Geotechnical Engineering Report

Islander Solar
North Smithfield, Rhode Island
January 21, 2021
Terracon Project No. J2205052

Prepared for:
Islander Solar LLC
Summit, NJ

Prepared by:
Terracon Consultants, Inc.
Rocky Hill, Connecticut

January 21, 2021

Islander Solar LLC
396 Springfield Avenue, 2nd Floor
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Attn: Mr. Scott K. Risley – Director of Development
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Re: Geotechnical Engineering Report
Islander Solar
Iron Mine Hill Road (AP 16 Lot 19)
North Smithfield, Rhode Island
Terracon Project No. J2205052

Dear Mr. Risley:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PJ2205052 dated October 19, 2020. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of the proposed solar facility.

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

Sincerely,
Terracon Consultants, Inc.

CARL W. THUNBERG
STATE OF RHODE ISLAND
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HOPE
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REPORT TOPICS

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

ATTACHMENTS

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

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INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed solar facility to be located south of Iron Mine Hill Road (AP 16 Lot 19) in North Smithfield, Rhode Island. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- L-Pile parameters
- Seismic site classification per IBC
- Foundation design and construction
- Laboratory test results
- Unpaved road design and construction
- Frost considerations
- Estimated settlement (shallow foundations)

The geotechnical engineering Scope of Services for this project included the advancement of five (5) test borings to depths ranging from approximately 2.5 to 6.9 feet below existing site grades.

Maps showing the site and boring locations are shown in **Site Location** and **Exploration Plan**. Boring logs and the results of laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs or as separate reports in **Exploration Results**.

SITE CONDITIONS

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

Item	Description
Parcel Information	The project site is located on the Town of North Smithfield Assessor's Plat 16 Lot 19 in North Smithfield, Rhode Island. The site is approximately 22 acres±. The proposed solar array covers approximately 8.2 acres. The approximate center coordinates of the site are 41.9528° N 71.5207° W. (See **Site Location**)

Item	Description
Existing Improvements	The site is located primarily on undeveloped dense-wooded land behind private property located at 850 Iron Mine Hill Road, North Smithfield, Rhode Island.
Current Ground Cover	Moderately- to heavily-wooded through most of the site.
Existing Topography	A Concept Plan, Drawing No. C-1, dated 7/24/2020 was provided. Ground surface grades within the site generally slope from approximately El 420± feet along the perimeter of the site to El 460± feet at the center. (See **Site Location**)
Geology	Our experience near the vicinity of the proposed development or geologic maps indicates subsurface conditions consist of glacial till overlying medium-to coarse-grained rock (Esmond Granite). According to US Department of Agriculture, the surficial soils consist of fine sandy loam. The wetland portions of the site are mapped as muck. The wetland is located in southwestern portion of the property. However, the wetland is not within the proposed perimeter of the solar array. (See **Exploration Plan**)

PROJECT DESCRIPTION

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

Item	Description
Information Provided	The following documents were provided: ■ Concept Plan, Drawing No. C-1, 07/24/2020
Project Description	The project consists of the design of a utility-scale photovoltaic power plant project producing 3.3 MW DC. The area of the proposed solar array is about 8.2 acres. The location of the substation is unknown at this time.
Proposed Structures	We anticipate the proposed solar PV arrays will be mounted on single-axis tracker racking systems and the foundations will be supported on driven steel posts (W-sections). Electrical equipment will be supported on concrete slabs-on-grade / mat foundations.
Assumed Array Construction	Steel-framed racking-system supported on driven W6x9 steel piles.
Finished Grade Elevation	Grading plan is not available at the time of this report. The project is expecting to follow the existing topography.

Item	Description
Estimated Maximum Loads	**Pile Foundation Loads** ■ Uplift: 2 to 3.5 kips (assumed – does not consider frost heave) ■ Lateral: 1 to 3 kips at 4 to 7 feet above grade (assumed) **Equipment Slabs** ■ 100 pounds per square foot (psf)
Access Roadways	We understand that access road cross sections used for construction of the project will be the responsibility of the EPC, and that only post construction traffic with an allowable rut depth of 2 inches is what we are to design for in this report. We anticipate low-volume, aggregate-surfaced and native soil access roads will have a maximum vehicle load of 30,000 lbs. and will travel over the access roads only once per week.
Estimated Start of Construction	TBD

GEOTECHNICAL CHARACTERIZATION

Subsurface Profile

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

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	**Subsoil**	Sandy Silt (ML), light brown, loose to medium dense
2	**Glacial Till**	Silty Sand (SM), Silty Gravel (GM) to Well-graded Sand (SW), with gravel and pieces of bedrock, light brown to gray, medium dense to very dense
3	**Bedrock**	Probable bedrock (inferred bedrock encounter based on increased auger resistance while drilling)

Groundwater Conditions

The presence and level of groundwater was not observed in all boreholes while drilling and after completion of drilling. Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structures may be higher or lower than the levels indicated on the boring logs. Additionally, water may become temporarily perched above dense or silty soil layers, or bedrock surfaces. Perched groundwater would be expected to be encountered at the interface between GeoModel layers 1 and 2. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project. If groundwater was encountered during any construction activities, the engineer should be contacted immediately for necessary modifications in design. The groundwater surface should be checked prior to construction to assess its effect on site work and other construction activities.

Corrosivity

Terracon collected soil samples from test borings to determine the potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials that will be used for project construction.

The table below lists the results of laboratory water soluble sulfate, soluble chloride, electrical resistivity, and pH testing. Results are also presented in the **Exploration Results** section. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary						
Location	**Sample Depth (feet)**	**Soil Description**	**Water Soluble Sulfate (ppm)**	**Soluble Chloride (ppm)**	**Electrical Resistivity (Ω-cm)**	**pH**
IB-1	1 to 3	Silty Sand (SM)	10	43	21,440	6.16
IB-3	1 to 2.5	Sandy Silt (ML)	36	50	36,850	6.11

Results of water-soluble sulfate testing indicate samples of the on-site soils tested have an exposure class of S0 when classified in accordance with Table 19.3.1.1 of the American Concrete Institute of Concrete (ACI) Design Manual. Concrete should be designed in accordance with the provisions of the ACI Design Manual, Section 318, Chapter 19.

These test results are provided to assist in determining the type and degree of corrosion protection that may be required. For protection against corrosion to buried metals, Terracon recommends that an experienced corrosion engineer be retained to design a suitable corrosion protection system for underground metal structures or components.

