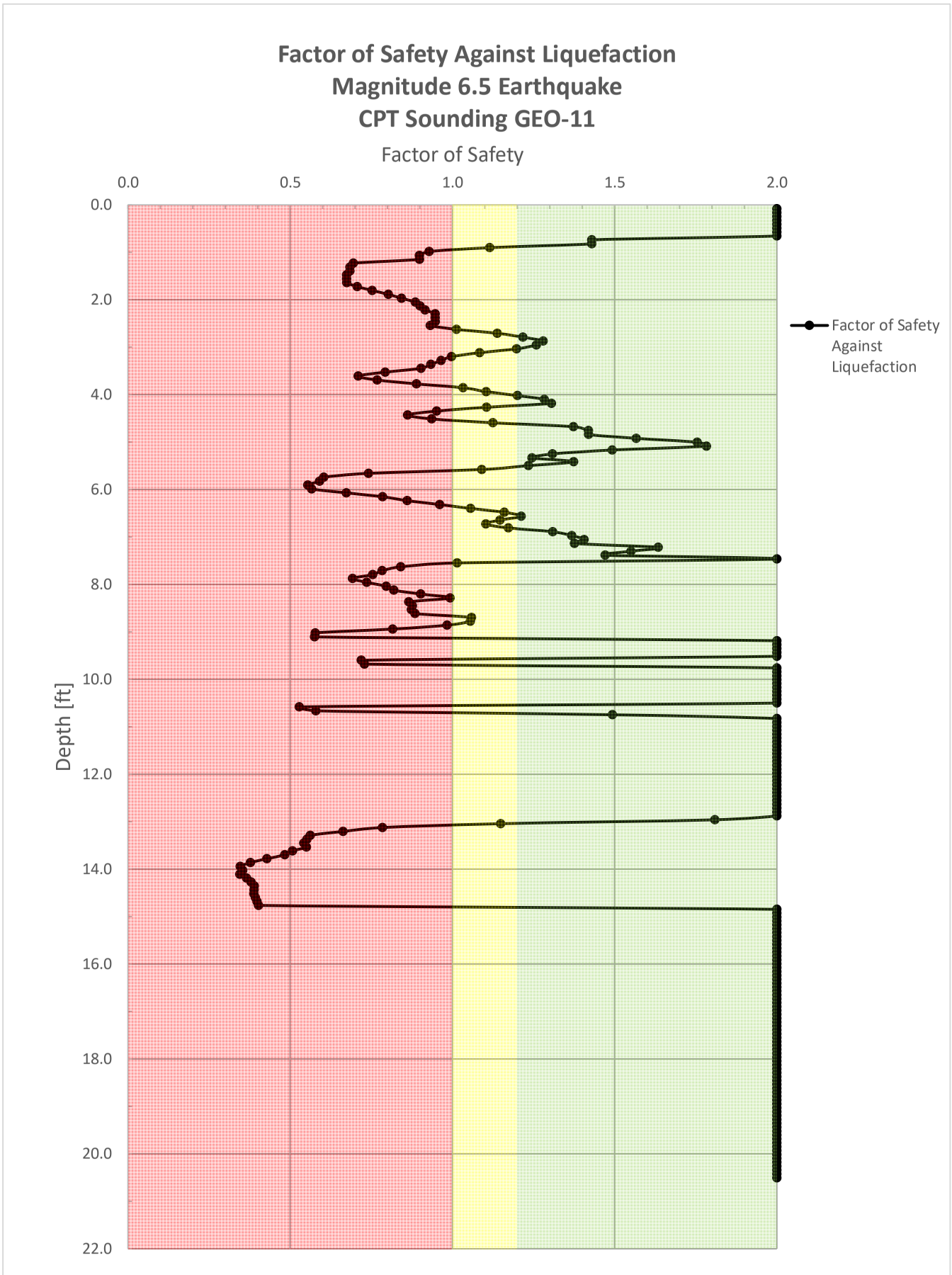

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-06A
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

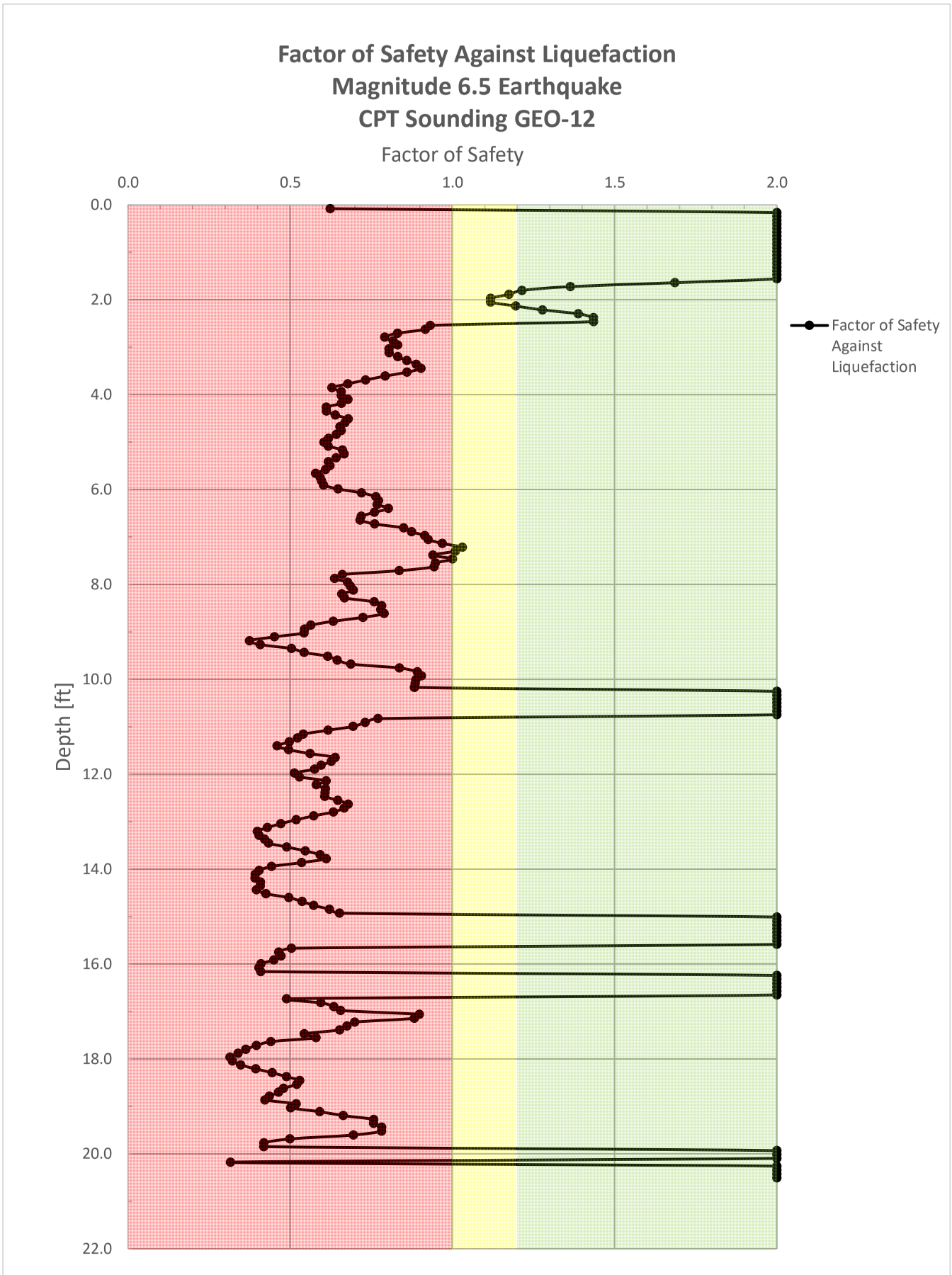

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-07A
Factor of Safety
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1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

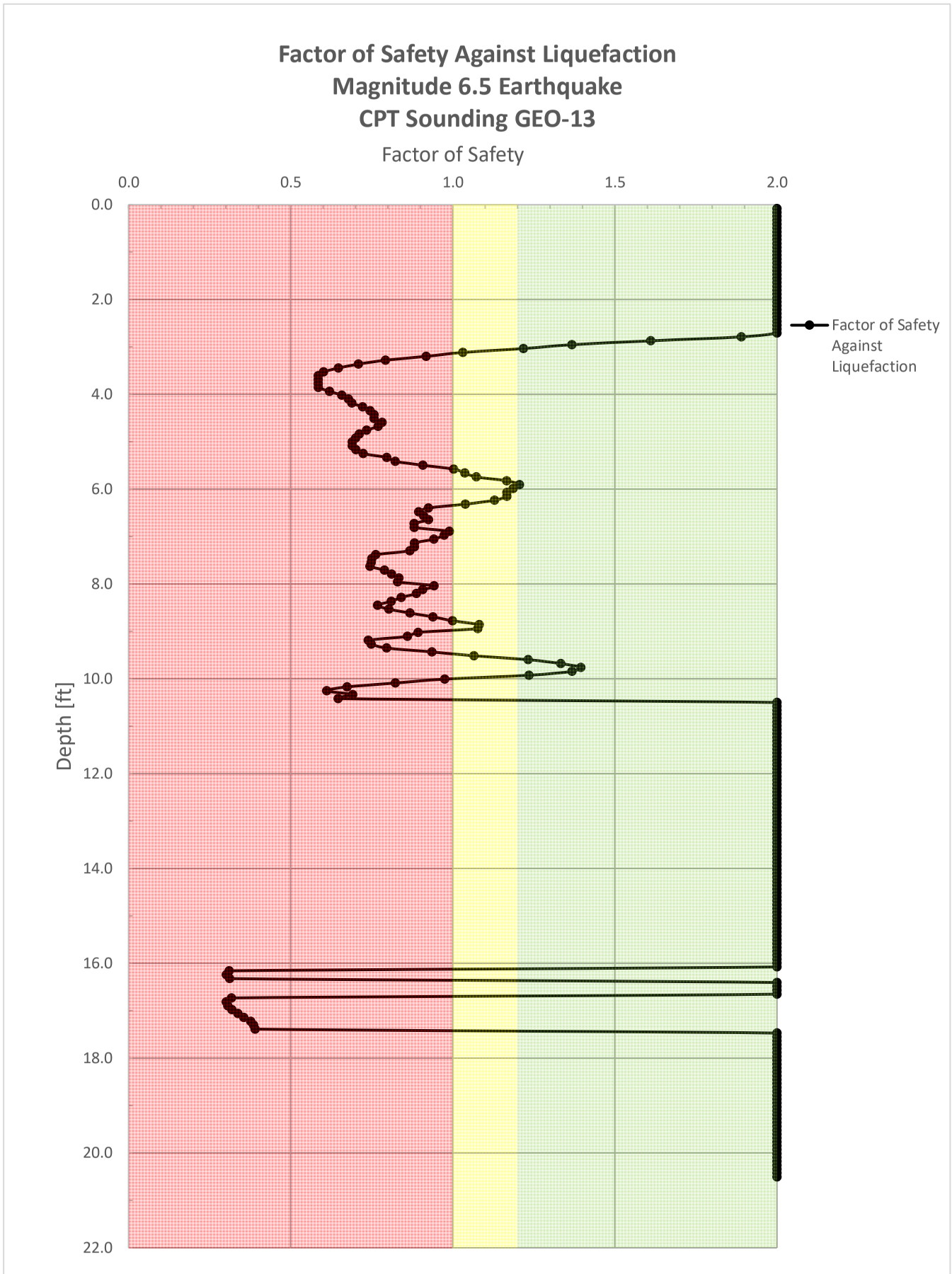

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-08A
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

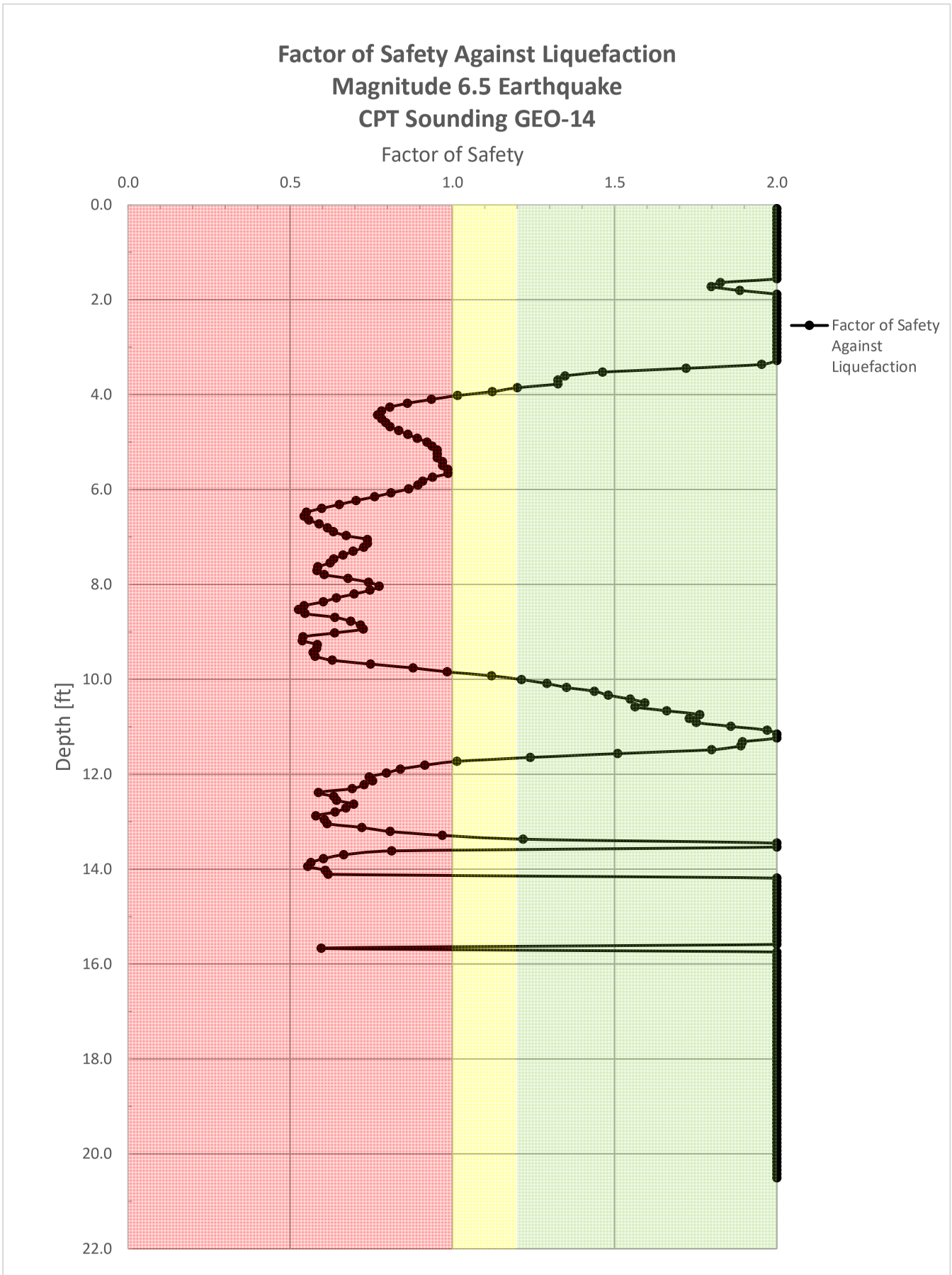

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-09
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

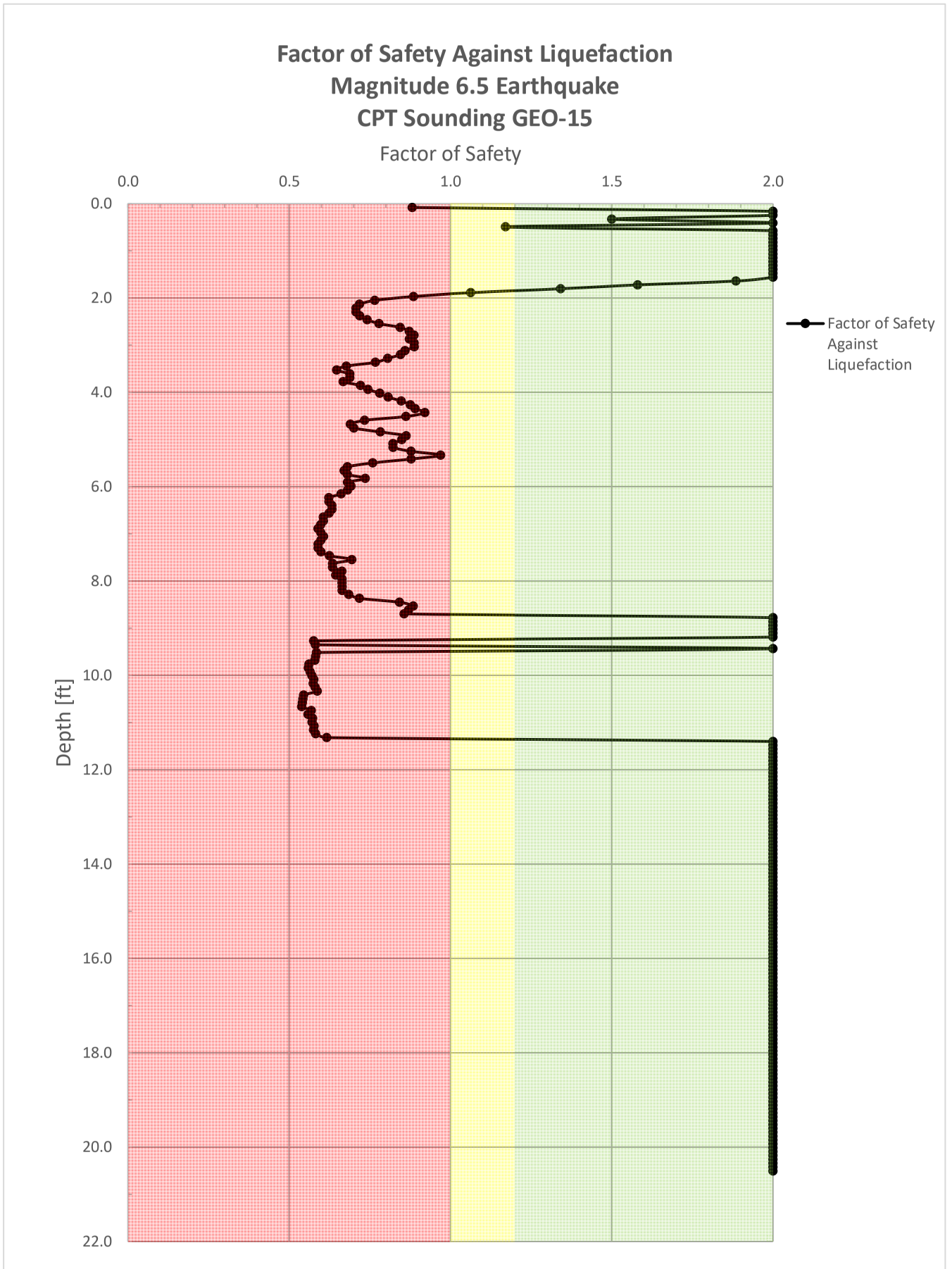

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-10A
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