Thermal Resistivity

Laboratory thermal resistivity testing was performed by Terracon on two (2) soil samples obtained during our field exploration from depths of approximately 1 to 3 feet below the existing ground surface. The thermal resistivity testing was performed in general accordance with the IEEE standard. The dry-out curves were developed from soil specimens compacted to 85% of the standard Proctor criteria (ASTM D698) at the optimum moisture content and dried to 0% moisture to develop the dry-out curves. The thermal resistivity ranged from approximately 81 to 87 °C-cm/watt for moisture content of soils ranging between 10% and 20%. The thermal resistivity was approximately 350 °C-cm/watt for dry soils. The results of the laboratory thermal resistivity testing are presented in the **Exploration Results** section.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil properties encountered at the site and as described on the exploration logs and results, it is our professional opinion that the **Seismic Site Classification is C**. Subsurface explorations at this site were extended to a maximum depth of 6.9 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.

FROST CONSIDERATIONS

Mats and Slabs

The soils on this site are frost susceptible, and surface water infiltration or migration or wetting of soil by capillary rise can affect the performance of the slabs on-grade exposed to freezing climate. Exterior slabs should be anticipated to heave during winter months. If frost action needs to be eliminated in critical areas, we recommend a minimum 24 inches of non-frost susceptible (NFS) fill beneath mats and slabs. Placement of NFS material in large areas may not be feasible; however, the following recommendations are provided to help reduce potential frost heave:

- Provide surface drainage away from slabs, and toward the site storm drainage system.
- Install drains below exterior slabs and connect them to the storm drainage system.
- Slope subgrades to allow potentially perched water in aggregate base layers to be directed toward a site drainage system.
- Place NFS fill as backfill beneath slabs critical to the project.
- Place a 3 horizontal to 1 vertical (3H:1V) transition zone between NFS fill and other soils.

As an alternative to extending NFS fill to the full frost depth, consideration can be made to placing extruded polystyrene or cellular concrete under a buffer of at least 2 feet of NFS material.

Solar Panel Support Piles and Ground Screws

The axial capacity of the steel piles/screws is highly dependent upon near surface conditions and must take into consideration environmental factors reducing the axial capacity in the near surface. One of the major environmental factors impacting pile length or ground screw embedment is adfreeze stress and the depth to which it applies. The soil in the active frost zone consists of sandy silty, silty clay, and silty sand, and is frost susceptible.

As the frost penetrates deeper into the soil and the ground swells due to freezing, the ground surface will rise due to frost heaving. The upward displacement is due to freezing water contained in the soil voids along with the formation of ice lenses in the soil. The freezing material grips the steel pile/screw and exerts an uplift force due to the adfreeze stress developed around the surface area of the pile/screw. The amount of upward force depends on the following:

- The thickness of ice lenses formed in the seasonal frozen ground;
- The bond between the steel pile/screw surface and the frozen ground; and
- The surface area of the steel pile/screw in the seasonally frozen ground.

We recommend an adfreeze stress of 1,500 psf be used to calculate the uplift loads due to frost heave. Due to variable near surface soil conditions, adfreeze depths may vary for this site. We recommend the depth to which the adfreeze stress applies to be 2 feet or to the bedrock surface, whichever is less. These adfreeze depths correspond to an air-freezing index for a 100-year return period. A load factor of 1.0 should be applied to the adfreeze stress.

Frost heave uplift forces may govern the design and length of the driven piles/screws. The factor of safety against uplift should be determined based on discussions with the owner and design engineer considering the desired level or risk, construction costs, and the long-term maintenance program.

GEOTECHNICAL OVERVIEW

Subsurface conditions below this site generally consist of subsoil overlying glacial till, which is in turn underlain by bedrock. We believe these subsurface conditions are generally suitable for the proposed development and construction of a solar plant. However, driven pile foundations may become a challenge since shallow bedrock is expected throughout the proposed property.

As presented in **Exploration Results**, the subsoil consisting of silty sand and/or sandy silt with trace of gravel was encountered up to 2.5 feet at all test boring locations. The glacial till deposits were encountered below the subsoil to the maximum depth of exploration at 6.9 feet below existing grade in boring IB-1. The glacial till deposits consist of medium dense to very dense silty sand with gravel, silty gravel with sand, and well-graded sand. Probable bedrock was encountered prior to planned exploration depth of all test borings as shown below.

Test Boring ID	Auger/Sampler Refusal Depth Below Grade (feet)
IB-1	6.9
IB-2	6.5
IB-3	2.5
IB-4	6.0
IB-5	2.5

We consider development of the photovoltaic solar project to be technically feasible from a geotechnical standpoint. However, piles driven into the subgrade can be expected to encounter damage and refusal due to very dense glacial till and the presence of probable bedrock expected to be within the subsurface at any given location, as demonstrated during the boring exploration program. Understanding that driven piles are the preferred foundation system for a solar PV project, and the potential presence of very dense glacial till or probable bedrock within the anticipated foundation driving depth, we recommend a pile driving program be developed to confirm the amount of piles deflected off their alignment due to probable bedrock, and record the drive times to assess the difficulty with which piles may penetrate the subgrade soil conditions on this site.

An alternative to driving piles would be to install piles in pre-drilled (undersized or oversized) holes. Another alternative would be to consider ground screw piles (Krinner, or similar). Design recommendations and construction considerations for the foundations are presented in the **Foundations** section of this report. The axial capacity of the steel piles is highly dependent upon near surface conditions and must take into consideration environmental factors reducing the axial capacity in the near surface. One of the major environmental factors impacting pile length is adfreeze stress and the depth to which it applies. The soil in the active frost zone consists primarily of sand with high silt content and is frost susceptible. We recommend an adfreeze stress of 1,500 psf be used to calculate the uplift loads due to frost heave. We recommend the depth to which the adfreeze stress applies to be 2 feet and that a load factor of 1.0 be applied.

We anticipate several small ancillary structures to house equipment and provide storage as part of the project. The proposed structure type and loading information was not available at the time of this report. We believe these ancillary structures may be supported on shallow spread footing foundation systems or reinforced concrete mat foundation systems bearing on a minimum of 2 feet of non-frost susceptible soil placed as presented in the **Site Preparation** section of this report. For loads exceeding 80 kips, we should be contacted to perform settlement analyses on a case-by-case basis. The **Slab on Grade or Mat** section addresses slab-on-grade/mat support of ancillary structures.

Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section. The **General Comments** section provides an understanding of the report limitations.

SOLAR ARRAY PILE FOUNDATION

This site presents cost considerations for supporting the solar panels on driven steel pile foundations. We expect driven piles to encounter refusal above the required embedment depth. Auger refusal on probable bedrock was encountered at a depth of 2.5 feet below grade in borings IB-3 and IB-5 and between 6.0 and 6.9 feet in borings IB-1, IB-2 and B-5. Therefore, pre-drilling of undersized or oversized holes to allow for the installation of the piles to the required embedment depth will likely be required. Pre-drilling over-sized holes will negatively affect the allowable skin friction capability. If pre-drilled over-sized holes are utilized, the boreholes should be grouted to develop adequate uplift and lateral capacity. In addition, ground screws (Krinner or similar) may also be used to support the racking system that supports the panels. Design recommendations and construction considerations for both foundation systems are presented below.

We understand driven piles are the preferred foundation system for support of the solar arrays. Piles used for foundation support transmit structural loads to a stratum of comparatively higher bearing capacity and should experience relatively small amounts of movement. Based on the geotechnical engineering analyses, subsurface exploration, and laboratory test results, the proposed arrays may be supported on ground screws, or alternatively on driven steel piles. The following section addresses support of the solar arrays using ground screws and/or driven piles. Where shallow bedrock is encountered, ground screws may be considered. Alternatively, steel piles may need to be installed in pre-drilled undersized holes or pre-drilled and grouted over-sized holes.

Preliminary soil resistance parameters are recommended in the following sections and may be used for design. However, we recommend a load testing program, which typically reduces the required embedment depth and pile sections, to finalize design embedment lengths. The load testing program should test various combinations of embedment conditions.

The axial capacity of driven piles may be estimated based on skin friction developed along the perimeter of the pile, while the compression capacity may be estimated using the skin friction and end bearing. When determining embedment depths, the perimeter of a wide flange beam should be taken as twice the sum of the flange width and web depth, and the upper 24 inches of soil for each pile should be neglected.

Axial Capacity Recommendations

The panels may be supported on either driven or pre-drilled and grouted steel piles, which should be structurally designed to resist compression, uplift, and bending forces. For design purposes, the upper 2 feet of soil should not be relied upon for axial compression and uplift resistance because it is within the active frost zone.

The following design parameters have been estimated based on static pile analysis for small W-section piles typically used for solar array support, driven into either native soil, under-sized pre-drilled holes, or grouted in 8-inch diameter pre-drilled over-sized holes. Note that conventional pile analyses typically underestimate the capacity of piles used in solar arrays, and the more effective means of determining pile capacities for tension, compression, or lateral loads is through pile load tests. The following parameters may be used for design. However, pile embedment depths and sections could be reduced following completion of pile load testing.

Pile Embedment Depth Below Ground Surface (feet)	Material	Ultimate Skin Friction (psf)[1]	Ultimate End Bearing Pressure (ksf)[1, 2]
0 to 2	Frost zone	Neglect	Neglect
2 to 7	Glacial Till Deposits	350	150
Varies	Bedrock	1,800	250

1. We recommend a factor of safety of 2.0 be applied to the ultimate skin friction and 3.0 to the end bearing.
2. The end bearing should be calculated using the cross-sectional area of the pile (i.e. 0.019 square feet for a W6x9).

The above values are to be used in the following equations to obtain the ultimate compressive or uplift capacity of a pile:

$$Q_{ult(compressive)} = q_t \times A \times (1{,}000\, {}^{lbs}\!/\!_{kips}) + q_s \times P \times H$$

$$Q_{ult(uplift)} = q_s \times P \times H$$

$Q_{ult\ (compressive)}$ = Ultimate compressive capacity of pile (lbs)
$Q_{ult\ (uplift)}$ = Ultimate uplift capacity of pile (lbs)
q_t = End bearing pressure per table above (ksf)
A = Cross sectional area of pile tip (sf, W6x9 = 0.019sf)
q_s = Skin friction per table above (psf)
P = Perimeter area per foot of pile (sf/ft, W6x9 = 1.64 sf/ft)
H = Depth of embedment of pile (ft)

The skin friction perimeter for pre-drilled and grouted piles can be calculated using the surface area of the pre-drilled hole. The values provided in the table represent ultimate values. Therefore, a factor of safety of 2 should be applied to the skin friction and 3 for end bearing values.

Lateral Capacity Recommendations

The parameters in the following tables can be used for analysis of the lateral capacity of steel piles driven in either native soil, under-sized pre-drilled holes, or over-sized pre-drilled and grouted holes for support of solar panel arrays. These parameters are based on correlations with SPT results, published values, and our experience with similar soil types.

Depth (feet)	Material	LPile Soil Model[1]	γ' (pcf)[2]	ϕ (°)[3]	k (pci)[4]	p-Multiplier
0 to 2	Subsoil	Sand (Reese)	100	29	Default	0.7
2 to 7	Glacial Till (Above Groundwater)		120	32		1.0
Varies	Bedrock	Strong Rock[5] (Vuggy Limestone)	102	N/A	N/A	1.0

1. p-y curve
2. γ': Effective Unit Weight
3. ϕ: Friction Angle of Soil
4. k: Soil Modulus
5. Estimated uniaxial compressive strength, q_u: 4,000 psi

The effective unit weight, friction angle, default soil modulus, and strain factor were based on the correlations of the field penetration resistance (SPT) values obtained from the borings, published values, and our experience with similar soil types. Existing p-y models typically under-predict the lateral capacity of shallow driven piles. Therefore, the p-multiplier is most likely higher but would need to be confirmed based on results of site-specific load test results. These results should be used for LPILE analysis only.

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

Driven Pile Construction Considerations

Very dense glacial till along with cobbles and boulders were encountered in the borings and are commonly found in glacially deposited soil. Pile installation via conventional methods – such as driving into a virgin subgrade may encounter difficulty and may result in early refusal and inadequate penetration, or else may cause excessive pile deflection, rotation or torsional rotation. We recommend a pile driving program be developed to confirm the amount of piles knocked off their alignment due to difficult driving conditions, and record the drive times to assess the difficulty with which piles may penetrate the subgrade soil conditions on this site. Obstructions should be anticipated based on the results of the borings and, such conditions may require pre-drilling either undersized or over-sized holes and grouting.

Auger drilling typically is unsuccessful for subgrades containing appreciable cobbles and boulders. We expect that percussive drilling methods such as ODEX or air-rotary will be necessary to complete pre-drilled holes to their design depth.

Piles set in a grout- or concrete-backfilled borehole would develop considerable axial and lateral capacity over a relatively short embedded distance. This would result in somewhat reduced pile lengths for the project, which may offset some of the expense of drilling and the use of grout or concrete backfill. Production pile testing should be performed on piles installed in predrilled holes with or without cement grout holes to confirm their capability to carry the foundation loads.