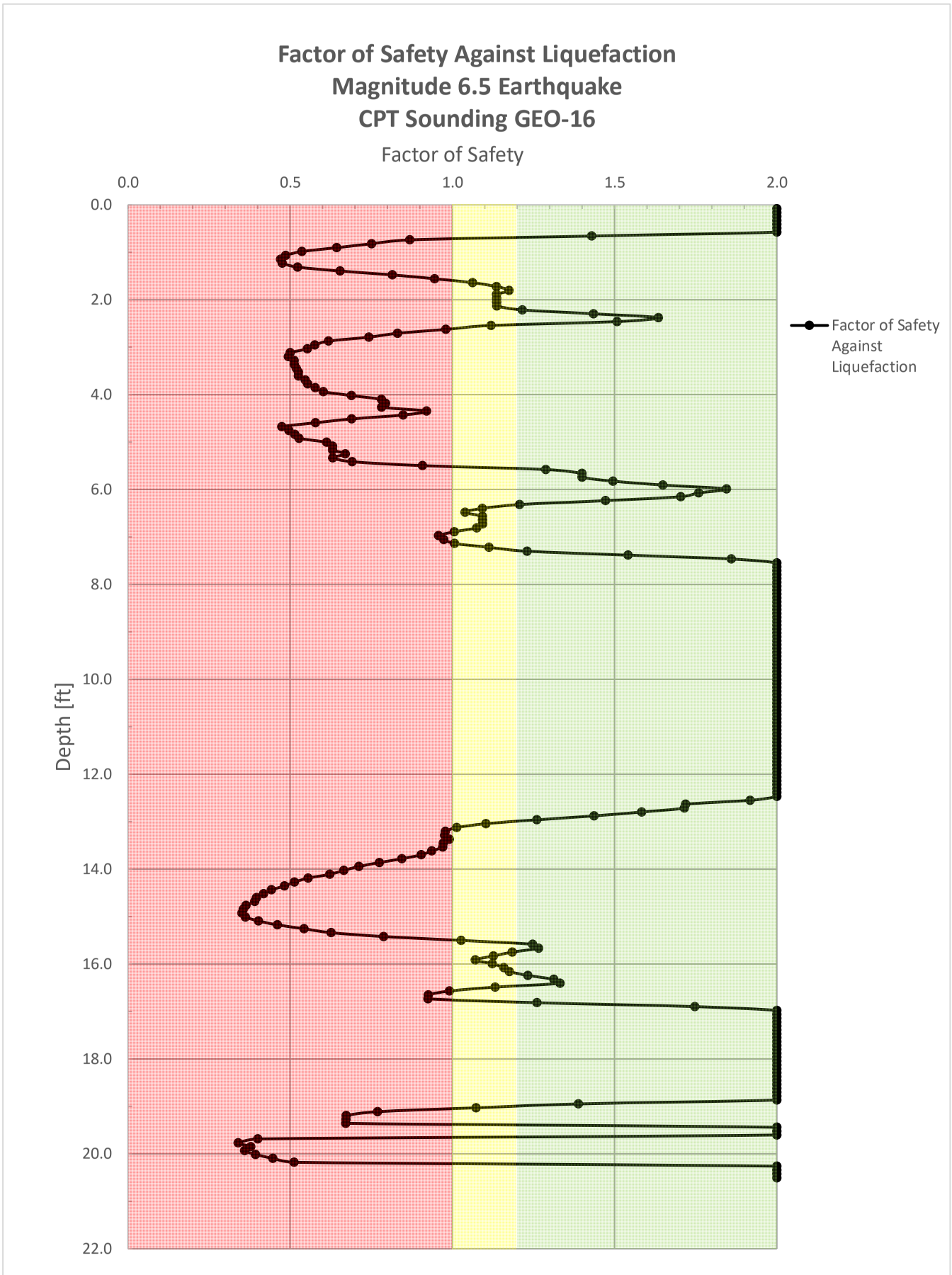

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-11
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

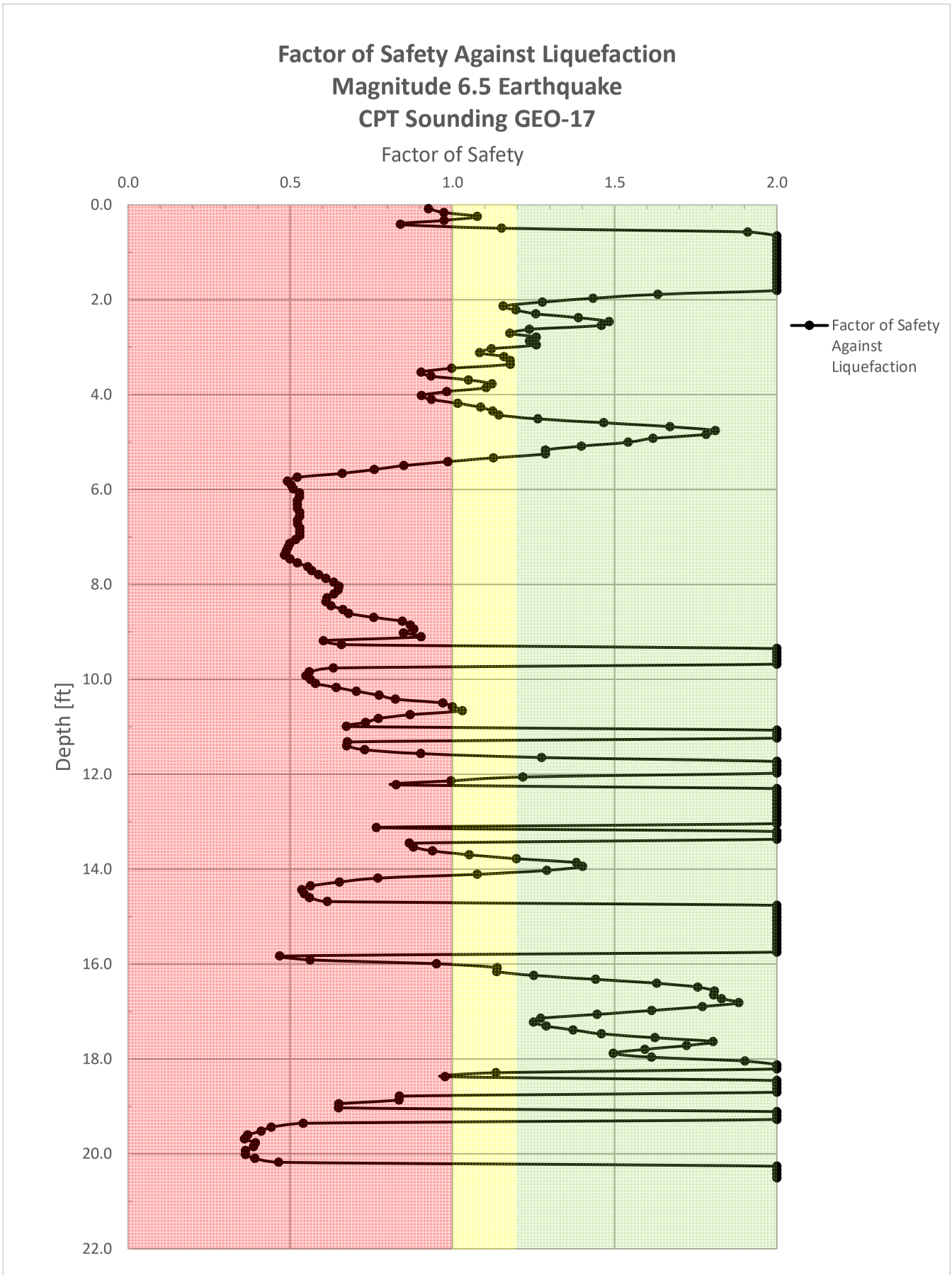

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-12
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

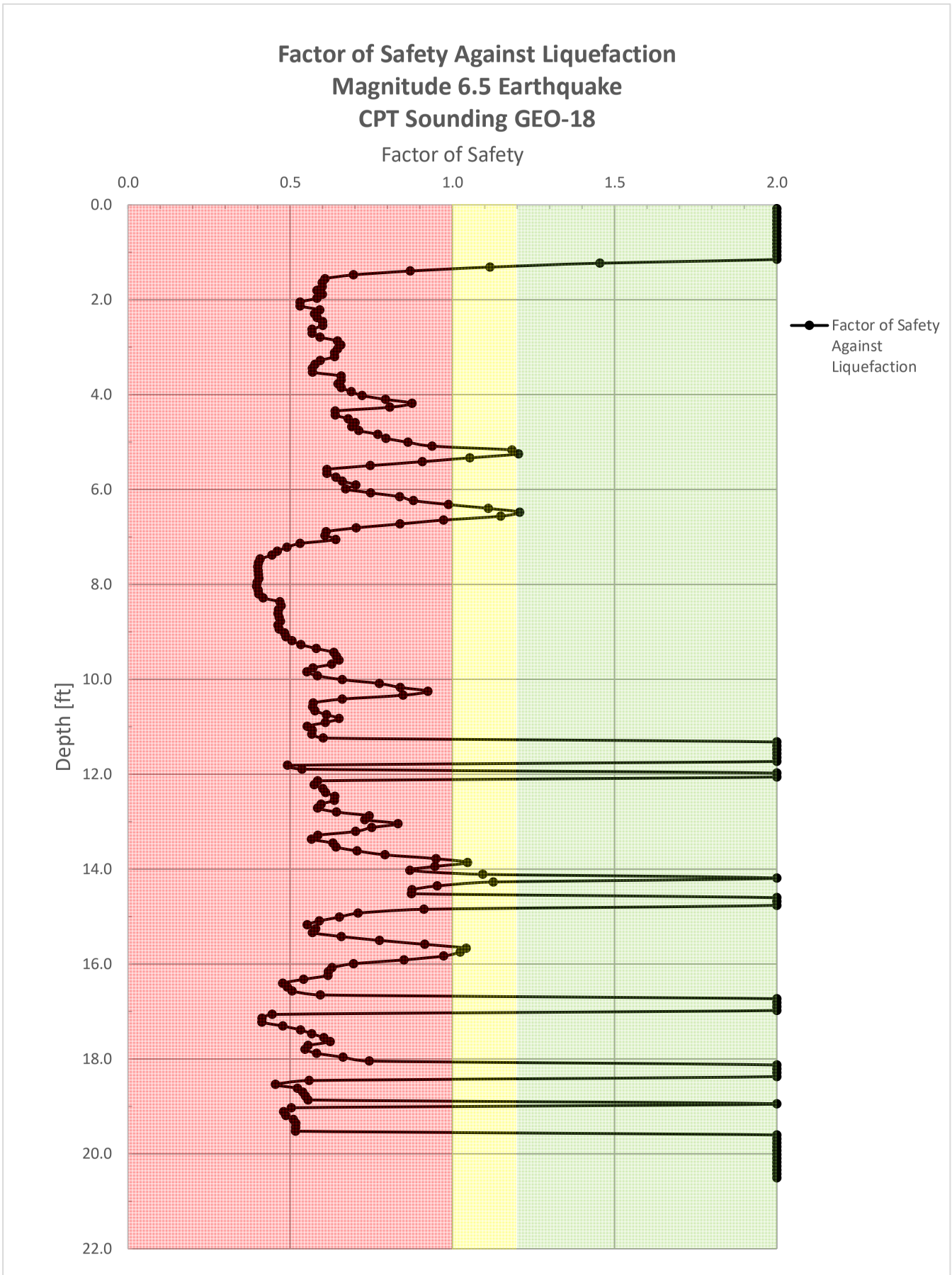

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-13
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

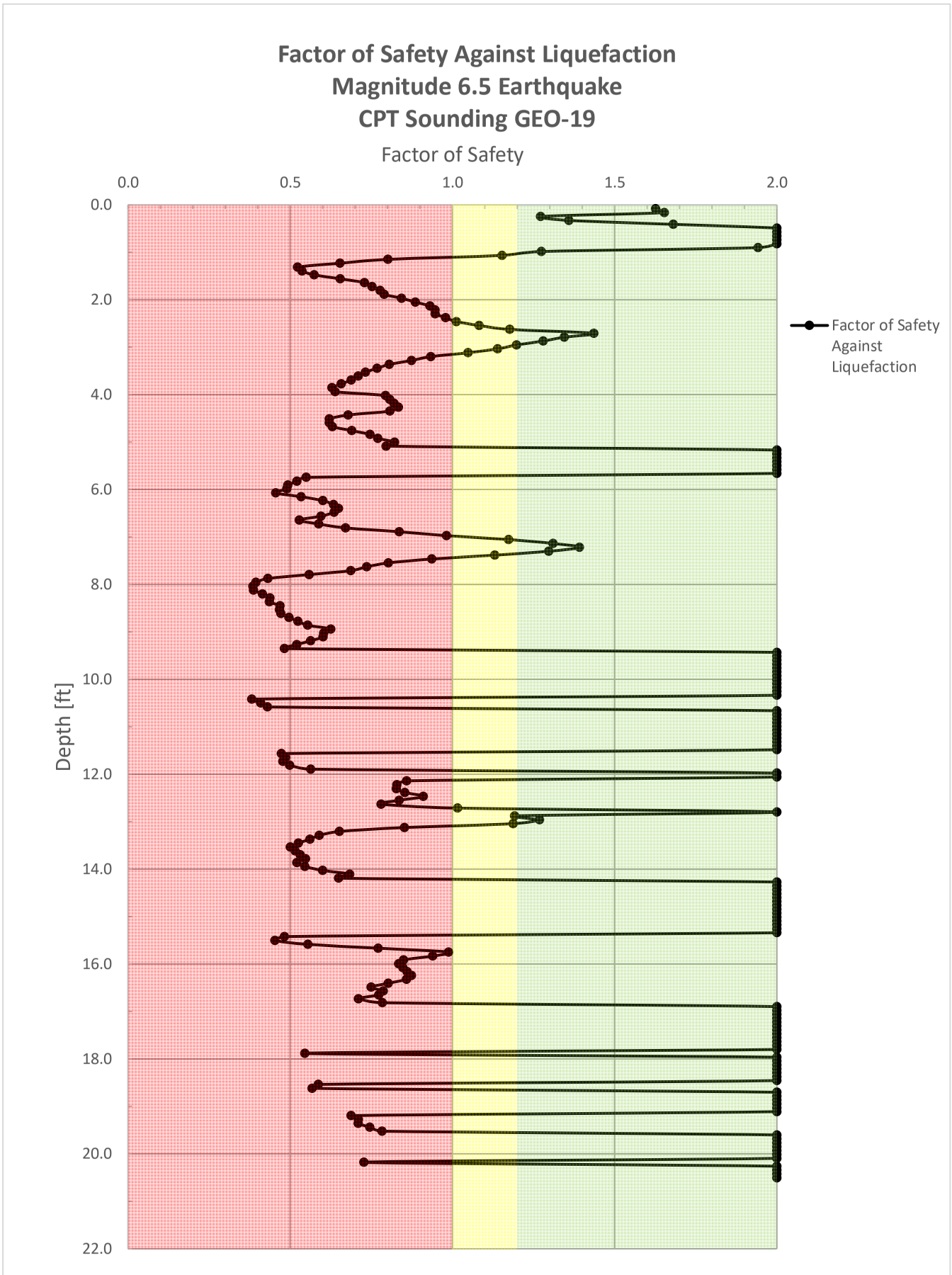

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-14
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

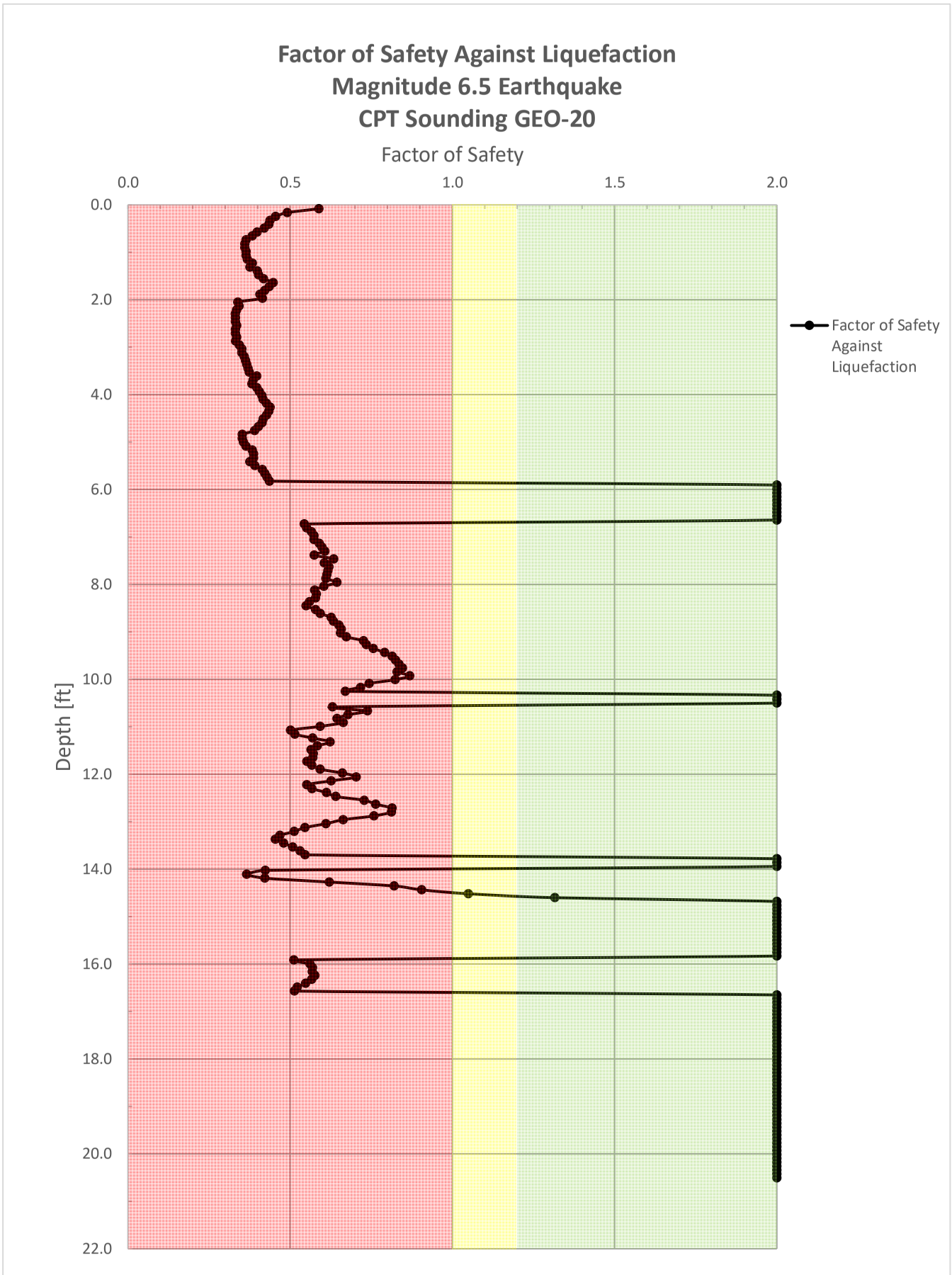

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-15
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

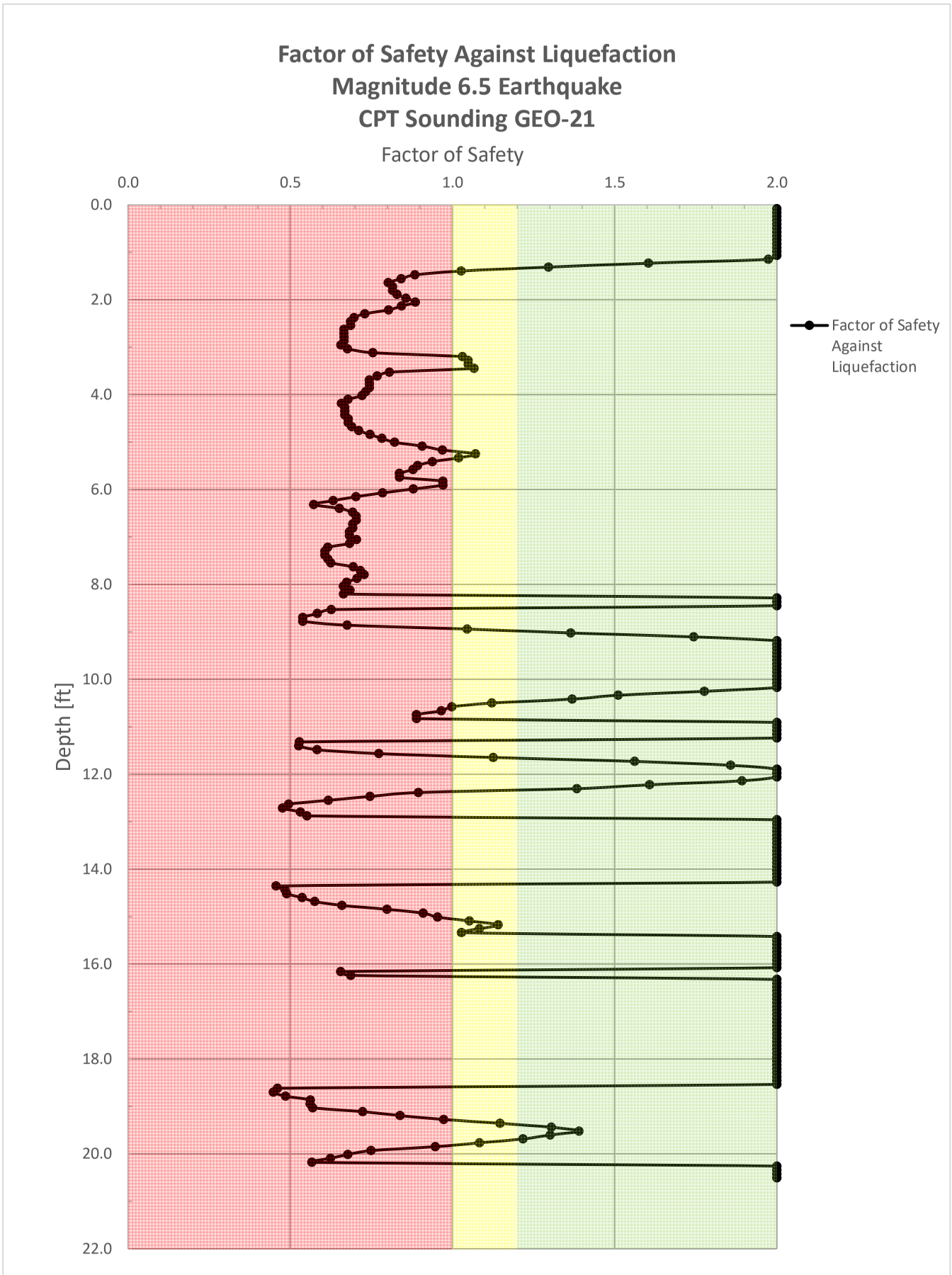

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-16
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

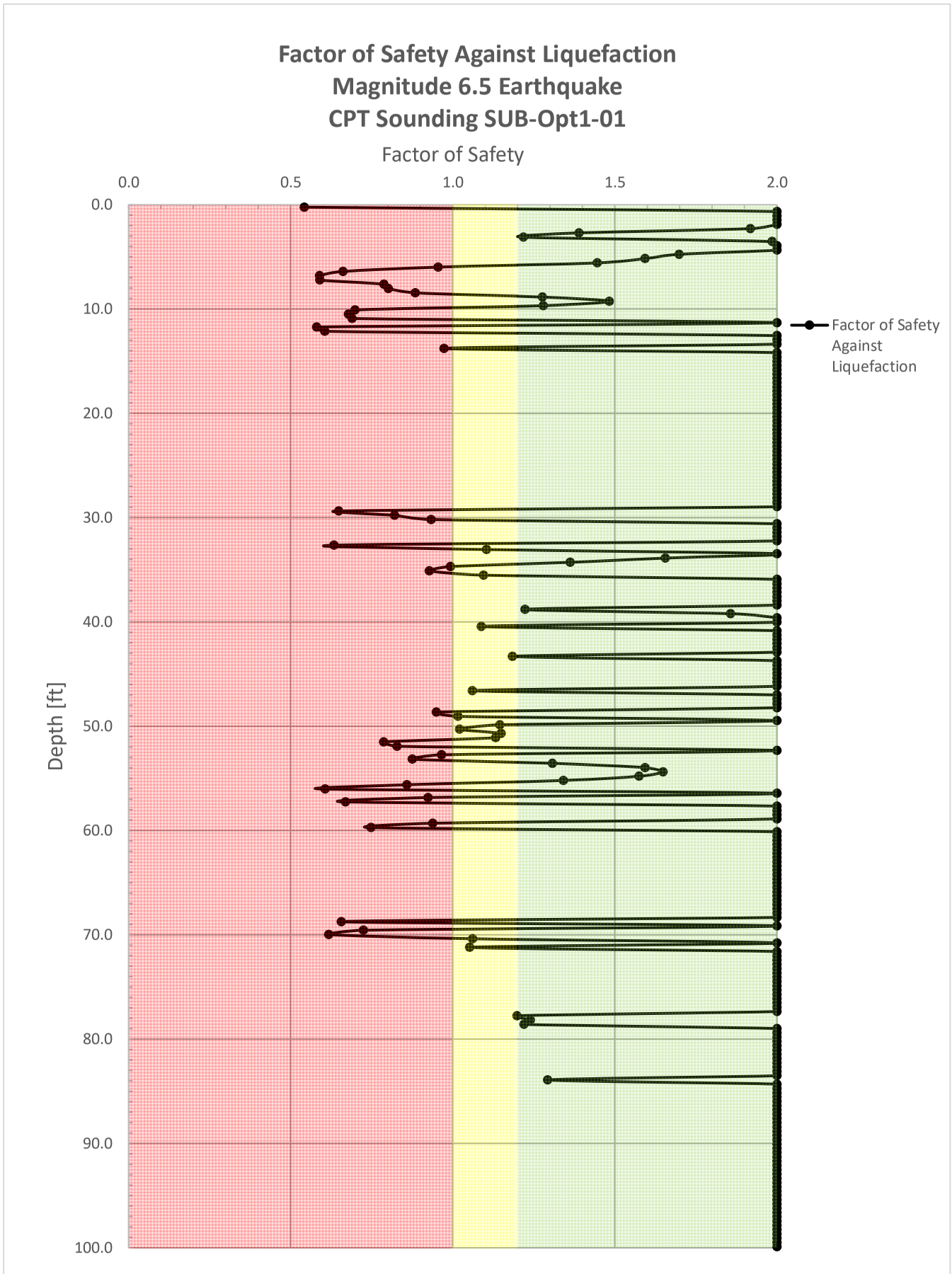

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-17
Factor of Safety
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0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction


Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-18
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction


Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-19
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

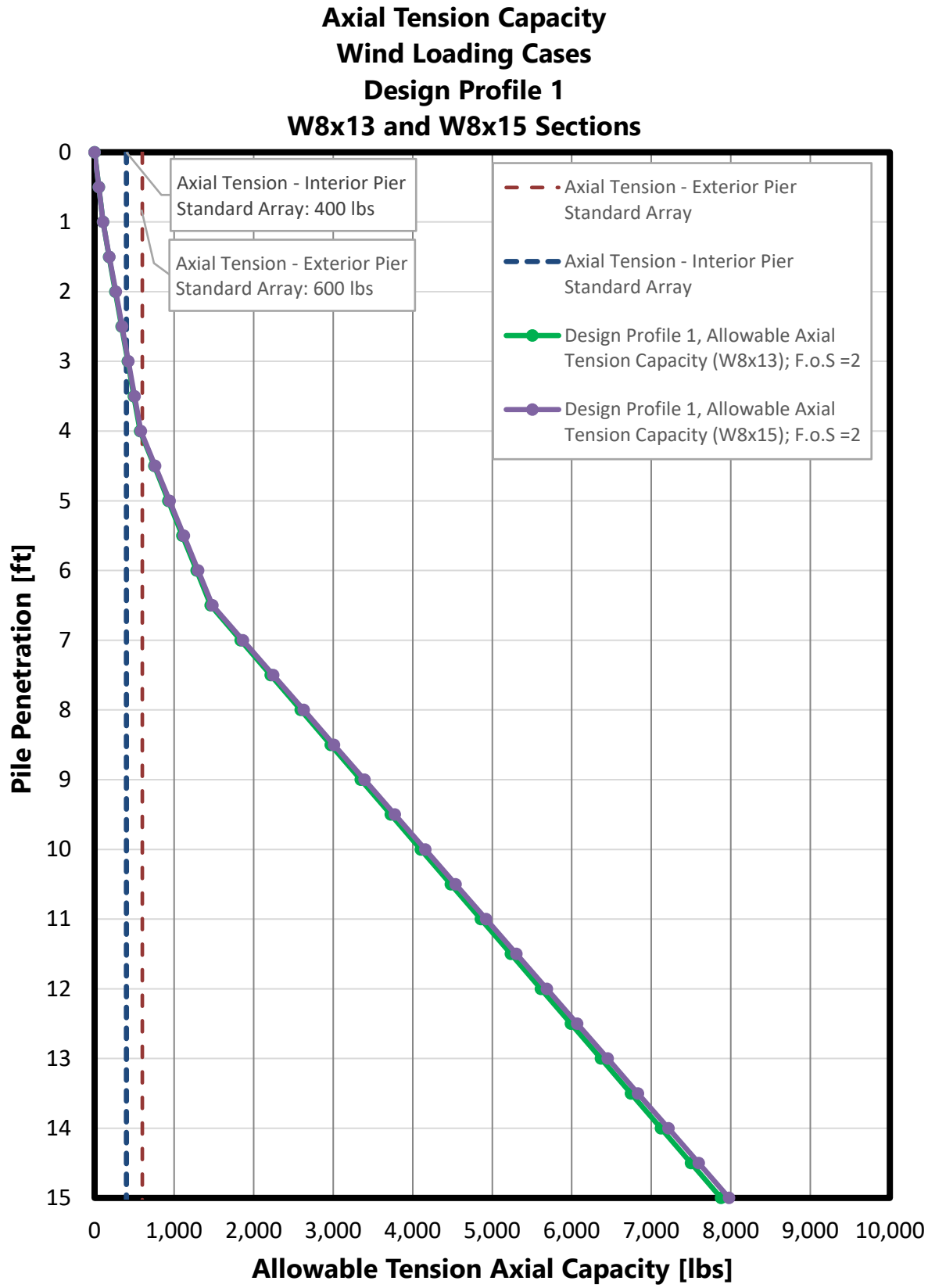

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-20
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

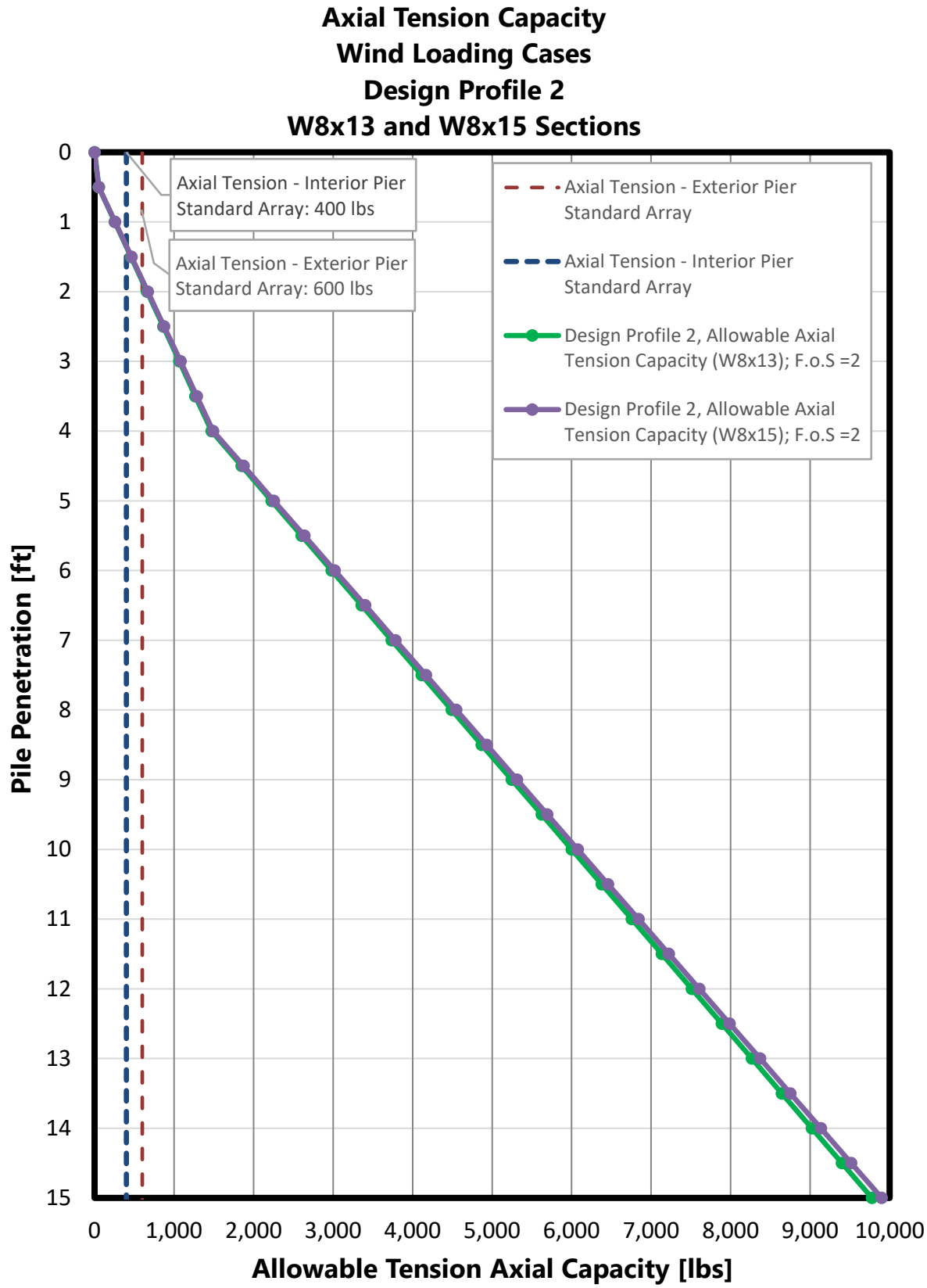

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding GEO-21
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
Factor of Safety Against Liquefaction