Where difficult pile driving conditions are encountered, pre-drilling either under-sized holes (approximately 5-inch diameter) or over-sized holes (minimum 8-inch diameter) could be explored to facilitate driving. A test program should be completed to determine the feasibility of using under-sized holes. Design bond strength between grout and steel piles is recommended to be 40 psi (5,760 psf) for grout with a 28-day strength of 3,000 psi.

Undersize Holes Design Recommendations

In areas of driven pile refusal prior to reaching the desired pile depth, it may be appropriate to predrill an undersized hole at the pile location to a depth less than the design depth of the pile. The predrilled hole may then be backfilled with the cuttings, provided cobbles and boulders are culled from the material. The objective of predrilling an undersized hole is to facilitate the driving of the web without disturbing the native soils supporting the flanges. Since the lateral and axial capacities are mostly reliant on the soil pile interaction at the flanges, the soil parameters used for design should be confirmed with a pile load testing program that includes pre-drilling undersized holes.

Ground Screw Foundation Recommendations

The photovoltaic panels may be supported on a ground screw system (Krinner, or similar) deriving support from medium dense to dense. The ground screws should be structurally designed to resist vertical loading and uplift, and also bending forces. The upper 2 feet in soil should not be relied upon for axial compression and uplift resistance because it is within the active frost zone.

The ground screws should be designed by the design-build engineer. Full-scale pull-out and lateral load testing should be performed on selected screws to assess compression, uplift and lateral capacities and screw length.

Lateral capacity of vertically installed ground screws is primarily dependent on the type and strength of the soil against which the screw is pushed by the horizontal load. Higher lateral capacities may be feasible; however, we recommend lateral load testing be performed

Ground Screw Construction Considerations

Ground screws should be installed by a contractor experienced in this type of foundation construction and licensed by the manufacturer of the foundation components. The allowable load carrying capacity of ground screws depends mainly on the final torque resistance. Each screw installation should be independently monitored and the depth and final torque resistance checked against the calculations by the Engineer for the manufacturer. Very dense soil conditions and bedrock were encountered in the explorations; the designer and contractor should consider these aspects in completing the design and choosing installation methods.

SLAB ON GRADE / MAT

Several pieces of equipment for the project will be supported on slabs or mats, constructed near the finished grade surface. Design parameters for slabs or mats assume the requirements in the **Earthwork** section have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the slab/mat.

The following sections present design recommendations and construction considerations for the shallow foundations for proposed lightly-loaded structures and related structural elements.

Design Recommendations

Item	Description
Slab-on-Grade or Mat Support[1]	Minimum 24 inches of NFS Fill compacted to at least 95% of ASTM D 1557 Minimum of 12 inches of NFS if exterior slabs are located in areas of shallow bedrock
Allowable Bearing Capacity[2]	3,500 psf
Settlement **Total** **Differential**	 <1.0 inch About ⅔ of total settlement
Estimated Modulus of Subgrade Reaction[3]	150 pounds per square inch per inch (psi/in) for point loads
Ultimate Coefficient of Sliding Friction	0.45

1. Slabs should be structurally independent of footings or walls to reduce the possibility of slab cracking caused by differential movements between the slab and foundation.
2. Allowable bearing capacity developed using factor of safety of 3.0.
3. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in the **Earthwork** section, and the slab support as noted in this table. It is provided for point loads. The modulus recommended is for compacted NFS or Structural Fill over dense native soil and point-load areas of 1 foot by 1 foot. An adjustment is necessary for larger mat sizes.

Construction Considerations

Finished subgrade within and for at least 10 feet beyond the slab/mat should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition. If the subgrade should become damaged or desiccated prior to construction of slabs/mats, the affected material should be removed and Structural Fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the slab/mat support course.

The Geotechnical Engineer should approve the condition of the subgrades immediately prior to placement of the slab/mat support course, reinforcing steel and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

EARTHWORK

Earthwork is anticipated to include clearing and grading for access road and ancillary equipment. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, slabs, and aggregate surfaced roadways.

Site Preparation

Prior to placing fill, existing vegetation and root mat should be removed. Complete stripping of the topsoil should be performed in the proposed equipment slab areas, access roadways, and staging areas. Exposed surfaces within the footprint of the self-contained structures should be free of mounds and depressions which could prevent uniform compaction.

Stripped materials consisting of vegetation and organic materials should be wasted from the site or used to revegetate landscaped areas or exposed slopes after completion of grading operations. If it is necessary to dispose of organic materials on-site, they should be placed in non-structural areas, and in fill sections not exceeding 5 feet in height.

Foundation, slab/mat and roadway inorganic subgrades should be proofrolled to aid in the identification of weak or unstable areas within the near surface soils. Proof-rolling should be performed with an adequately loaded vehicle such as a fully-loaded tandem-axle dump truck. The proof-rolling should be performed under the direction of the Geotechnical Engineer. Areas excessively deflecting under the proofroll should be delineated and subsequently addressed by the Geotechnical Engineer.

Based on the outcome of the proof-rolling operations, some undercutting or subgrade stabilization may be expected. Methods of stabilization, outlined below, could include scarification and re-compaction and/or replacing unstable materials with granular fill (with or without geotextiles). The more suitable method of stabilization, if required, will be dependent upon factors such as schedule, weather, size of area to be stabilized and the nature of the instability.

- **Scarification and Re-compaction** – It may be feasible to scarify, dry, and re-compact the exposed subgrades during periods of dry weather. The success of this procedure would depend primarily upon the extent of the disturbed area. Stable subgrades may not be achievable if the thickness of the soft soil is greater than 12 inches.

- **Granular Fill –** The use of Crushed Stone or Structural Fill could be considered to improve subgrade stability. Typical undercut depths would range from about 8 to 24 inches. The use of high modulus geotextiles should be limited to outside of the array area. The maximum particle size of granular material placed immediately over geotextile fabric or geogrid should not exceed 2 inches.

Over-excavations should be backfilled with Structural Fill placed and compacted in accordance with the following sections. Subgrade preparation and selection, placement, and compaction of Structural Fill should be performed under engineering-controlled conditions in accordance with the project specifications.