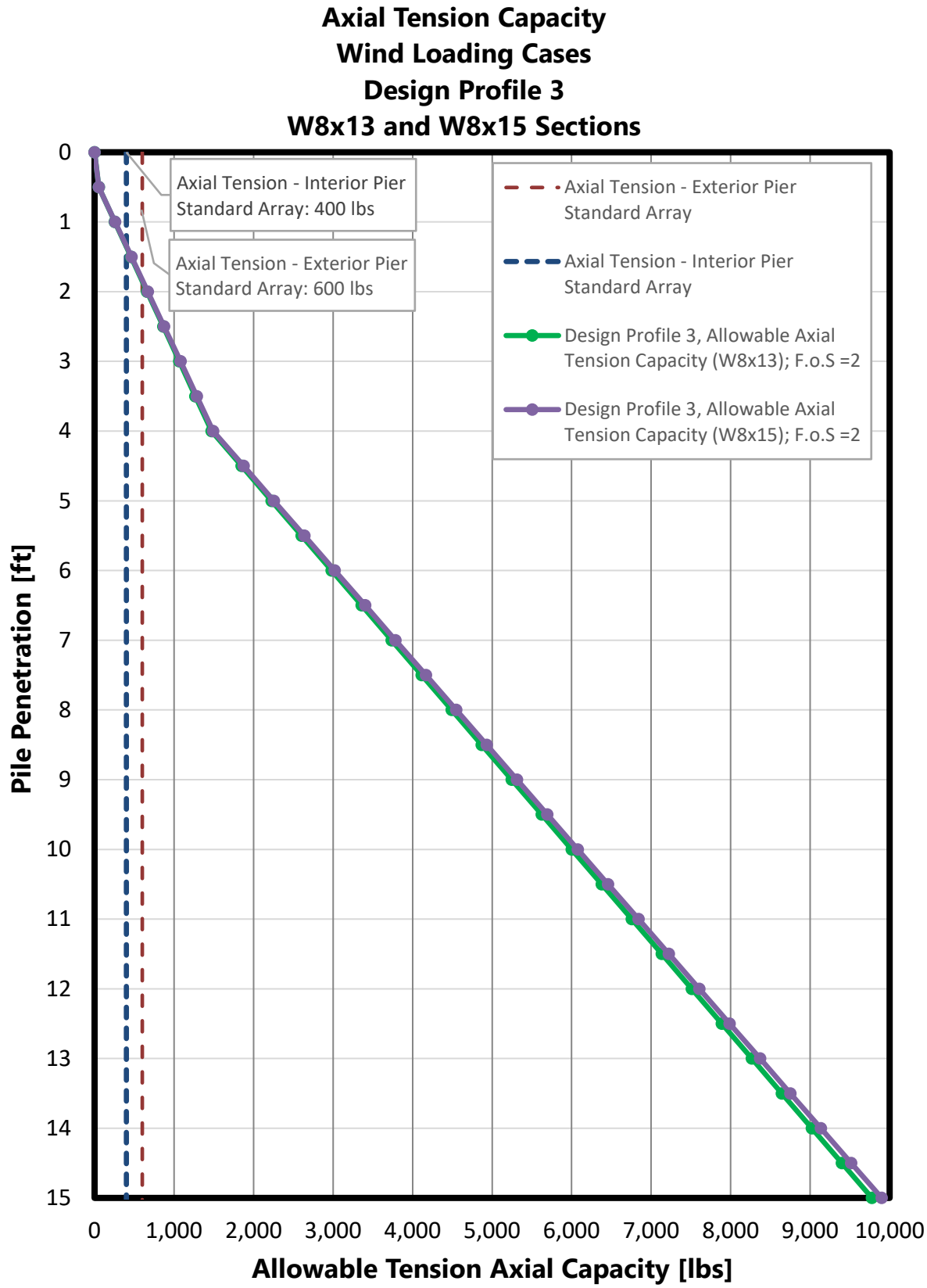

Factor of Safety Against Liquefaction
Magnitude 6.5 Earthquake
CPT Sounding SUB-Opt1-01
Factor of Safety
0.0
0.5
1.0
1.5
2.0
Depth [ft]
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
Factor of Safety Against Liquefaction

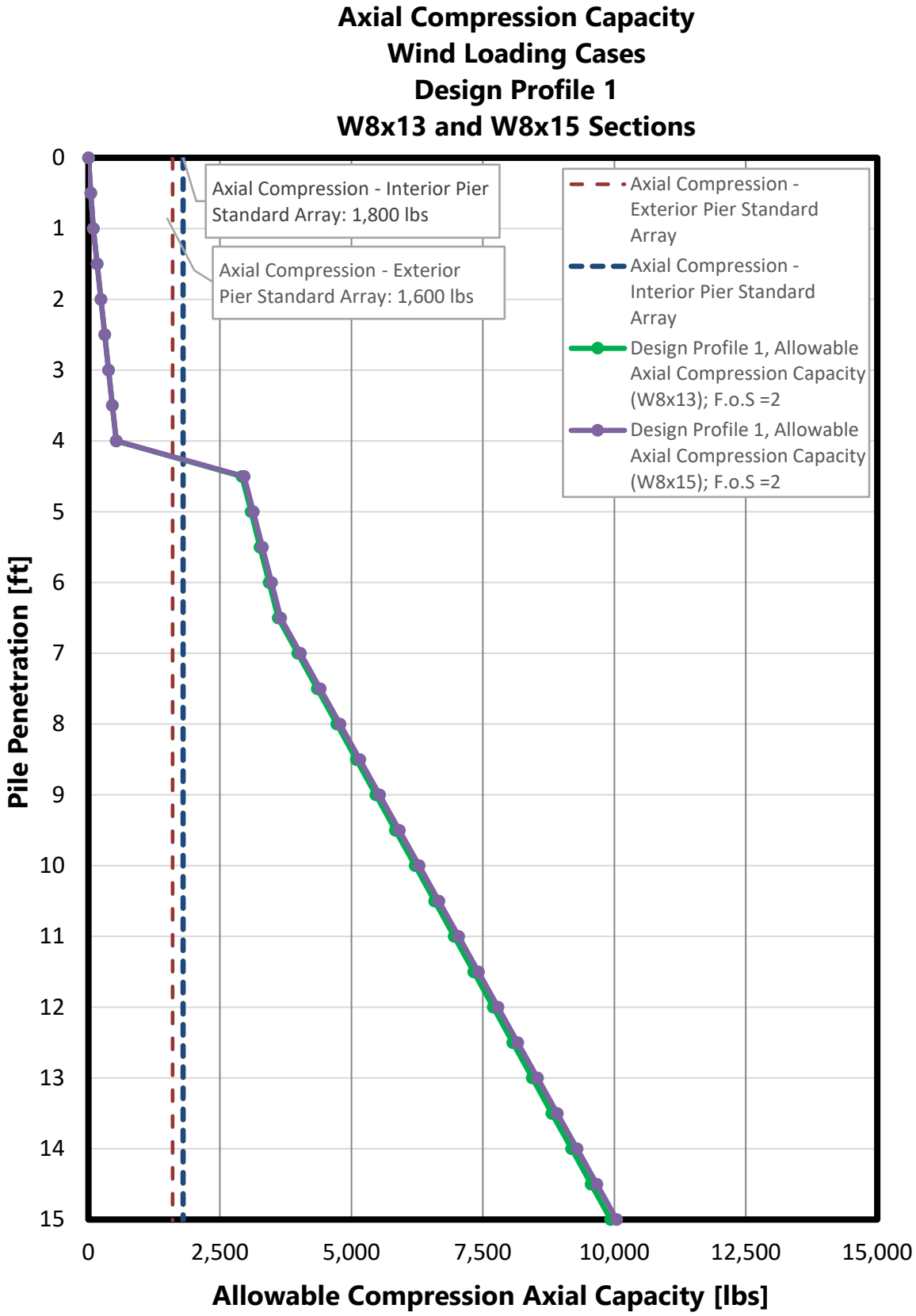
Appendix M

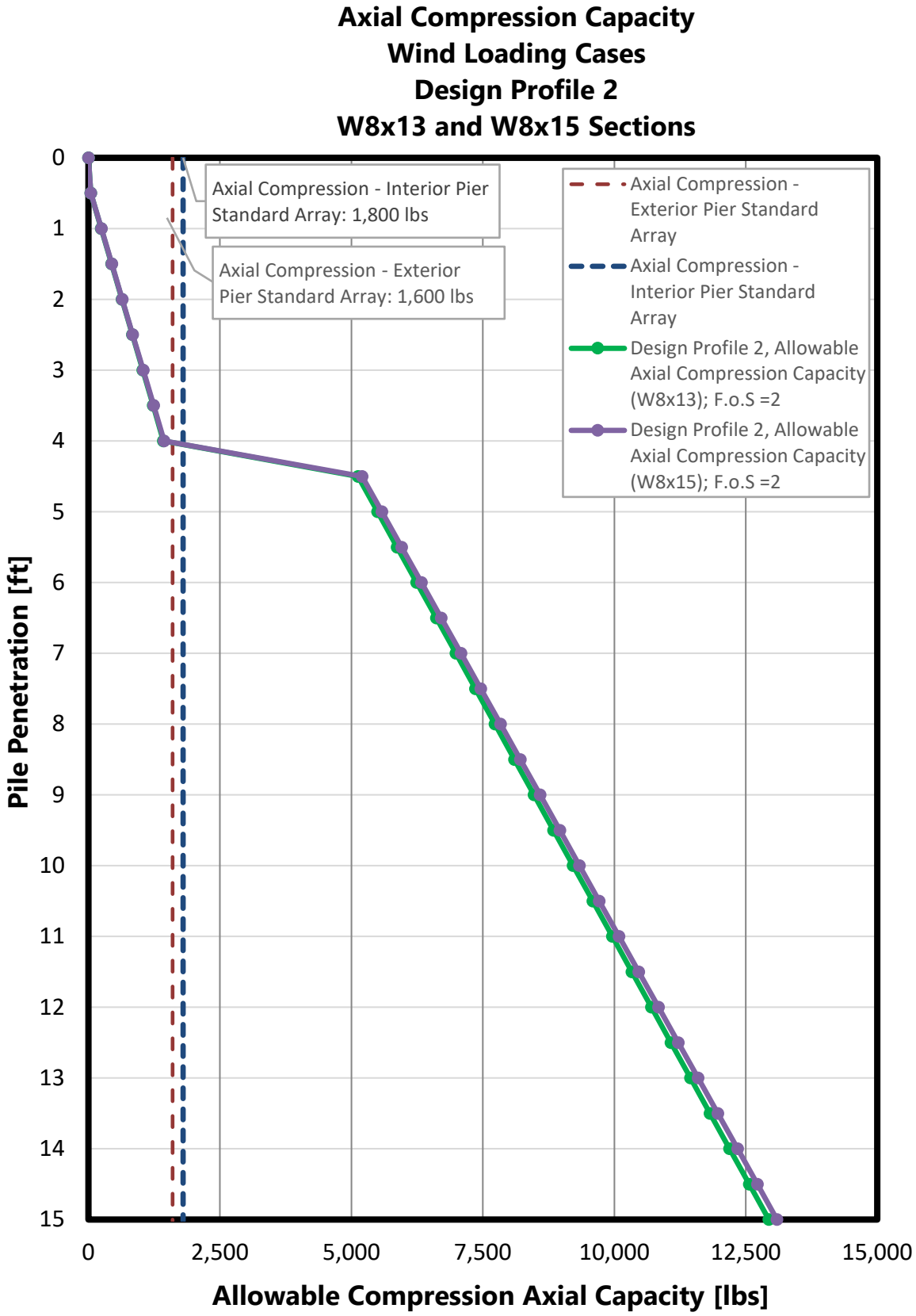
Preliminary Axial Pile Capacity Analysis

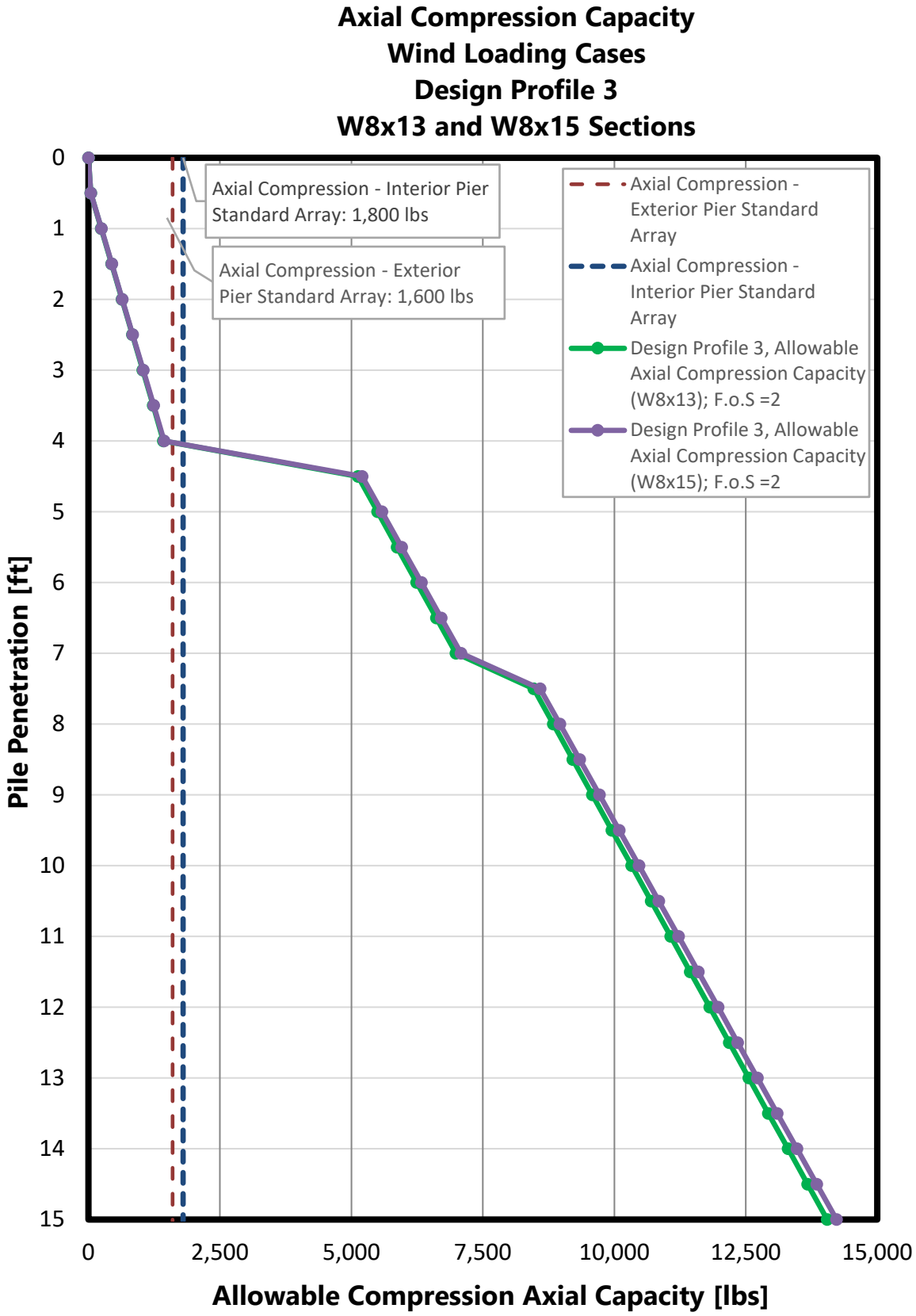

Axial Tension Capacity
Wind Loading Cases
Design Profile 1
W8x13 and W8x15 Sections
Axial Tension - Interior Pier Standard Array: 400 lbs
Axial Tension - Exterior Pier Standard Array: 600 lbs
Axial Tension - Exterior Pier Standard Array
Axial Tension - Interior Pier Standard Array
Design Profile 1, Allowable Axial Tension Capacity (W8x13); F.o.S =2
Design Profile 1, Allowable Axial Tension Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
Allowable Tension Axial Capacity [lbs]


Axial Tension Capacity
Wind Loading Cases
Design Profile 2
W8x13 and W8x15 Sections
Axial Tension - Interior Pier Standard Array: 400 lbs
Axial Tension - Exterior Pier Standard Array: 600 lbs
Axial Tension - Exterior Pier Standard Array
Axial Tension - Interior Pier Standard Array
Design Profile 2, Allowable Axial Tension Capacity (W8x13); F.o.S =2
Design Profile 2, Allowable Axial Tension Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
Allowable Tension Axial Capacity [lbs]


Axial Tension Capacity
Wind Loading Cases
Design Profile 3
W8x13 and W8x15 Sections
Axial Tension - Interior Pier Standard Array: 400 lbs
Axial Tension - Exterior Pier Standard Array: 600 lbs
Axial Tension - Exterior Pier Standard Array
Axial Tension - Interior Pier Standard Array
Design Profile 3, Allowable Axial Tension Capacity (W8x13); F.o.S =2
Design Profile 3, Allowable Axial Tension Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
Allowable Tension Axial Capacity [lbs]


Axial Compression Capacity
Wind Loading Cases
Design Profile 1
W8x13 and W8x15 Sections
Axial Compression - Interior Pier Standard Array: 1,800 lbs
Axial Compression - Exterior Pier Standard Array: 1,600 lbs
Axial Compression - Exterior Pier Standard Array
Axial Compression - Interior Pier Standard Array
Design Profile 1, Allowable Axial Compression Capacity (W8x13); F.o.S =2
Design Profile 1, Allowable Axial Compression Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
2,500
5,000
7,500
10,000
12,500
15,000
Allowable Compression Axial Capacity [lbs]