Fill Material Types

Fill required to achieve design grade should be classified as Structural Fill and General Fill. Structural Fill is material used below, or within 10 feet of structures such as mats/slabs, access roads, or constructed slopes. General Fill is material used to achieve grade outside of these areas. Earthen materials used for Structural and General Fill should meet the following material property requirements:

Fill Type [1]	Rhode Island Department of Transportation (RIDOT) Item	Acceptable Location for Placement
General Fill [2]	M.01.01 Common Borrow	General raise in grade fill. General Fill should not be placed within the foundation bearing zone of settlement sensitive structures.
Structural Fill	M.01.09 Table 1 - Gravel Borrow Type 1a	Beneath exterior slabs as raising grade below NFS.
Crushed Stone	M.02.03 – ¾ inch Coarse Aggregate	For use over wet subgrades as needed.
Non-Frost Susceptible Fill (NFS) [3]	M1.09 Table 1 - Crushed Stone or Crushed Gravel	Beneath exterior slabs on grade or mats.
Aggregate Surface Course	M1.09 Table 1 - Crushed Stone or Crushed Gravel	Access road surface course.
Aggregate Subbase Course	M.01.09 Table 1 - Gravel Borrow Type 1a	Access road subbase course.

1. Compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used. Fill should not be placed on frozen subgrade.
2. General Fill should have a maximum particle size of 6 inches and no more than 25 percent by weight passing the No. 200 sieve.
3. Non-Frost Susceptible (NFS) Fill should contain less than 5 percent material passing No. 200 sieve size.

Fill Compaction Requirements

Structural and General Fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Fill Lift Thickness	■ 12 inches or less in loose thickness when heavy, self-propelled compaction equipment is used. ■ 8 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used.	
Minimum Compaction Requirements[1, 2]	At least 95% of the maximum dry density as determined by ASTM D1557, Method C	93% of maximum dry density as determined by ASTM D698
Water Content Range[1]	Low plasticity cohesive: ±2% of optimum Granular: ±3% of optimum	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the standard Proctor test (ASTM D698 or D1557). We recommend testing fill for moisture content and compaction during placement. If the results of in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested, as required, until the specified moisture and compaction requirements are achieved.
2. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254).

Utility Trench Backfill

Trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If backfilled with relatively clean granular material, utility trenches should be capped with at least 12 inches of cohesive fill in unpaved areas to reduce the infiltration and preferential conveyance of surface water through the trench backfill. Alternatively, trenches should be backfilled with material that approximately matches the permeability characteristics of the surrounding soil. Fill placed as backfill for utilities located below the slab should consist of compacted Structural Fill or suitable bedding material.

Grading and Drainage

We understand there will be limited change to site grading. Adequate drainage should be provided to reduce the likelihood of an increase in moisture content of the foundation soils. Runoff should be directed away from the slab foundation.

Earthwork Construction Considerations

Unstable subgrade conditions could develop during general construction operations, particularly if the soils are wetted and/or subjected to repetitive construction traffic. Should unstable subgrade conditions develop, stabilization measures will need to be employed.

Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become frozen, wet, or disturbed, the affected material should be removed, or should be scarified, moisture conditioned, and recompacted.

As a minimum, temporary excavations should be sloped or braced, as required by Occupational Safety and Health Administration (OSHA) regulations, to provide stability and safe working conditions. The contractor, by his contract, is usually responsible for designing and constructing stable, temporary excavations and should shore, slope or bench the sides of the excavations, as required, to maintain stability of both the excavation sides and bottom. All excavations should comply with applicable local, State, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

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

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the Geotechnical Engineer. Monitoring should include documentation of adequate removal of mulch, topsoil, and bituminous concrete, proof-rolling and mitigation of areas delineated by the proof-roll to require mitigation.

Each lift of compacted fill should be tested, evaluated, and reworked as necessary until approved by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 5,000 square feet of compacted fill around carport structures and equipment slabs. One density and water content test for every 50 linear feet of compacted utility trench backfill should be performed.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the Geotechnical Engineer. In the event unanticipated conditions are encountered, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer’s evaluation of subsurface conditions, including assessing variations and associated design changes.

ACCESS ROADS

Aggregate – Surfaced Roadways

Approximate traffic loading for the project were not provided, however we anticipate low-volume, aggregate-surfaced and native soil access roads will have a maximum vehicle load of 30,000 lbs and vehicles will travel over the access roads only once per week. If greater load repetitions or a higher degree of reliability are necessary, it will be necessary for us to revise our recommendations. Our recommendations should be considered minimum recommendations based on the conditions observed during our explorations.

Pavements – Subgrade Preparation

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are typically placed and compacted in a uniform manner. However, as construction proceeds, the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, or rainfall/snow melt. As a result, the aggregate-surfaced roadway subgrade may not be suitable for construction and corrective action will be required. The subgrade should be carefully evaluated at the time of construction for signs of disturbance or instability. We recommend the subgrade be thoroughly proofrolled with a loaded tandem-axle dump truck prior to final grading. All aggregate-surfaced roadway subgrade areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to placement of the aggregate surfacing.

Pavements – Design Recommendations

We understand that access road cross sections used for construction of the project will be the responsibility of the EPC, and that only post construction traffic with an allowable rut depth of 2 inches is what we are to design for in this report. Based on the above assumptions, we have provided the following minimum aggregate thicknesses for the access roadways.

Layer	Material Type and Recommended Thickness (inches)
Aggregate Surface	6 inches of RIDOT M1.09 Table 1 - Crushed Stone or Crushed Gravel
Aggregate Sub-Base	6 inches of RIDOT M.01.09 Table 1 - Gravel Borrow Type 1a

Roadway aggregate surfacing materials should consist of a blend of gravel, sand, and fines (clay and silt). We believe the maximum size particle should not exceed 2.5 inch in diameter and the gravel should be crushed with angular edges (not rounded). The blend of materials should be selected to allow for easy compaction resulting in a firm, low permeable surface promoting surface drainage off of the roadway surface. Aggregate base course should be placed in lifts not exceeding 6 inches and compacted to a minimum of 95 percent of the maximum dry unit weight as determined by ASTM D1557.

A roadway aggregate surfacing material should also contain approximately 10 percent fines (silt and clay-sized particles passing the No. 200 sieve). The fines should exhibit low to moderate plasticity (plastic index less than 15) and will act as a binder to help reduce risk for wash boarding. If the fines content of a roadway surfacing material is comprised mostly of silt, the fines will be non-plastic and the surfacing materials will not have the benefit of the binder or cohesive aspects.

In order to reduce dust, reclaimed and processed granular material may be used as the upper 2 to 4 inches of the aggregate-surfacing. The material should be graded to the specified limits for RIDOT M.01.02.2. Periodic (1 to 2 times a year following maintenance grading) spraying of the surface with magnesium chloride or other dust suppressant may also be considered to reduce dust and wash boarding.

Aggregate-surfaced roadways performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of aggregate-surfaced roadways:

- Site grades should slope a minimum of 10 percent away from the roadways;
- The subgrade and the aggregate-surfaced roadways have a minimum 10 percent slope to promote proper surface drainage;
- Consider appropriate edge drainage; and
- Install pavement drainage surrounding areas anticipated for frequent wetting such as adjacent to wetlands.

Pavements – Maintenance

The aggregate sections are considered minimal sections based upon the expected traffic and the composite subgrade conditions; however, they are expected to function with periodic maintenance if good drainage is provided and maintained.

Preventative maintenance should be planned and provided for an ongoing aggregate-surfaced roadways management program in order to enhance future roadway performance. Preventative maintenance is usually the first priority when implementing a planned maintenance program and provides the highest return on investment for aggregate-surfaced roadways.

Periodic maintenance extends the service life of the aggregate-surfaced roadways and should include re-grading and replacement of aggregate base course in any deteriorated areas. Also, thicker aggregated base course sections could be used to reduce the required maintenance and extend the service life of the aggregate-surfaced roadways. Design alternatives which could reduce the risk of subgrade saturation and improve long-term performance include installing surface drains next to any areas where surface water could pond. Properly designed and constructed subsurface drainage will reduce the time subgrade soils are saturated and can also improve subgrade strength and performance.

GENERAL COMMENTS

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

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

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

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

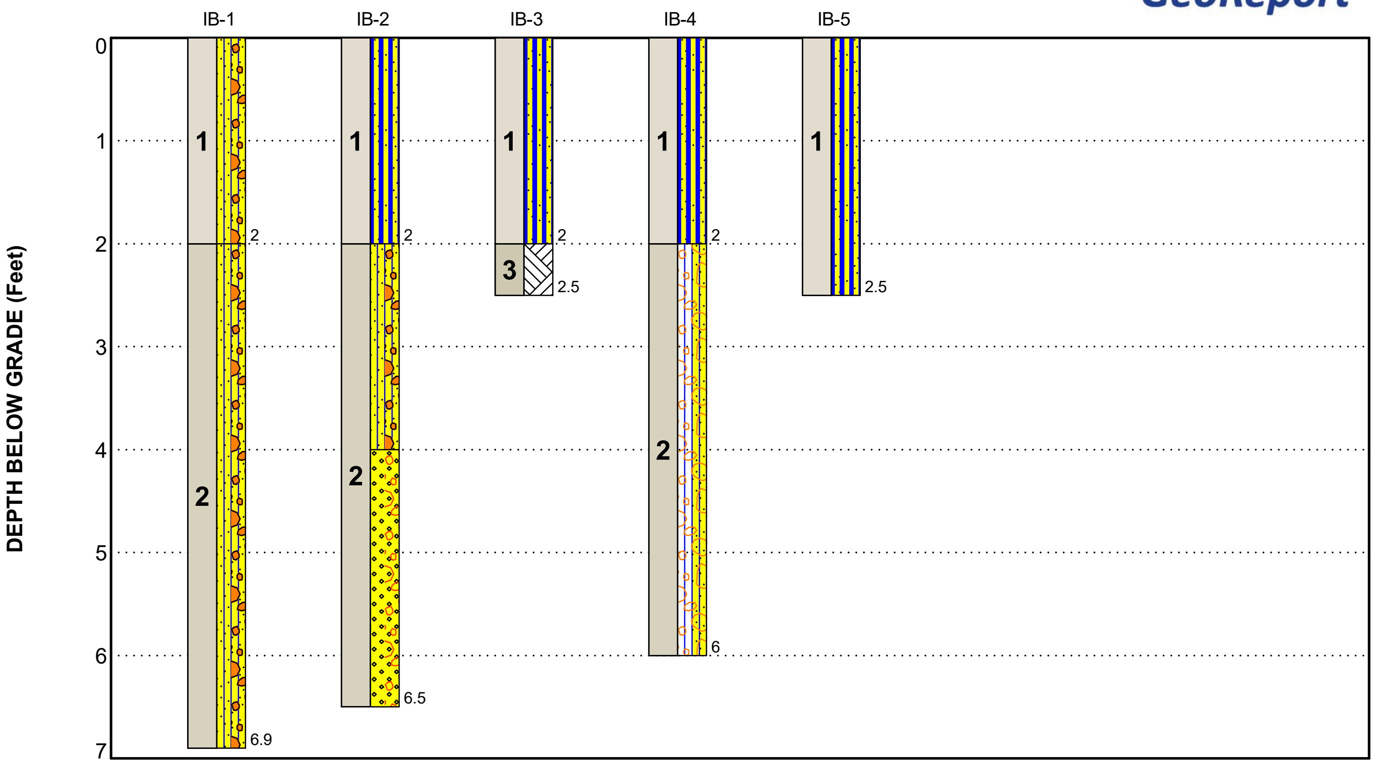
FIGURES

Contents:

GeoModel

GEOMODEL

Islander Solar ■ North Smithfield, Rhode Island
Terracon Project No. J2205052



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

Model Layer	Layer Name	General Description
1	**Subsoil**	Sandy Silt (ML), light brown, loose to medium dense
2	**Glacial Till**	Silty Sand (SM), Silty Gravel (GM) to Well-graded Sand (SW), with gravel and pieces of bedrock, light brown to gray, medium dense to very dense
3	**Bedrock**	Probable bedrock (inferred bedrock encounter based on increased auger resistance while drilling)

LEGEND

- Silty Sand with Gravel
- Sandy Silt
- Well-graded Sand with Gravel
- Bedrock
- Silty Gravel with Sand

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

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Number of Borings	Boring Depth (feet)	Planned Location
3	2.5 to 6.9	Solar Array (IB-1 to IB-3)
2	2.5 to 6.0	Access Road (IB-4 and IB-5)

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

Subsurface Exploration Procedures: Terracon observed the advancement of five (5) test borings (B-1 through B-5) throughout the site from November 18, 2020 using an all-terrain vehicle (ATV)-mounted rotary drill rig. The borings were advanced using 4¼-inch inside diameter continuous flight hollow-stem augers. At all boring locations, soil sampling was terminated prior to reaching the planned exploration depth due to auger/SPT refusal. Soil sampling was performed using split-barrel sampling procedures using a standard 2-inch outer diameter split-barrel sampling spoon driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The split-barrel samplers were driven in accordance with ASTM D 1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. The number of blows required to advance the sampling spoon the middle 12 inches of a normal 24-inch penetration was recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the depths where they are performed. Sampler refusal (50 blows less than 6 inches) was encountered within the depth of exploration in all borings.

Descriptive classifications of the soils indicated on the boring logs are in accordance with the General Notes and the Unified Soil Classification System (USCS). USCS symbols are also shown. A brief description of the USCS is attached to this report. Classification was generally by visual/manual procedures, aided by laboratory testing.