Axial Compression Capacity
Wind Loading Cases
Design Profile 2
W8x13 and W8x15 Sections
Axial Compression - Interior Pier Standard Array: 1,800 lbs
Axial Compression - Exterior Pier Standard Array: 1,600 lbs
Axial Compression - Exterior Pier Standard Array
Axial Compression - Interior Pier Standard Array
Design Profile 2, Allowable Axial Compression Capacity (W8x13); F.o.S =2
Design Profile 2, Allowable Axial Compression Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
2,500
5,000
7,500
10,000
12,500
15,000
Allowable Compression Axial Capacity [lbs]


Axial Compression Capacity
Wind Loading Cases
Design Profile 3
W8x13 and W8x15 Sections
Axial Compression - Interior Pier Standard Array: 1,800 lbs
Axial Compression - Exterior Pier Standard Array: 1,600 lbs
Axial Compression - Exterior Pier Standard Array
Axial Compression - Interior Pier Standard Array
Design Profile 3, Allowable Axial Compression Capacity (W8x13); F.o.S =2
Design Profile 3, Allowable Axial Compression Capacity (W8x15); F.o.S =2
Pile Penetration [ft]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
0
2,500
5,000
7,500
10,000
12,500
15,000
Allowable Compression Axial Capacity [lbs]

Appendix N

Preliminary Lateral Pile Capacity Analysis

==========

LPile for Windows, Version 2022-12.004

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analysis\LPILE\design profile 1\interior\

Name of the input data file:
rex solaire solar_prelim_design profile 1_interior column_w8x13_9ft embedment depth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 1_interior column_w8x13_9ft embedment depth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 1_interior column_w8x13_9ft embedment depth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 1_interior column_w8x13_9ft embedment depth.lp12r

---

Date and Time of Analysis

Date: October 9, 2024 Time: 14:46:01

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 1, Interior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 14.000 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0000
2	14.000	4.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	14.000000 ft
Flange Width	=	4.000000 in
Section Depth	=	7.990000 in
Flange Thickness	=	0.255000 in
Web Thickness	=	0.230000 in
Section Area	=	3.760400 sq. in
Moment of Inertia	=	38.545891 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	7.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	7.000000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf

```
   Undrained cohesion at top of layer                  =        1500. psf
   Undrained cohesion at bottom of layer               =        1500. psf
   Epsilon-50 at top of layer                          =     0.007000
   Epsilon-50 at bottom of layer                       =     0.007000


Layer 4 is stiff clay without free water

   Distance from top of pile to top of layer           =    10.000000 ft
   Distance from top of pile to bottom of layer        =    25.000000 ft
   Effective unit weight at top of layer               =    42.600000 pcf
   Effective unit weight at bottom of layer            =    42.600000 pcf
   Undrained cohesion at top of layer                  =        1500. psf
   Undrained cohesion at bottom of layer               =        1500. psf
   Epsilon-50 at top of layer                          =     0.007000
   Epsilon-50 at bottom of layer                       =     0.007000


 (Depth of the lowest soil layer extends 11.000 ft below the pile tip)



-------------------------------------------------------------------------------
                              Static Loading Type
-------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



-------------------------------------------------------------------------------
                Pile-head Loading and Pile-head Fixity Conditions
-------------------------------------------------------------------------------

Number of loads specified = 1

Load    Load         Condition              Condition            Axial Thrust
 No.    Type             1                      2                 Force, lbs
-----   ----   --------------------   ----------------------   ---------------
   1     1     V =        2200. lbs   M =       36300. in-lbs            1500.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.
```

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.1064
2	5.5000	0.4144	Yes	No	39.1064	922.5046
3	7.0000	0.5594	No	No	961.6111	9340.
4	10.0000	3.5593	Yes	No	10301.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	2200.0 lbs
Applied moment at pile head	=	36300.0 in-lbs
Axial thrust load on pile head	=	1500.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.72790	36300.	2200.	0.000	1.118E+09
0.14000	1.69329	40048.	2200.	0.000	1.118E+09
0.28000	1.65877	43796.	2200.	0.000	1.118E+09
0.42000	1.62436	47543.	2200.	0.000	1.118E+09
0.56000	1.59008	51291.	2200.	0.000	1.118E+09
0.70000	1.55592	55038.	2200.	0.000	1.118E+09
0.84000	1.52191	58785.	2200.	0.000	1.118E+09
0.98000	1.48804	62532.	2200.	0.000	1.118E+09
1.12000	1.45433	66278.	2200.	0.000	1.118E+09
1.26000	1.42078	70025.	2200.	0.000	1.118E+09
1.40000	1.38742	73771.	2200.	0.000	1.118E+09
1.54000	1.35424	77517.	2200.	0.000	1.118E+09
1.68000	1.32125	81262.	2200.	0.000	1.118E+09
1.82000	1.28847	85007.	2200.	0.000	1.118E+09
1.96000	1.25591	88752.	2200.	0.000	1.118E+09
2.10000	1.22357	92497.	2200.	0.000	1.118E+09
2.24000	1.19146	96241.	2200.	0.000	1.118E+09
2.38000	1.15960	99984.	2200.	0.000	1.118E+09
2.52000	1.12798	103728.	2200.	0.000	1.118E+09
2.66000	1.09663	107471.	2200.	0.000	1.118E+09
2.80000	1.06556	111214.	2200.	0.000	1.118E+09
2.94000	1.03476	114956.	2200.	0.000	1.118E+09
3.08000	1.00425	118697.	2200.	0.000	1.118E+09
3.22000	0.97404	122439.	2200.	0.000	1.118E+09
3.36000	0.94414	126180.	2200.	0.000	1.118E+09
3.50000	0.91456	129920.	2200.	0.000	1.118E+09
3.64000	0.88531	133660.	2200.	0.000	1.118E+09
3.78000	0.85640	137399.	2200.	0.000	1.118E+09
3.92000	0.82783	141138.	2200.	0.000	1.118E+09
4.06000	0.79962	144876.	2200.	0.000	1.118E+09
4.20000	0.77177	148614.	2200.	0.000	1.118E+09
4.34000	0.74430	152351.	2200.	0.000	1.118E+09
4.48000	0.71722	156088.	2200.	0.000	1.118E+09
4.62000	0.69053	159824.	2200.	0.000	1.118E+09
4.76000	0.66424	163559.	2200.	0.000	1.118E+09
4.90000	0.63836	167294.	2200.	0.000	1.118E+09
5.04000	0.61291	171028.	2199.	-0.8836	1.118E+09
5.18000	0.58789	174760.	2195.	-4.5320	1.118E+09
5.32000	0.56331	178477.	2184.	-8.5621	1.118E+09
5.46000	0.53918	182170.	2166.	-12.4994	1.118E+09
5.60000	0.51551	185827.	2138.	-21.3246	1.118E+09
5.74000	0.49232	189423.	2097.	-27.1691	1.118E+09
5.88000	0.46959	192941.	2047.	-32.1910	1.118E+09
6.02000	0.44736	196368.	1989.	-37.1031	1.118E+09
6.16000	0.42562	199689.	1922.	-42.9416	1.118E+09

6.30000	0.40439	202889.	1844.	-49.7515	1.118E+09
6.44000	0.38367	205947.	1754.	-56.7102	1.118E+09
6.58000	0.36346	208845.	1650.	-67.0226	1.118E+09
6.72000	0.34379	211552.	1528.	-78.0650	1.118E+09
6.86000	0.32465	214038.	1387.	-90.4750	1.118E+09
7.00000	0.30605	216269.	1213.	-116.7768	1.118E+09
7.14000	0.28799	218168.	1013.	-121.5230	1.118E+09
7.28000	0.27049	219724.	804.67506	-126.0397	1.118E+09
7.42000	0.25354	220924.	589.33206	-130.3210	1.118E+09
7.56000	0.23715	221755.	366.99894	-134.3612	1.118E+09
7.70000	0.22132	222205.	138.08627	-138.1539	1.118E+09
7.84000	0.20605	222265.	-96.9843	-141.6921	1.118E+09
7.98000	0.19134	221924.	-337.7791	-144.9684	1.118E+09
8.12000	0.17720	221174.	-583.8514	-147.9748	1.118E+09
8.26000	0.16361	220004.	-834.7402	-150.7022	1.118E+09
8.40000	0.15057	218409.	-1090.	-153.1406	1.118E+09
8.54000	0.13809	216380.	-1349.	-155.2784	1.118E+09
8.68000	0.12615	213913.	-1611.	-157.1028	1.118E+09
8.82000	0.11476	211001.	-1877.	-158.5985	1.118E+09
8.96000	0.10389	207640.	-2144.	-159.7477	1.118E+09
9.10000	0.09355	203829.	-2413.	-160.5289	1.118E+09
9.24000	0.08373	199563.	-2683.	-160.9164	1.118E+09
9.38000	0.07441	194842.	-2953.	-160.8780	1.118E+09
9.52000	0.06558	189667.	-3223.	-160.3734	1.118E+09
9.66000	0.05723	184038.	-3492.	-159.3503	1.118E+09
9.80000	0.04935	177958.	-3758.	-157.7400	1.118E+09
9.94000	0.04191	171433.	-4021.	-155.4486	1.118E+09
10.08000	0.03491	164469.	-4280.	-152.3043	1.118E+09
10.22000	0.02832	157074.	-4532.	-148.1253	1.118E+09
10.36000	0.02213	149260.	-4775.	-140.5961	1.118E+09
10.50000	0.01631	141049.	-5002.	-130.2801	1.118E+09
10.64000	0.01086	132470.	-5210.	-117.6675	1.118E+09
10.78000	0.005732	123558.	-5394.	-100.3062	1.118E+09
10.92000	0.000921	114362.	-5531.	-63.5079	1.118E+09
11.06000	-0.003602	104987.	-5509.	89.30184	1.118E+09
11.20000	-0.007860	95863.	-5343.	108.53839	1.118E+09
11.34000	-0.0119	87046.	-5151.	120.33609	1.118E+09
11.48000	-0.0157	78568.	-4942.	128.97671	1.118E+09
11.62000	-0.0193	70453.	-4719.	135.81561	1.118E+09
11.76000	-0.0227	62722.	-4486.	141.47763	1.118E+09
11.90000	-0.0260	55389.	-4245.	146.30875	1.118E+09
12.04000	-0.0291	48470.	-3995.	150.52308	1.118E+09
12.18000	-0.0321	41974.	-3739.	154.26356	1.118E+09
12.32000	-0.0350	35915.	-3477.	157.63088	1.118E+09
12.46000	-0.0378	30300.	-3210.	160.69889	1.118E+09
12.60000	-0.0405	25138.	-2937.	163.52344	1.118E+09
12.74000	-0.0432	20438.	-2661.	166.14771	1.118E+09
12.88000	-0.0458	16207.	-2379.	168.60571	1.118E+09
13.02000	-0.0483	12451.	-2094.	170.92453	1.118E+09
13.16000	-0.0509	9178.	-1805.	173.12589	1.118E+09

13.30000	-0.0534	6394.	-1513.	175.22734	1.118E+09
13.44000	-0.0559	4104.	-1216.	177.24300	1.118E+09
13.58000	-0.0584	2314.	-917.0280	179.18419	1.118E+09
13.72000	-0.0609	1030.	-614.4229	181.05992	1.118E+09
13.86000	-0.0633	256.84637	-308.7157	182.87723	1.118E+09
14.00000	-0.0658	0.000	0.000	184.64147	1.118E+09

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.72790258 inches
Computed slope at pile head      =     -0.0206328 radians
Maximum bending moment           =        222265. inch-lbs
Maximum shear force              =         -5531. lbs
Depth of maximum bending moment  =     7.84000000 feet below pile head
Depth of maximum shear force     =    10.92000000 feet below pile head
Number of iterations             =             45
Number of zero deflection points =              1
Pile deflection at ground        =     0.62018429 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

```
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians
```

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.727903	-0.02063	-5531.	222265.

```
Maximum pile-head deflection = 1.7279025811 inches
Maximum pile-head rotation   = -0.0206327591 radians = -1.182170 deg.
```

The analysis ended normally.

```
================================================================================

                    LPile for Windows, Version 2022-12.004

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
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================================================================================

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--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analys
is\LPILE\design profile 1\exterior\

Name of the input data file:
rex solaire solar_prelim_design profile 1_exterior column_w8x15_10ft embedment d
epth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 1_exterior column_w8x15_10ft embedment d
epth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 1_exterior column_w8x15_10ft embedment d
epth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 1_exterior column_w8x15_10ft embedment d
epth.lp12r
```

Date and Time of Analysis

Date: October 9, 2024 Time: 14:42:19

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 1, Exterior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 15.000 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0150
2	15.000	4.0150

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	15.000000 ft
Flange Width	=	4.015000 in
Section Depth	=	8.110000 in
Flange Thickness	=	0.315000 in
Web Thickness	=	0.245000 in
Section Area	=	4.362050 sq. in
Moment of Inertia	=	46.989100 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	7.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	7.000000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf

```
   Undrained cohesion at top of layer                  =        1500. psf
   Undrained cohesion at bottom of layer               =        1500. psf
   Epsilon-50 at top of layer                          =     0.007000
   Epsilon-50 at bottom of layer                       =     0.007000


Layer 4 is stiff clay without free water

   Distance from top of pile to top of layer           =    10.000000 ft
   Distance from top of pile to bottom of layer        =    25.000000 ft
   Effective unit weight at top of layer               =    42.600000 pcf
   Effective unit weight at bottom of layer            =    42.600000 pcf
   Undrained cohesion at top of layer                  =        1500. psf
   Undrained cohesion at bottom of layer               =        1500. psf
   Epsilon-50 at top of layer                          =     0.007000
   Epsilon-50 at bottom of layer                       =     0.007000


 (Depth of the lowest soil layer extends 10.000 ft below the pile tip)



-------------------------------------------------------------------------------
                              Static Loading Type
-------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



-------------------------------------------------------------------------------
                Pile-head Loading and Pile-head Fixity Conditions
-------------------------------------------------------------------------------

Number of loads specified = 1

Load    Load         Condition               Condition            Axial Thrust
 No.    Type             1                       2                 Force, lbs
-----   ----   --------------------   ----------------------   ----------------
   1     1     V =        2500. lbs   M =       34200. in-lbs             2000.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.
```

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.2444
2	5.5000	0.4144	Yes	No	39.2444	924.2862
3	7.0000	0.5587	No	No	963.5306	9356.
4	10.0000	3.5587	Yes	No	10319.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	2500.0 lbs
Applied moment at pile head	=	34200.0 in-lbs
Axial thrust load on pile head	=	2000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.54827	34200.	2500.	0.000	1.363E+09
0.15000	1.51516	38766.	2500.	0.000	1.363E+09
0.30000	1.48214	43332.	2500.	0.000	1.363E+09
0.45000	1.44922	47898.	2500.	0.000	1.363E+09
0.60000	1.41642	52464.	2500.	0.000	1.363E+09
0.75000	1.38375	57029.	2500.	0.000	1.363E+09
0.90000	1.35121	61594.	2500.	0.000	1.363E+09
1.05000	1.31881	66159.	2500.	0.000	1.363E+09
1.20000	1.28658	70723.	2500.	0.000	1.363E+09
1.35000	1.25451	75288.	2500.	0.000	1.363E+09
1.50000	1.22262	79851.	2500.	0.000	1.363E+09
1.65000	1.19092	84415.	2500.	0.000	1.363E+09
1.80000	1.15942	88978.	2500.	0.000	1.363E+09
1.95000	1.12814	93540.	2500.	0.000	1.363E+09
2.10000	1.09707	98102.	2500.	0.000	1.363E+09
2.25000	1.06624	102664.	2500.	0.000	1.363E+09
2.40000	1.03565	107225.	2500.	0.000	1.363E+09
2.55000	1.00532	111786.	2500.	0.000	1.363E+09
2.70000	0.97525	116346.	2500.	0.000	1.363E+09
2.85000	0.94546	120906.	2500.	0.000	1.363E+09
3.00000	0.91596	125465.	2500.	0.000	1.363E+09
3.15000	0.88676	130023.	2500.	0.000	1.363E+09
3.30000	0.85786	134581.	2500.	0.000	1.363E+09
3.45000	0.82929	139138.	2500.	0.000	1.363E+09
3.60000	0.80104	143694.	2500.	0.000	1.363E+09
3.75000	0.77314	148250.	2500.	0.000	1.363E+09
3.90000	0.74559	152805.	2500.	0.000	1.363E+09
4.05000	0.71840	157360.	2500.	0.000	1.363E+09
4.20000	0.69159	161913.	2500.	0.000	1.363E+09
4.35000	0.66516	166466.	2500.	0.000	1.363E+09
4.50000	0.63913	171018.	2500.	0.000	1.363E+09
4.65000	0.61351	175570.	2500.	0.000	1.363E+09
4.80000	0.58830	180120.	2500.	0.000	1.363E+09
4.95000	0.56352	184669.	2500.	0.000	1.363E+09
5.10000	0.53918	189218.	2498.	-2.3661	1.363E+09
5.25000	0.51529	193758.	2490.	-6.5570	1.363E+09
5.40000	0.49186	198276.	2474.	-10.8978	1.363E+09
5.55000	0.46890	202758.	2447.	-19.2701	1.363E+09
5.70000	0.44642	207176.	2406.	-25.7104	1.363E+09
5.85000	0.42444	211510.	2355.	-31.3710	1.363E+09
6.00000	0.40296	215742.	2294.	-36.5510	1.363E+09
6.15000	0.38199	219854.	2223.	-42.5677	1.363E+09
6.30000	0.36155	223826.	2140.	-49.9109	1.363E+09
6.45000	0.34163	227637.	2043.	-57.4913	1.363E+09
6.60000	0.32226	231259.	1929.	-68.6368	1.363E+09