The depths of soil sampling, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Dynamic Cone Penetrometer (DCP) Test

The DCP (ASTM D6951) is a robust instrument for rapid in-situ measurement of the structural properties of the proposed unpaved access road subgrade layers. The DCP apparatus consists of a ⅝-inch diameter steel rod with a 60-degree conical tip. The rod is topped with an anvil that is connected to a second steel rod. This rod is used as a guide to allow a 17.6-lb (8-kg) hammer to be repeatedly raised and dropped from a height of 22.6 inches (575 mm). The connection between the two rods consists of anvil to allow for quick connections between the rods and for efficient energy transfer from the falling weight to the penetrating rod. Using a correlation between the data collected during DCP testing and the California Bearing Ratio (CBR), DCP data can be used to estimate CBR values for the unpaved access road subsurface layers.

In Situ DCP Test Locations	Test Quantity
IB-4 and IB-5	2

Field Electrical Resistivity Testing

Field electrical resistivity of in-situ soil was completed at two (2) locations, IB-1 and IB-2 as shown on the **Exploration Plan**. Measurements were taken along two relatively perpendicular lines having a common center point. Measurements were made in general accordance with ASTM G 57-06 (2012) using a Wenner array configuration at “a” spacings at 2½, 5, 10, 20, 40, and 50 feet. The results of field electrical resistivity are presented in our **Exploration Results**.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests to understand the engineering properties of the various soil strata, as necessary, for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods were applied because of local practice or professional judgment. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- Five (5) ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- Five (5) ASTM D422/C136 Standard Test Method for Particle-Size Distribution of Soils/ Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates
- Two (2) Corrosion Suite
  - pH Analysis (ASTM G51)
  - Water Soluble Sulfate (ASTM C1580)

- Sulfide Content (AWWA 4500-S D)
- Chloride (ASTM D512)
- Oxidation-Reduction Potential (ASTM G200)
- Total Salts (AWWA 2540)
- Electrical Resistivity – Saturated (ASTM G187)

- Two (2) ASTM D5334 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure including two (2) ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort

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

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Islander Solar ■ North Smithfield, Rhode Island

January 21, 2021 ■ Terracon Project No. J2205052

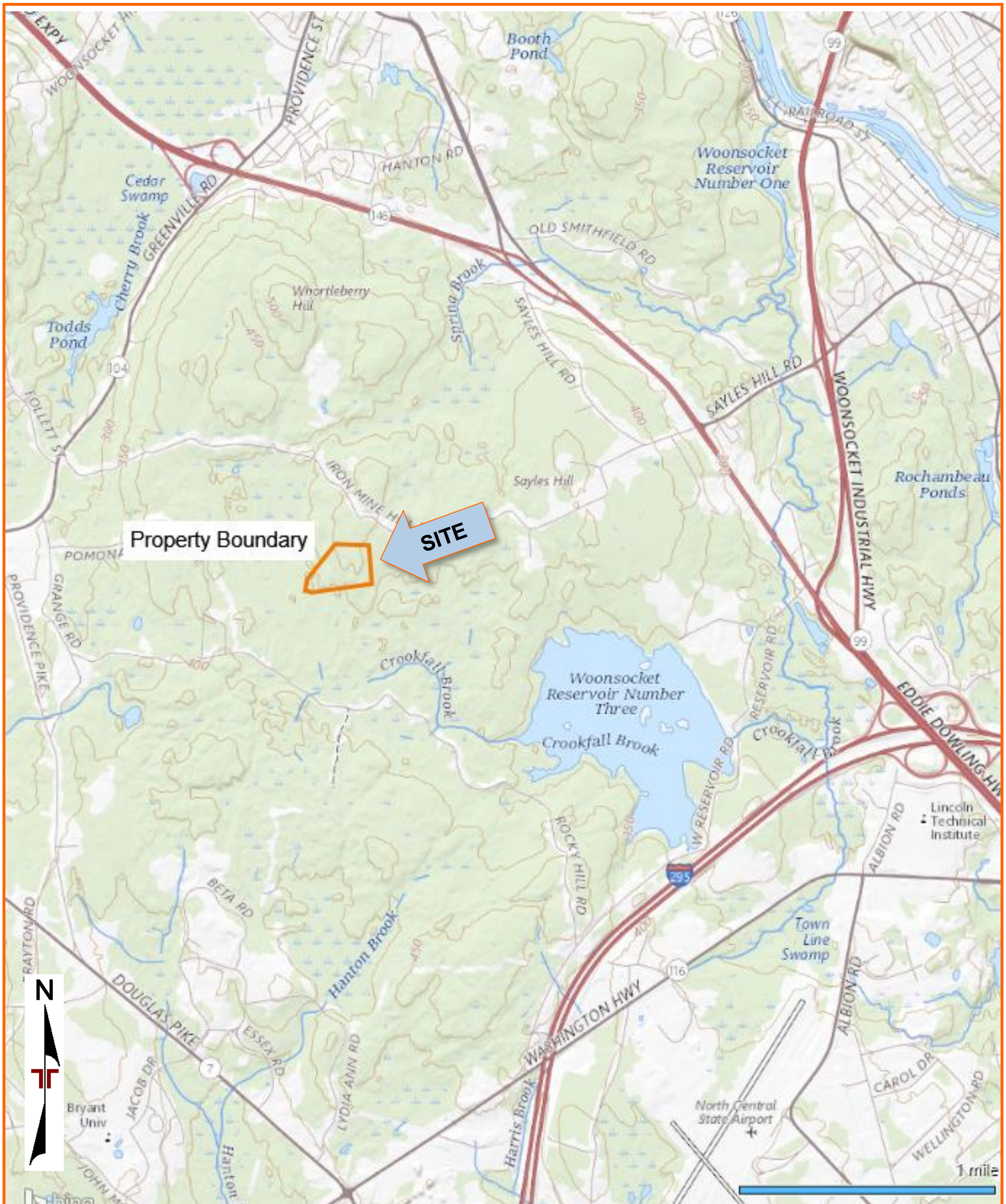


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN
Islander Solar ■ North Smithfield, Rhode Island
January 21, 2021 ■ Terracon Project No. J2205052