6.75000	0.30344	234659.	1795.	-80.6171	1.363E+09
6.90000	0.28517	237796.	1638.	-94.3060	1.363E+09
7.05000	0.26748	240626.	1449.	-115.3863	1.363E+09
7.20000	0.25035	243081.	1237.	-120.2207	1.363E+09
7.35000	0.23380	245146.	1016.	-124.7963	1.363E+09
7.50000	0.21784	246805.	787.82216	-129.1067	1.363E+09
7.65000	0.20246	248045.	551.79581	-133.1448	1.363E+09
7.80000	0.18767	248852.	308.75209	-136.9037	1.363E+09
7.95000	0.17347	249214.	59.20061	-140.3757	1.363E+09
8.10000	0.15987	249121.	-196.3348	-143.5525	1.363E+09
8.25000	0.14685	248561.	-457.3150	-146.4255	1.363E+09
8.40000	0.13443	247525.	-723.1844	-148.9849	1.363E+09
8.55000	0.12260	246006.	-993.3690	-151.2203	1.363E+09
8.70000	0.11135	243995.	-1267.	-153.1199	1.363E+09
8.85000	0.10068	241488.	-1544.	-154.6705	1.363E+09
9.00000	0.09059	238478.	-1824.	-155.8571	1.363E+09
9.15000	0.08106	234961.	-2105.	-156.6623	1.363E+09
9.30000	0.07210	230936.	-2387.	-157.0656	1.363E+09
9.45000	0.06368	226401.	-2670.	-157.0425	1.363E+09
9.60000	0.05580	221357.	-2952.	-156.5630	1.363E+09
9.75000	0.04844	215804.	-3233.	-155.5897	1.363E+09
9.90000	0.04160	209745.	-3512.	-154.0746	1.363E+09
10.05000	0.03526	203187.	-3787.	-151.9299	1.363E+09
10.20000	0.02940	196135.	-4058.	-149.0500	1.363E+09
10.35000	0.02401	188600.	-4322.	-143.8927	1.363E+09
10.50000	0.01906	180597.	-4574.	-135.8311	1.363E+09
10.65000	0.01455	172153.	-4810.	-126.9552	1.363E+09
10.80000	0.01044	163298.	-5030.	-116.8546	1.363E+09
10.95000	0.006723	154063.	-5229.	-104.6793	1.363E+09
11.10000	0.003372	144488.	-5402.	-88.0943	1.363E+09
11.25000	0.000364	134626.	-5527.	-50.5230	1.363E+09
11.40000	-0.002324	124601.	-5500.	80.25270	1.363E+09
11.55000	-0.004715	114835.	-5342.	95.78763	1.363E+09
11.70000	-0.006833	105379.	-5161.	105.10031	1.363E+09
11.85000	-0.008701	96263.	-4966.	111.64602	1.363E+09
12.00000	-0.0103	87508.	-4761.	116.56880	1.363E+09
12.15000	-0.0118	79130.	-4547.	120.40817	1.363E+09
12.30000	-0.0130	71142.	-4328.	123.46803	1.363E+09
12.45000	-0.0141	63554.	-4104.	125.93963	1.363E+09
12.60000	-0.0150	56373.	-3875.	127.95293	1.363E+09
12.75000	-0.0158	49607.	-3643.	129.60144	1.363E+09
12.90000	-0.0165	43261.	-3409.	130.95552	1.363E+09
13.05000	-0.0170	37338.	-3172.	132.06999	1.363E+09
13.20000	-0.0175	31844.	-2933.	132.98876	1.363E+09
13.35000	-0.0179	26780.	-2693.	133.74782	1.363E+09
13.50000	-0.0183	22149.	-2452.	134.37720	1.363E+09
13.65000	-0.0185	17953.	-2210.	134.90220	1.363E+09
13.80000	-0.0188	14195.	-1966.	135.34434	1.363E+09
13.95000	-0.0190	10875.	-1723.	135.72197	1.363E+09
14.10000	-0.0192	7995.	-1478.	136.05066	1.363E+09

14.25000	-0.0193	5555.	-1233.	136.34352	1.363E+09
14.40000	-0.0195	3557.	-987.1270	136.61138	1.363E+09
14.55000	-0.0196	2002.	-741.0001	136.86296	1.363E+09
14.70000	-0.0198	890.19022	-494.4290	137.10490	1.363E+09
14.85000	-0.0199	222.60171	-247.4269	137.34193	1.363E+09
15.00000	-0.0201	0.000	0.000	137.57683	1.363E+09

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.54826608 inches
Computed slope at pile head      =     -0.0184168 radians
Maximum bending moment           =        249214. inch-lbs
Maximum shear force              =         -5527. lbs
Depth of maximum bending moment  =     7.95000000 feet below pile head
Depth of maximum shear force     =    11.25000000 feet below pile head
Number of iterations             =             40
Number of zero deflection points =              1
Pile deflection at ground        =     0.55540480 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

```
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians
```

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.548266	-0.01842	-5527.	249214.

```
Maximum pile-head deflection = 1.5482660770 inches
Maximum pile-head rotation   = -0.0184167927 radians = -1.055204 deg.
```

The analysis ended normally.

==

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Files Used for Analysis

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analys
is\LPILE\design profile 2\interior\

Name of the input data file:
rex solaire solar_prelim_design profile 2_interior column_w8x13_8ft embedment de
pth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 2_interior column_w8x13_8ft embedment de
pth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 2_interior column_w8x13_8ft embedment de
pth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 2_interior column_w8x13_8ft embedment de
pth.lp12r

Date and Time of Analysis

Date: October 9, 2024 Time: 14:55:52

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 2, Interior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 13.000 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0000
2	13.000	4.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	13.000000 ft
Flange Width	=	4.000000 in
Section Depth	=	7.990000 in
Flange Thickness	=	0.255000 in
Web Thickness	=	0.230000 in
Section Area	=	3.760400 sq. in
Moment of Inertia	=	38.545891 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	42.600000 pcf
Effective unit weight at bottom of layer	=	42.600000 pcf

```
   Undrained cohesion at top of layer                  =        2000. psf
   Undrained cohesion at bottom of layer               =        2000. psf
   Epsilon-50 at top of layer                          =     0.006000
   Epsilon-50 at bottom of layer                       =     0.006000


 (Depth of the lowest soil layer extends 12.000 ft below the pile tip)



--------------------------------------------------------------------------------
                               Static Loading Type
--------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



--------------------------------------------------------------------------------
                Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 1

Load    Load         Condition               Condition            Axial Thrust
 No.    Type             1                       2                 Force, lbs
-----   ----   --------------------   ----------------------   ----------------
   1     1     V =        2200. lbs   M =       36300. in-lbs             1500.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.



--------------------------------------------------------------------------------
     Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:
-------------------
```

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.1064
2	5.5000	0.4144	Yes	No	39.1064	12627.
3	10.0000	3.3807	No	No	12666.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 2200.0 lbs
Applied moment at pile head = 36300.0 in-lbs
Axial thrust load on pile head = 1500.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.57435	36300.	2200.	0.000	1.118E+09
0.13000	1.54392	39778.	2200.	0.000	1.118E+09
0.26000	1.51357	43255.	2200.	0.000	1.118E+09
0.39000	1.48332	46733.	2200.	0.000	1.118E+09
0.52000	1.45316	50210.	2200.	0.000	1.118E+09
0.65000	1.42312	53687.	2200.	0.000	1.118E+09
0.78000	1.39320	57164.	2200.	0.000	1.118E+09
0.91000	1.36339	60640.	2200.	0.000	1.118E+09

1.04000	1.33372	64117.	2200.	0.000	1.118E+09
1.17000	1.30419	67593.	2200.	0.000	1.118E+09
1.30000	1.27481	71069.	2200.	0.000	1.118E+09
1.43000	1.24558	74545.	2200.	0.000	1.118E+09
1.56000	1.21652	78021.	2200.	0.000	1.118E+09
1.69000	1.18762	81496.	2200.	0.000	1.118E+09
1.82000	1.15890	84971.	2200.	0.000	1.118E+09
1.95000	1.13037	88446.	2200.	0.000	1.118E+09
2.08000	1.10203	91920.	2200.	0.000	1.118E+09
2.21000	1.07389	95395.	2200.	0.000	1.118E+09
2.34000	1.04596	98869.	2200.	0.000	1.118E+09
2.47000	1.01824	102342.	2200.	0.000	1.118E+09
2.60000	0.99074	105815.	2200.	0.000	1.118E+09
2.73000	0.96348	109288.	2200.	0.000	1.118E+09
2.86000	0.93645	112761.	2200.	0.000	1.118E+09
2.99000	0.90967	116233.	2200.	0.000	1.118E+09
3.12000	0.88314	119705.	2200.	0.000	1.118E+09
3.25000	0.85688	123176.	2200.	0.000	1.118E+09
3.38000	0.83088	126647.	2200.	0.000	1.118E+09
3.51000	0.80515	130118.	2200.	0.000	1.118E+09
3.64000	0.77971	133588.	2200.	0.000	1.118E+09
3.77000	0.75456	137058.	2200.	0.000	1.118E+09
3.90000	0.72971	140527.	2200.	0.000	1.118E+09
4.03000	0.70517	143996.	2200.	0.000	1.118E+09
4.16000	0.68094	147464.	2200.	0.000	1.118E+09
4.29000	0.65703	150932.	2200.	0.000	1.118E+09
4.42000	0.63344	154399.	2200.	0.000	1.118E+09
4.55000	0.61020	157866.	2200.	0.000	1.118E+09
4.68000	0.58730	161333.	2200.	0.000	1.118E+09
4.81000	0.56474	164798.	2200.	0.000	1.118E+09
4.94000	0.54255	168264.	2200.	0.000	1.118E+09
5.07000	0.52073	171728.	2199.	-1.6006	1.118E+09
5.20000	0.49927	175189.	2194.	-5.0977	1.118E+09
5.33000	0.47820	178636.	2183.	-8.8493	1.118E+09
5.46000	0.45752	182061.	2166.	-12.4994	1.118E+09
5.59000	0.43724	185455.	2140.	-20.8734	1.118E+09
5.72000	0.41735	188798.	2103.	-26.4022	1.118E+09
5.85000	0.39788	192076.	2058.	-31.2306	1.118E+09
5.98000	0.37883	195277.	2006.	-35.6713	1.118E+09
6.11000	0.36020	198391.	1947.	-40.4510	1.118E+09
6.24000	0.34201	201406.	1878.	-46.8705	1.118E+09
6.37000	0.32425	204306.	1801.	-53.0164	1.118E+09
6.50000	0.30694	207076.	1712.	-61.0386	1.118E+09
6.63000	0.29008	209697.	1609.	-70.8838	1.118E+09
6.76000	0.27367	212145.	1490.	-81.3929	1.118E+09
6.89000	0.25773	214394.	1354.	-93.2922	1.118E+09
7.02000	0.24225	216416.	1198.	-105.9955	1.118E+09
7.15000	0.22725	218179.	1022.	-119.5029	1.118E+09
7.28000	0.21272	219650.	824.74221	-133.8143	1.118E+09
7.41000	0.19867	220795.	604.20192	-148.9297	1.118E+09

7.54000	0.18510	221576.	359.45443	-164.8491	1.118E+09
7.67000	0.17201	221956.	89.24548	-181.5726	1.118E+09
7.80000	0.15940	221893.	-207.6792	-199.1001	1.118E+09
7.93000	0.14728	221345.	-530.1826	-214.3658	1.118E+09
8.06000	0.13564	220275.	-868.1784	-218.9621	1.118E+09
8.19000	0.12448	218671.	-1213.	-222.5830	1.118E+09
8.32000	0.11379	216524.	-1562.	-225.2327	1.118E+09
8.45000	0.10358	213829.	-1915.	-226.9239	1.118E+09
8.58000	0.09383	210581.	-2269.	-227.6770	1.118E+09
8.71000	0.08454	206778.	-2624.	-227.5194	1.118E+09
8.84000	0.07570	202420.	-2978.	-226.4842	1.118E+09
8.97000	0.06730	197511.	-3330.	-224.6092	1.118E+09
9.10000	0.05934	192055.	-3678.	-221.5744	1.118E+09
9.23000	0.05179	186059.	-4020.	-216.8268	1.118E+09
9.36000	0.04464	179534.	-4353.	-210.2090	1.118E+09
9.49000	0.03788	172497.	-4674.	-201.5261	1.118E+09
9.62000	0.03151	164970.	-4980.	-190.5226	1.118E+09
9.75000	0.02549	156978.	-5267.	-176.8451	1.118E+09
9.88000	0.01981	148555.	-5529.	-159.9733	1.118E+09
10.01000	0.01445	139742.	-5780.	-160.6460	1.118E+09
10.14000	0.009402	130538.	-6020.	-147.7478	1.118E+09
10.27000	0.004635	120974.	-6234.	-126.7108	1.118E+09
10.40000	0.000132	111102.	-6374.	-53.0780	1.118E+09
10.53000	-0.004129	101099.	-6316.	128.06259	1.118E+09
10.66000	-0.008170	91408.	-6098.	151.87721	1.118E+09
10.79000	-0.0120	82087.	-5849.	167.23734	1.118E+09
10.92000	-0.0157	73172.	-5579.	178.74325	1.118E+09
11.05000	-0.0192	64692.	-5293.	187.98868	1.118E+09
11.18000	-0.0225	56669.	-4993.	195.73680	1.118E+09
11.31000	-0.0258	49122.	-4683.	202.41907	1.118E+09
11.44000	-0.0289	42068.	-4363.	208.30598	1.118E+09
11.57000	-0.0320	35520.	-4033.	213.57943	1.118E+09
11.70000	-0.0349	29493.	-3697.	218.36811	1.118E+09
11.83000	-0.0378	23996.	-3352.	222.76652	1.118E+09
11.96000	-0.0407	19042.	-3002.	226.84601	1.118E+09
12.09000	-0.0435	14639.	-2645.	230.66162	1.118E+09
12.22000	-0.0462	10798.	-2282.	234.25641	1.118E+09
12.35000	-0.0490	7527.	-1914.	237.66444	1.118E+09
12.48000	-0.0517	4834.	-1541.	240.91279	1.118E+09
12.61000	-0.0545	2728.	-1163.	244.02303	1.118E+09
12.74000	-0.0572	1215.	-779.5937	247.01228	1.118E+09
12.87000	-0.0599	303.38764	-392.0067	249.89403	1.118E+09
13.00000	-0.0626	0.000	0.000	252.67873	1.118E+09

Output Summary for Load Case No. 1:

Pile-head deflection = 1.57435206 inches
Computed slope at pile head = -0.0195346 radians
Maximum bending moment = 221956. inch-lbs
Maximum shear force = -6374. lbs

```
Depth of maximum bending moment  =     7.67000000 feet below pile head
Depth of maximum shear force     =    10.40000000 feet below pile head
Number of iterations             =             36
Number of zero deflection points =              1
Pile deflection at ground        =     0.53247841 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.574352	-0.01953	-6374.	221956.

Maximum pile-head deflection = 1.5743520582 inches
Maximum pile-head rotation   = -0.0195346329 radians = -1.119252 deg.

The analysis ended normally.

```
================================================================================

                    LPile for Windows, Version 2022-12.004

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
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--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analys
is\LPILE\design profile 2\exterior\

Name of the input data file:
rex solaire solar_prelim_design profile 2_exterior column_w8x13_8.5ft embedment
depth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 2_exterior column_w8x13_8.5ft embedment
depth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 2_exterior column_w8x13_8.5ft embedment
depth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 2_exterior column_w8x13_8.5ft embedment
depth.lp12r
```

Date and Time of Analysis

Date: October 9, 2024 Time: 14:52:14

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 2, Exterior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 13.500 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0000
2	13.500	4.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	13.500000 ft
Flange Width	=	4.000000 in
Section Depth	=	7.990000 in
Flange Thickness	=	0.255000 in
Web Thickness	=	0.230000 in
Section Area	=	3.760400 sq. in
Moment of Inertia	=	38.545891 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is stiff clay without free water

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	42.600000 pcf
Effective unit weight at bottom of layer	=	42.600000 pcf

```
   Undrained cohesion at top of layer                  =        2000. psf
   Undrained cohesion at bottom of layer               =        2000. psf
   Epsilon-50 at top of layer                          =     0.006000
   Epsilon-50 at bottom of layer                       =     0.006000


 (Depth of the lowest soil layer extends 11.500 ft below the pile tip)



--------------------------------------------------------------------------------
                               Static Loading Type
--------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



--------------------------------------------------------------------------------
                Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 1

Load    Load         Condition               Condition            Axial Thrust
 No.    Type             1                       2                 Force, lbs
-----   ----   --------------------   ----------------------   ----------------
   1     1     V =        2500. lbs   M =       34200. in-lbs             2000.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.



--------------------------------------------------------------------------------
     Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:
-------------------
```

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.1064
2	5.5000	0.4144	Yes	No	39.1064	12627.
3	10.0000	3.3807	No	No	12666.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head = 2500.0 lbs
Applied moment at pile head = 34200.0 in-lbs
Axial thrust load on pile head = 2000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.70162	34200.	2500.	0.000	1.118E+09
0.13500	1.66745	38318.	2500.	0.000	1.118E+09
0.27000	1.63338	42436.	2500.	0.000	1.118E+09
0.40500	1.59940	46554.	2500.	0.000	1.118E+09
0.54000	1.56553	50672.	2500.	0.000	1.118E+09
0.67500	1.53179	54790.	2500.	0.000	1.118E+09
0.81000	1.49817	58907.	2500.	0.000	1.118E+09
0.94500	1.46469	63024.	2500.	0.000	1.118E+09

1.08000	1.43136	67141.	2500.	0.000	1.118E+09
1.21500	1.39818	71257.	2500.	0.000	1.118E+09
1.35000	1.36517	75373.	2500.	0.000	1.118E+09
1.48500	1.33234	79489.	2500.	0.000	1.118E+09
1.62000	1.29970	83604.	2500.	0.000	1.118E+09
1.75500	1.26725	87719.	2500.	0.000	1.118E+09
1.89000	1.23501	91833.	2500.	0.000	1.118E+09
2.02500	1.20298	95947.	2500.	0.000	1.118E+09
2.16000	1.17118	100061.	2500.	0.000	1.118E+09
2.29500	1.13962	104174.	2500.	0.000	1.118E+09
2.43000	1.10829	108287.	2500.	0.000	1.118E+09
2.56500	1.07723	112399.	2500.	0.000	1.118E+09
2.70000	1.04642	116510.	2500.	0.000	1.118E+09
2.83500	1.01589	120621.	2500.	0.000	1.118E+09
2.97000	0.98565	124732.	2500.	0.000	1.118E+09
3.10500	0.95569	128842.	2500.	0.000	1.118E+09
3.24000	0.92604	132951.	2500.	0.000	1.118E+09
3.37500	0.89670	137060.	2500.	0.000	1.118E+09
3.51000	0.86769	141168.	2500.	0.000	1.118E+09
3.64500	0.83900	145275.	2500.	0.000	1.118E+09
3.78000	0.81066	149382.	2500.	0.000	1.118E+09
3.91500	0.78266	153488.	2500.	0.000	1.118E+09
4.05000	0.75503	157593.	2500.	0.000	1.118E+09
4.18500	0.72776	161698.	2500.	0.000	1.118E+09
4.32000	0.70088	165801.	2500.	0.000	1.118E+09
4.45500	0.67438	169904.	2500.	0.000	1.118E+09
4.59000	0.64829	174007.	2500.	0.000	1.118E+09
4.72500	0.62260	178108.	2500.	0.000	1.118E+09
4.86000	0.59733	182209.	2500.	0.000	1.118E+09
4.99500	0.57249	186308.	2500.	0.000	1.118E+09
5.13000	0.54809	190407.	2497.	-3.1507	1.118E+09
5.26500	0.52413	194497.	2489.	-6.9699	1.118E+09
5.40000	0.50063	198567.	2475.	-10.8585	1.118E+09
5.53500	0.47759	202608.	2451.	-18.5565	1.118E+09
5.67000	0.45504	206600.	2416.	-24.3747	1.118E+09
5.80500	0.43296	210526.	2372.	-29.6539	1.118E+09
5.94000	0.41138	214374.	2321.	-34.1500	1.118E+09
6.07500	0.39031	218131.	2262.	-38.9065	1.118E+09
6.21000	0.36974	221784.	2193.	-45.4071	1.118E+09
6.34500	0.34970	225318.	2115.	-51.8637	1.118E+09
6.48000	0.33019	228714.	2024.	-59.5804	1.118E+09
6.61500	0.31121	231954.	1920.	-69.7157	1.118E+09
6.75000	0.29278	235009.	1798.	-80.5234	1.118E+09
6.88500	0.27490	237851.	1657.	-92.8196	1.118E+09
7.02000	0.25757	240449.	1496.	-105.9955	1.118E+09
7.15500	0.24081	242767.	1313.	-120.0384	1.118E+09
7.29000	0.22463	244770.	1107.	-134.9484	1.118E+09
7.42500	0.20901	246417.	875.34113	-150.7255	1.118E+09
7.56000	0.19398	247667.	617.68412	-167.3696	1.118E+09
7.69500	0.17953	248477.	332.36131	-184.8808	1.118E+09

7.83000	0.16566	248801.	17.96805	-203.2590	1.118E+09
7.96500	0.15237	248590.	-326.9003	-222.5043	1.118E+09
8.10000	0.13967	247794.	-693.1248	-229.6247	1.118E+09
8.23500	0.12755	246394.	-1068.	-232.9254	1.118E+09
8.37000	0.11600	244381.	-1447.	-235.1582	1.118E+09
8.50500	0.10504	241751.	-1829.	-236.3564	1.118E+09
8.64000	0.09464	238498.	-2212.	-236.5642	1.118E+09
8.77500	0.08479	234624.	-2595.	-235.8353	1.118E+09
8.91000	0.07551	230130.	-2975.	-234.2318	1.118E+09
9.04500	0.06676	225020.	-3353.	-231.8232	1.118E+09
9.18000	0.05853	219301.	-3725.	-228.2194	1.118E+09
9.31500	0.05083	212982.	-4091.	-222.8978	1.118E+09
9.45000	0.04362	206076.	-4446.	-215.7471	1.118E+09
9.58500	0.03690	198604.	-4788.	-206.6300	1.118E+09
9.72000	0.03064	190588.	-5114.	-195.3673	1.118E+09
9.85500	0.02483	182059.	-5419.	-181.7117	1.118E+09
9.99000	0.01945	173052.	-5700.	-165.3006	1.118E+09
10.12500	0.01448	163610.	-5967.	-164.1394	1.118E+09
10.26000	0.009886	153737.	-6224.	-152.8621	1.118E+09
10.39500	0.005656	143462.	-6458.	-136.1156	1.118E+09
10.53000	0.001763	132829.	-6652.	-103.4636	1.118E+09
10.66500	-0.001819	121924.	-6651.	104.35617	1.118E+09
10.80000	-0.005114	111292.	-6457.	135.10120	1.118E+09
10.93500	-0.008148	101014.	-6225.	151.77834	1.118E+09
11.07000	-0.0109	91134.	-5970.	163.39552	1.118E+09
11.20500	-0.0135	81683.	-5698.	172.28160	1.118E+09
11.34000	-0.0159	72683.	-5413.	179.43511	1.118E+09
11.47500	-0.0181	64154.	-5118.	185.38846	1.118E+09
11.61000	-0.0202	56111.	-4813.	190.46411	1.118E+09
11.74500	-0.0221	48567.	-4501.	194.87455	1.118E+09
11.88000	-0.0240	41535.	-4182.	198.76847	1.118E+09
12.01500	-0.0257	35024.	-3857.	202.25456	1.118E+09
12.15000	-0.0273	29044.	-3527.	205.41494	1.118E+09
12.28500	-0.0289	23603.	-3192.	208.31317	1.118E+09
12.42000	-0.0304	18708.	-2852.	210.99929	1.118E+09
12.55500	-0.0319	14368.	-2508.	213.51317	1.118E+09
12.69000	-0.0334	10587.	-2161.	215.88672	1.118E+09
12.82500	-0.0348	7373.	-1809.	218.14551	1.118E+09
12.96000	-0.0362	4731.	-1454.	220.30989	1.118E+09
13.09500	-0.0376	2668.	-1095.	222.39587	1.118E+09
13.23000	-0.0389	1188.	-733.4255	224.41572	1.118E+09
13.36500	-0.0403	296.80311	-368.2821	226.37856	1.118E+09
13.50000	-0.0417	0.000	0.000	228.29067	1.118E+09

Output Summary for Load Case No. 1:

Pile-head deflection = 1.70161721 inches
Computed slope at pile head = -0.0211145 radians
Maximum bending moment = 248801. inch-lbs
Maximum shear force = -6652. lbs

```
Depth of maximum bending moment  =     7.83000000 feet below pile head
Depth of maximum shear force     =    10.53000000 feet below pile head
Number of iterations             =             36
Number of zero deflection points =              1
Pile deflection at ground        =     0.57158643 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.701617	-0.02111	-6652.	248801.

Maximum pile-head deflection = 1.7016172054 inches
Maximum pile-head rotation   = -0.0211144528 radians = -1.209769 deg.

The analysis ended normally.

```
================================================================================

                    LPile for Windows, Version 2022-12.004

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
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--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analys
is\LPILE\design profile 3\interior\

Name of the input data file:
rex solaire solar_prelim_design profile 3_interior column_w8x13_7.5ft embedment
depth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 3_interior column_w8x13_7.5ft embedment
depth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 3_interior column_w8x13_7.5ft embedment
depth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 3_interior column_w8x13_7.5ft embedment
depth.lp12r
```

Date and Time of Analysis

Date: October 9, 2024 Time: 15:10:07

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 3, Interior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 12.500 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0000
2	12.500	4.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	12.500000 ft
Flange Width	=	4.000000 in
Section Depth	=	7.990000 in
Flange Thickness	=	0.255000 in
Web Thickness	=	0.230000 in
Section Area	=	3.760400 sq. in
Moment of Inertia	=	38.545891 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	18.500000 ft
Effective unit weight at top of layer	=	42.600000 pcf
Effective unit weight at bottom of layer	=	42.600000 pcf

```
   Friction angle at top of layer                      =    36.000000 deg.
   Friction angle at bottom of layer                   =    36.000000 deg.
   Subgrade k at top of layer                          =   125.000000 pci
   Subgrade k at bottom of layer                       =   125.000000 pci


Layer 4 is stiff clay without free water

   Distance from top of pile to top of layer           =    18.500000 ft
   Distance from top of pile to bottom of layer        =    25.000000 ft
   Effective unit weight at top of layer               =    42.600000 pcf
   Effective unit weight at bottom of layer            =    42.600000 pcf
   Undrained cohesion at top of layer                  =        1500. psf
   Undrained cohesion at bottom of layer               =        1500. psf
   Epsilon-50 at top of layer                          =     0.007000
   Epsilon-50 at bottom of layer                       =     0.007000


 (Depth of the lowest soil layer extends 12.500 ft below the pile tip)



--------------------------------------------------------------------------------
                              Static Loading Type
--------------------------------------------------------------------------------

Static loading criteria were used when computing p-y curves for all analyses.



--------------------------------------------------------------------------------
                Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 1

Load    Load         Condition              Condition            Axial Thrust
 No.    Type             1                      2                 Force, lbs
-----   ----   --------------------   ----------------------   ---------------
   1     1     V =        2200. lbs   M =       36300. in-lbs            1500.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with
specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.
```

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.1064
2	5.5000	0.4144	Yes	No	39.1064	12627.
3	10.0000	4.9143	Yes	No	12666.	23947.
4	18.5000	13.5000	No	No	36613.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	2200.0 lbs
Applied moment at pile head	=	36300.0 in-lbs
Axial thrust load on pile head	=	1500.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.59722	36300.	2200.	0.000	1.118E+09
0.12500	1.56771	39644.	2200.	0.000	1.118E+09
0.25000	1.53828	42988.	2200.	0.000	1.118E+09
0.37500	1.50894	46332.	2200.	0.000	1.118E+09
0.50000	1.47969	49676.	2200.	0.000	1.118E+09
0.62500	1.45055	53020.	2200.	0.000	1.118E+09
0.75000	1.42150	56364.	2200.	0.000	1.118E+09
0.87500	1.39258	59707.	2200.	0.000	1.118E+09
1.00000	1.36377	63050.	2200.	0.000	1.118E+09
1.12500	1.33509	66393.	2200.	0.000	1.118E+09
1.25000	1.30654	69736.	2200.	0.000	1.118E+09
1.37500	1.27814	73079.	2200.	0.000	1.118E+09
1.50000	1.24988	76421.	2200.	0.000	1.118E+09
1.62500	1.22177	79763.	2200.	0.000	1.118E+09
1.75000	1.19383	83105.	2200.	0.000	1.118E+09
1.87500	1.16605	86447.	2200.	0.000	1.118E+09
2.00000	1.13845	89788.	2200.	0.000	1.118E+09
2.12500	1.11102	93129.	2200.	0.000	1.118E+09
2.25000	1.08379	96470.	2200.	0.000	1.118E+09
2.37500	1.05675	99811.	2200.	0.000	1.118E+09
2.50000	1.02991	103151.	2200.	0.000	1.118E+09
2.62500	1.00327	106491.	2200.	0.000	1.118E+09
2.75000	0.97685	109831.	2200.	0.000	1.118E+09
2.87500	0.95066	113170.	2200.	0.000	1.118E+09
3.00000	0.92469	116509.	2200.	0.000	1.118E+09
3.12500	0.89895	119847.	2200.	0.000	1.118E+09
3.25000	0.87346	123186.	2200.	0.000	1.118E+09
3.37500	0.84821	126524.	2200.	0.000	1.118E+09
3.50000	0.82322	129861.	2200.	0.000	1.118E+09
3.62500	0.79849	133198.	2200.	0.000	1.118E+09
3.75000	0.77403	136535.	2200.	0.000	1.118E+09
3.87500	0.74984	139871.	2200.	0.000	1.118E+09
4.00000	0.72594	143207.	2200.	0.000	1.118E+09
4.12500	0.70232	146542.	2200.	0.000	1.118E+09
4.25000	0.67900	149877.	2200.	0.000	1.118E+09
4.37500	0.65598	153212.	2200.	0.000	1.118E+09
4.50000	0.63327	156546.	2200.	0.000	1.118E+09
4.62500	0.61087	159880.	2200.	0.000	1.118E+09
4.75000	0.58879	163213.	2200.	0.000	1.118E+09
4.87500	0.56705	166545.	2200.	0.000	1.118E+09
5.00000	0.54564	169877.	2200.	0.000	1.118E+09
5.12500	0.52457	173209.	2198.	-3.0164	1.118E+09
5.25000	0.50385	176533.	2191.	-6.5351	1.118E+09
5.37500	0.48348	179842.	2178.	-10.1491	1.118E+09
5.50000	0.46348	183128.	2160.	-13.8310	1.118E+09

5.62500	0.44384	186382.	2133.	-22.4374	1.118E+09
5.75000	0.42458	189585.	2095.	-27.5418	1.118E+09
5.87500	0.40570	192725.	2051.	-32.0362	1.118E+09
6.00000	0.38721	195793.	1999.	-36.3990	1.118E+09
6.12500	0.36912	198778.	1941.	-41.2002	1.118E+09
6.25000	0.35142	201670.	1875.	-47.3551	1.118E+09
6.37500	0.33413	204455.	1799.	-53.2449	1.118E+09
6.50000	0.31725	207120.	1714.	-61.0385	1.118E+09
6.62500	0.30079	209646.	1615.	-70.4934	1.118E+09
6.75000	0.28475	212013.	1502.	-80.5233	1.118E+09
6.87500	0.26914	214199.	1372.	-91.8781	1.118E+09
7.00000	0.25396	216177.	1225.	-103.9887	1.118E+09
7.12500	0.23921	217920.	1060.	-116.8428	1.118E+09
7.25000	0.22490	219400.	874.40343	-130.4402	1.118E+09
7.37500	0.21104	220585.	667.98759	-144.7810	1.118E+09
7.50000	0.19761	221445.	439.50304	-159.8651	1.118E+09
7.62500	0.18464	221943.	187.83473	-175.6926	1.118E+09
7.75000	0.17211	222046.	-88.1324	-192.2636	1.118E+09
7.87500	0.16002	221716.	-389.5135	-209.5778	1.118E+09
8.00000	0.14839	220913.	-715.9958	-225.7319	1.118E+09
8.12500	0.13720	219602.	-1058.	-230.0836	1.118E+09
8.25000	0.12645	217773.	-1406.	-233.5148	1.118E+09
8.37500	0.11613	215417.	-1758.	-236.0301	1.118E+09
8.50000	0.10626	212530.	-2113.	-237.6409	1.118E+09
8.62500	0.09681	209107.	-2470.	-238.3650	1.118E+09
8.75000	0.08778	205148.	-2827.	-238.2254	1.118E+09
8.87500	0.07916	200651.	-3184.	-237.2496	1.118E+09
9.00000	0.07095	195621.	-3539.	-235.4687	1.118E+09
9.12500	0.06313	190060.	-3890.	-232.8333	1.118E+09
9.25000	0.05569	183974.	-4236.	-228.7331	1.118E+09
9.37500	0.04863	177374.	-4575.	-222.9254	1.118E+09
9.50000	0.04192	170271.	-4903.	-215.2441	1.118E+09
9.62500	0.03555	162683.	-5219.	-205.4772	1.118E+09
9.75000	0.02951	154633.	-5518.	-193.3412	1.118E+09
9.87500	0.02378	146147.	-5797.	-178.4388	1.118E+09
10.00000	0.01835	137259.	-6034.	-137.6293	1.118E+09
10.12500	0.01319	128061.	-6213.	-101.4247	1.118E+09
10.25000	0.008294	118635.	-6338.	-65.3162	1.118E+09
10.37500	0.003634	109061.	-6409.	-29.2957	1.118E+09
10.50000	-0.000807	99421.	-6426.	6.66146	1.118E+09
10.62500	-0.005048	89796.	-6389.	42.59553	1.118E+09
10.75000	-0.009109	80266.	-6298.	78.56103	1.118E+09
10.87500	-0.0130	70912.	-6153.	114.62534	1.118E+09
11.00000	-0.0168	61817.	-5954.	150.86702	1.118E+09
11.12500	-0.0204	53061.	-5701.	187.37398	1.118E+09
11.25000	-0.0239	44726.	-5392.	224.24146	1.118E+09
11.37500	-0.0274	36895.	-5028.	261.56982	1.118E+09
11.50000	-0.0307	29653.	-4607.	299.22069	1.118E+09
11.62500	-0.0340	23084.	-4142.	321.38537	1.118E+09
11.75000	-0.0373	17238.	-3643.	342.92208	1.118E+09

11.87500	-0.0405	12164.	-3113.	363.98242	1.118E+09
12.00000	-0.0437	7909.	-2552.	384.68557	1.118E+09
12.12500	-0.0469	4519.	-1959.	405.12534	1.118E+09
12.25000	-0.0500	2040.	-1336.	425.37464	1.118E+09
12.37500	-0.0532	518.94757	-683.2463	445.48859	1.118E+09
12.50000	-0.0564	0.000	0.000	465.50654	1.118E+09

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.59721667 inches
Computed slope at pile head      =     -0.0196958 radians
Maximum bending moment           =        222046. inch-lbs
Maximum shear force              =         -6426. lbs
Depth of maximum bending moment  =     7.75000000 feet below pile head
Depth of maximum shear force     =    10.50000000 feet below pile head
Number of iterations             =             19
Number of zero deflection points =              1
Pile deflection at ground        =     0.54563633 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

```
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians
```

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.597217	-0.01970	-6426.	222046.

```
Maximum pile-head deflection = 1.5972166696 inches
Maximum pile-head rotation   = -0.0196958354 radians = -1.128488 deg.
```

The analysis ended normally.

```
================================================================================

                    LPile for Windows, Version 2022-12.004

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
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--------------------------------------------------------------------------------
                             Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations on this computer:
\Mpls\05 CA\16\05161003 Rex Solaire Solar Project\WorkFiles\analysis\pile analys
is\LPILE\design profile 3\exterior\

Name of the input data file:
rex solaire solar_prelim_design profile 3_exterior column_w8x13_8.5ft embedment
depth.lp12d

Name of the output report file:
rex solaire solar_prelim_design profile 3_exterior column_w8x13_8.5ft embedment
depth.lp12o

Name of the plot output file:
rex solaire solar_prelim_design profile 3_exterior column_w8x13_8.5ft embedment
depth.lp12p

Name of the runtime message file:
rex solaire solar_prelim_design profile 3_exterior column_w8x13_8.5ft embedment
depth.lp12r
```

Date and Time of Analysis

Date: October 9, 2024 Time: 15:07:02

Problem Title

Project Name: Rex Solaire Solar Project

Job Number: 05161003.00

Client: Terra-Gen Development Company, LLC

Engineer: PJH3

Description: Design Profile 3, Exterior Column Lateral Analysis

---

Program Options and Settings

Computational Options:
- Conventional Analysis

Engineering Units Used for Data Input and Computations:
- US Customary System Units (pounds, feet, inches)

Analysis Control Options:

- Maximum number of iterations allowed	=	1000
- Deflection tolerance for convergence	=	1.0000E-05 in
- Maximum allowable deflection	=	5.0000 in
- Number of pile increments	=	100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using narrow report formats
   (Note: Some output information is omitted from the narrow report formats)

Pile Structural Properties and Geometry

Number of pile sections defined = 1
Total length of pile = 13.500 ft
Depth of ground surface below top of pile = 5.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over the length of the pile. A summary of values of pile diameter vs. depth follows.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	4.0000
2	13.500	4.0000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile

Cross-sectional Shape	=	Strong H-Pile
Length of section	=	13.500000 ft
Flange Width	=	4.000000 in
Section Depth	=	7.990000 in
Flange Thickness	=	0.255000 in
Web Thickness	=	0.230000 in
Section Area	=	3.760400 sq. in
Moment of Inertia	=	38.545891 in^4
Elastic Modulus	=	29000000. psi

Soil and Rock Layering Information

The soil profile is modelled using 4 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.000000 ft
Distance from top of pile to bottom of layer	=	5.500000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	30.000000 deg.
Friction angle at bottom of layer	=	30.000000 deg.
Subgrade k at top of layer	=	10.000000 pci
Subgrade k at bottom of layer	=	10.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	5.500000 ft
Distance from top of pile to bottom of layer	=	10.000000 ft
Effective unit weight at top of layer	=	100.000000 pcf
Effective unit weight at bottom of layer	=	100.000000 pcf
Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	200.000000 pci
Subgrade k at bottom of layer	=	200.000000 pci

Layer 3 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer	=	10.000000 ft
Distance from top of pile to bottom of layer	=	18.500000 ft
Effective unit weight at top of layer	=	42.600000 pcf
Effective unit weight at bottom of layer	=	42.600000 pcf

Friction angle at top of layer	=	36.000000 deg.
Friction angle at bottom of layer	=	36.000000 deg.
Subgrade k at top of layer	=	125.000000 pci
Subgrade k at bottom of layer	=	125.000000 pci

Layer 4 is stiff clay without free water

Distance from top of pile to top of layer	=	18.500000 ft
Distance from top of pile to bottom of layer	=	25.000000 ft
Effective unit weight at top of layer	=	42.600000 pcf
Effective unit weight at bottom of layer	=	42.600000 pcf
Undrained cohesion at top of layer	=	1500. psf
Undrained cohesion at bottom of layer	=	1500. psf
Epsilon-50 at top of layer	=	0.007000
Epsilon-50 at bottom of layer	=	0.007000

(Depth of the lowest soil layer extends 11.500 ft below the pile tip)

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs
1	1	V = 2500. lbs	M = 34200. in-lbs	2000.

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

---

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	5.0000	0.00	N.A.	No	0.00	39.1064
2	5.5000	0.4144	Yes	No	39.1064	12627.
3	10.0000	4.9143	Yes	No	12666.	35217.
4	18.5000	13.5000	No	No	47883.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Pile-head conditions are Shear and Moment (Loading Type 1)

Shear force at pile head	=	2500.0 lbs
Applied moment at pile head	=	34200.0 in-lbs
Axial thrust load on pile head	=	2000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Soil Res. p lb/inch	Bending Stiffness lb-in^2
0.000	1.66270	34200.	2500.	0.000	1.118E+09
0.13500	1.62904	38317.	2500.	0.000	1.118E+09
0.27000	1.59547	42434.	2500.	0.000	1.118E+09
0.40500	1.56201	46551.	2500.	0.000	1.118E+09
0.54000	1.52865	50668.	2500.	0.000	1.118E+09
0.67500	1.49541	54785.	2500.	0.000	1.118E+09
0.81000	1.46230	58901.	2500.	0.000	1.118E+09
0.94500	1.42933	63017.	2500.	0.000	1.118E+09
1.08000	1.39650	67132.	2500.	0.000	1.118E+09
1.21500	1.36383	71248.	2500.	0.000	1.118E+09
1.35000	1.33133	75363.	2500.	0.000	1.118E+09
1.48500	1.29901	79477.	2500.	0.000	1.118E+09
1.62000	1.26688	83592.	2500.	0.000	1.118E+09
1.75500	1.23494	87706.	2500.	0.000	1.118E+09
1.89000	1.20320	91819.	2500.	0.000	1.118E+09
2.02500	1.17168	95932.	2500.	0.000	1.118E+09
2.16000	1.14039	100045.	2500.	0.000	1.118E+09
2.29500	1.10933	104157.	2500.	0.000	1.118E+09
2.43000	1.07852	108268.	2500.	0.000	1.118E+09
2.56500	1.04796	112379.	2500.	0.000	1.118E+09
2.70000	1.01766	116490.	2500.	0.000	1.118E+09
2.83500	0.98764	120600.	2500.	0.000	1.118E+09
2.97000	0.95790	124710.	2500.	0.000	1.118E+09
3.10500	0.92846	128818.	2500.	0.000	1.118E+09
3.24000	0.89931	132927.	2500.	0.000	1.118E+09
3.37500	0.87048	137034.	2500.	0.000	1.118E+09
3.51000	0.84197	141141.	2500.	0.000	1.118E+09
3.64500	0.81379	145248.	2500.	0.000	1.118E+09
3.78000	0.78595	149353.	2500.	0.000	1.118E+09
3.91500	0.75847	153458.	2500.	0.000	1.118E+09
4.05000	0.73134	157563.	2500.	0.000	1.118E+09
4.18500	0.70458	161666.	2500.	0.000	1.118E+09
4.32000	0.67820	165769.	2500.	0.000	1.118E+09
4.45500	0.65222	169871.	2500.	0.000	1.118E+09
4.59000	0.62663	173972.	2500.	0.000	1.118E+09
4.72500	0.60145	178073.	2500.	0.000	1.118E+09
4.86000	0.57668	182172.	2500.	0.000	1.118E+09
4.99500	0.55235	186271.	2500.	0.000	1.118E+09
5.13000	0.52845	190368.	2497.	-3.1508	1.118E+09
5.26500	0.50500	194457.	2489.	-6.9700	1.118E+09
5.40000	0.48201	198527.	2475.	-10.8586	1.118E+09
5.53500	0.45948	202567.	2451.	-18.5565	1.118E+09
5.67000	0.43742	206557.	2416.	-24.3748	1.118E+09
5.80500	0.41586	210482.	2372.	-29.6540	1.118E+09
5.94000	0.39478	214329.	2321.	-34.1501	1.118E+09

6.07500	0.37421	218085.	2262.	-38.9066	1.118E+09
6.21000	0.35415	221738.	2193.	-45.4073	1.118E+09
6.34500	0.33462	225270.	2115.	-51.8639	1.118E+09
6.48000	0.31561	228666.	2024.	-59.5807	1.118E+09
6.61500	0.29714	231904.	1920.	-69.7160	1.118E+09
6.75000	0.27921	234958.	1798.	-80.5238	1.118E+09
6.88500	0.26183	237799.	1657.	-92.8201	1.118E+09
7.02000	0.24502	240396.	1496.	-105.9960	1.118E+09
7.15500	0.22876	242714.	1313.	-120.0391	1.118E+09
7.29000	0.21308	244715.	1107.	-134.9492	1.118E+09
7.42500	0.19797	246361.	875.33371	-150.7264	1.118E+09
7.56000	0.18344	247610.	617.67507	-167.3707	1.118E+09
7.69500	0.16949	248419.	332.35032	-184.8821	1.118E+09
7.83000	0.15612	248742.	17.95476	-203.2606	1.118E+09
7.96500	0.14334	248530.	-320.6963	-214.8272	1.118E+09
8.10000	0.13114	247753.	-672.0234	-218.9099	1.118E+09
8.23500	0.11953	246400.	-1029.	-221.9722	1.118E+09
8.37000	0.10849	244464.	-1390.	-224.0423	1.118E+09
8.50500	0.09802	241938.	-1754.	-225.1606	1.118E+09
8.64000	0.08813	238821.	-2119.	-225.3787	1.118E+09
8.77500	0.07879	235110.	-2484.	-224.7579	1.118E+09
8.91000	0.07001	230809.	-2847.	-223.3681	1.118E+09
9.04500	0.06177	225921.	-3207.	-221.1323	1.118E+09
9.18000	0.05406	220451.	-3562.	-217.4398	1.118E+09
9.31500	0.04686	214409.	-3910.	-212.1580	1.118E+09
9.45000	0.04017	207810.	-4248.	-205.2085	1.118E+09
9.58500	0.03397	200671.	-4574.	-196.4965	1.118E+09
9.72000	0.02824	193016.	-4883.	-185.8994	1.118E+09
9.85500	0.02296	184872.	-5174.	-173.2494	1.118E+09
9.99000	0.01812	176272.	-5443.	-158.3010	1.118E+09
10.12500	0.01368	167256.	-5656.	-105.2017	1.118E+09
10.26000	0.009647	157963.	-5803.	-76.1156	1.118E+09
10.39500	0.005980	148469.	-5904.	-48.3953	1.118E+09
10.53000	0.002662	138848.	-5961.	-22.0814	1.118E+09
10.66500	-0.000330	129169.	-5977.	2.80633	1.118E+09
10.80000	-0.003019	119496.	-5953.	26.26763	1.118E+09
10.93500	-0.005428	109891.	-5893.	48.32040	1.118E+09
11.07000	-0.007578	100413.	-5798.	68.99954	1.118E+09
11.20500	-0.009493	91115.	-5670.	88.35543	1.118E+09
11.34000	-0.0112	82049.	-5512.	106.45243	1.118E+09
11.47500	-0.0127	73261.	-5326.	123.36733	1.118E+09
11.61000	-0.0140	64798.	-5113.	139.18766	1.118E+09
11.74500	-0.0152	56699.	-4876.	154.00997	1.118E+09
11.88000	-0.0163	49004.	-4615.	167.93807	1.118E+09
12.01500	-0.0172	41749.	-4333.	181.08120	1.118E+09
12.15000	-0.0180	34970.	-4029.	193.55211	1.118E+09
12.28500	-0.0188	28698.	-3706.	205.46520	1.118E+09
12.42000	-0.0195	22966.	-3364.	216.93447	1.118E+09
12.55500	-0.0201	17803.	-3003.	228.07151	1.118E+09
12.69000	-0.0207	13238.	-2625.	238.98338	1.118E+09

12.82500	-0.0213	9300.	-2229.	249.77050	1.118E+09
12.96000	-0.0218	6018.	-1816.	260.52436	1.118E+09
13.09500	-0.0223	3419.	-1385.	271.32527	1.118E+09
13.23000	-0.0229	1533.	-936.5387	282.23996	1.118E+09
13.36500	-0.0234	386.90596	-473.6943	289.17287	1.118E+09
13.50000	-0.0239	0.000	0.000	295.63493	1.118E+09

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.66269949 inches
Computed slope at pile head      =     -0.0208009 radians
Maximum bending moment           =        248742. inch-lbs
Maximum shear force              =         -5977. lbs
Depth of maximum bending moment  =     7.83000000 feet below pile head
Depth of maximum shear force     =    10.66500000 feet below pile head
Number of iterations             =             15
Number of zero deflection points =              1
Pile deflection at ground        =     0.55146353 inches
```

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

```
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians
```

Load Case No.	Load Type	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	1	1.662699	-0.02080	-5977.	248742.

```
Maximum pile-head deflection = 1.6626994908 inches
Maximum pile-head rotation   = -0.0208008695 radians = -1.191802 deg.
```

The analysis ended normally.

Appendix O

Summary of Geotechnical Parameters for Preliminary Substation Foundation Design

Appendix O
Summary of Geotechnical Parameters for Preliminary Substation Foundation Design

Geotechnical Investigation ID	Depth to Top of Layer [ft]	Depth to Bottom of Layer [ft]	Soil Type	Effective Unit Weight [pcf]	Depth to Groundwater [ft]	Friction Angle [deg.][2,4]	Cohesion [psf][3,4]	Soil Elastic Modulus[5] [ksf]	LPILE Soil Type	Drilled Shafts				Drained		Undrained		Drilled Shafts (Drained)		Drilled Shafts (Undrained)		Spread Footing Net Allowable Bearing Capacity (5'x5') [psf][5]	Spread Footing Net Allowable Bearing Capacity (15'x25') [psf][5]
										p-y Modulus, *k* (Static Loading) [lb/in^3][4,6]	p-y Modulus, *k* (Cyclic Loading) [lb/in^3][4,6]	Strain Factor, e50[4,6]	Pressuremeter Modulus, E_{pmt} [ksi][4,5]	Active Earth Pressure Coefficient, K_a[5]	Passive Earth Pressure Coefficient, K_p[5]	Active Earth Pressure Coefficient, K_a[5]	Passive Earth Pressure Coefficient, K_p[5]	Ultimate Skin Friction [kips/ft^2][5]	Ultimate End Bearing [kips/ft^2][5]	Ultimate Skin Friction [kips/ft^2][5]	Ultimate End Bearing [kips/ft^2][5]		
SUB-Opt1-01	0	0.5	Sand (Frost)	100	22	28	0	150	Sand	10	--	N/A	0.2	0.36	2.77	N/A	N/A	0.01	N/A	N/A	N/A	N/A	N/A
	0.5	7.5	Sand	100	22	32	0	510	Sand	75	75	N/A	0.6	0.31	3.25	N/A	N/A	0.08	8.9	N/A	N/A	2,000	1,700
	7.5	10	Clay	100	22	0	2,000	1,000	Stiff Clay	750	300	0.006	1.4	0.38	2.66	1.00	1.00	0.18	10.7	0.80	18.0	N/A	N/A
	10	22	Clay	100	22	0	1,000	500	Stiff Clay	225	90	0.008	0.9	0.38	2.66	1.00	1.00	0.32	19.5	0.40	9.0	N/A	N/A
	22	27	Clay	42.6	22	0	1,250	625	Stiff Clay	250	100	0.008	1.0	0.38	2.66	1.00	1.00	0.46	20.4	0.50	11.3	N/A	N/A
	27	30	Clay	42.6	22	0	1,500	750	Stiff Clay	400	160	0.007	1.1	0.38	2.66	1.00	1.00	0.50	20.4	0.60	13.5	N/A	N/A
	30	50	Clay	42.6	22	0	2,250	1,125	Stiff Clay	800	300	0.006	1.5	0.38	2.66	1.00	1.00	0.60	20.4	0.90	20.3	N/A	N/A
	50	56	Sand	42.6	22	35	0	870	Sand	100	100	N/A	0.9	0.27	3.69	N/A	N/A	0.74	28.0	N/A	N/A	N/A	N/A
	56	65	Clay	42.6	22	0	1,500	750	Stiff Clay	400	160	0.007	1.1	0.38	2.66	1.00	1.00	0.77	20.4	0.60	13.5	N/A	N/A
	65	95	Clay	42.6	22	0	1,750	875	Stiff Clay	500	200	0.007	1.2	0.38	2.66	1.00	1.00	0.94	20.4	0.70	15.8	N/A	N/A
	95	100	Clay	42.6	22	0	1,500	750	Stiff Clay	400	160	0.007	1.1	0.38	2.66	1.00	1.00	1.09	20.4	0.60	13.5	N/A	N/A

1) Soil strength values for lateral resistance of drilled piers should be reduced in the frost zone or zones that may be affected by erosion or scour.

2) Mayne, P.W., 2006. Undisturbed Sand Strength from Seismic Cone Tests, James K. Mitchell Lecture. Georgia Institute of Technology, School of Civil and Environmental Engineering, Atlanta, GA, p 20-21.

3) Lunne, T., P.K. Roberston, and J.J.M. Powell, 1997. Cone Penetration Testing in Geotechnical Practice, Spon Press.

4) Parameter determined from CPT correlations.

5) Das, B. M., Principles of Foundation Engineering, Thomson Canada Limited, 2007 (pages 79-80, 126, 240, 324, 327, 599-604).

6) L-Pile Reference Manual, Ensoft, 2022.

7) For transient, short term loading, the design profile should consist of undrained parameters for the cohesive layers and drained parameters for the granular layers. For static, long-term loading, the design profile should consist of drained parameters for both cohesive and granular layers.

8) Based on the results of the preliminary liquefaction risk assessment presented in Section 5, some of the soils encountered within CPT sounding SUB-Opt1-01 are susceptible to liquefaction. The preliminary liquefaction risk assessment was based on available information and did not consider the impacts of amplification or de-amplification of seismic waves through the soils at the site. Therefore, the recommended geotechnical parameters for use in preliminary substation foundation do not consider the risk of liquefaction. The design considerations related to liquefaction should be further evaluated during later phases of design.