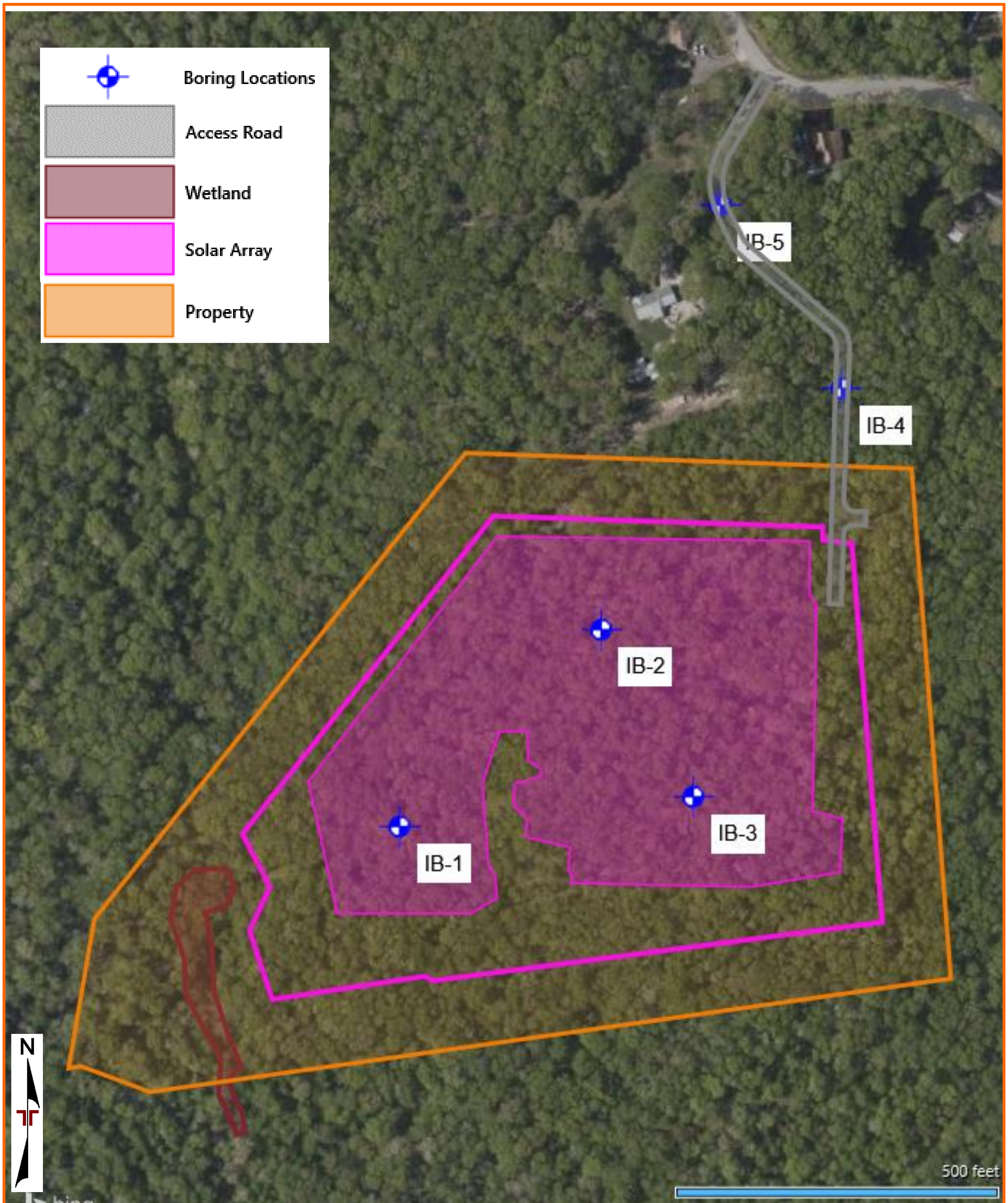


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES
MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

BORING LOG NO. IB-1

PROJECT: Islander Solar

CLIENT: Islander Solar LLC
Summit, New Jersey

SITE: 850 Iron Mine Hill Road
North Smithfield, Rhode Island

LOCATION See Exploration Plan

Latitude: 41.9527° Longitude: -71.5217°

Approximate Surface Elev.: 425 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)
1 2		**<u>SILTY SAND WITH GRAVEL (SM)</u>**, light brown, medium dense, **(SUBSOIL)** Note: Collect one bucket sample between 1 to 3 ft. 2.0	423+/-			X	11	3-4-10-5 N=14	7.8
2		**<u>SILTY SAND (SM)</u>**, with gravel and pieces of rock, light brown to gray, very dense, **(GLACIAL TILL)**				X	3	50/5"	
				5		X	10	38-50/5"	4.5
		6.9	418+/-						
		Auger Refusal on Probable Bedrock at 6.9 Feet							

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

Samples obtained using a 2-in. O.D. split spoon sampler

Hammer Type: Automatic

Advancement Method:
0-6.9 ft: Continuous flight augers

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations taken from Google Earth

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

Boring Started: 11-18-2020	Boring Completed: 11-18-2020
Drill Rig: CME-75	Driller: P. Michaud
Project No.: J2205052	

BORING LOG NO. IB-2

PROJECT: Islander Solar

CLIENT: Islander Solar LLC
Summit, New Jersey

SITE: 850 Iron Mine Hill Road
North Smithfield, Rhode Island

LOCATION See Exploration Plan

Latitude: 41.9531° Longitude: -71.5205°

Approximate Surface Elev.: 458 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)
1, 2		2.0	**SANDY SILT (ML)**, light brown, loose, **(SUBSOIL)** Note: Collect two bucket samples between 1 to 3 ft.	456+/-			X	8	2-4-4-6 N=8	
2		4.0	**SILTY SAND (SM)**, trace gravel, gray, medium dense, **(GLACIAL TILL)**	454+/-			X	15	7-7-16-25 N=23	6.1
		6.5	**WELL GRADED SAND (SW)**, trace gravel, gray, very dense, **(GLACIAL TILL)**	451.5+/-	5		X	10	33-50/5"	
			Auger Refusal on Probable Bedrock at 6.5 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.
Samples obtained using a 2-in. O.D. split spoon sampler

Hammer Type: Automatic

Advancement Method:
0-6.5 ft: Continuous flight augers

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations taken from Google Earth

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

Boring Started: 11-18-2020	Boring Completed: 11-18-2020
Drill Rig: CME-75	Driller: P. Michaud
Project No.: J2205052	

BORING LOG NO. IB-3

PROJECT: Islander Solar

CLIENT: Islander Solar LLC
Summit, New Jersey

SITE: 850 Iron Mine Hill Road
North Smithfield, Rhode Island

LOCATION See Exploration Plan

Latitude: 41.9526° Longitude: -71.5201°

Approximate Surface Elev.: 449 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH		ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)
1			**SANDY SILT (ML)**, light brown, loose, **(SUBSOIL)** Note: Collect 1 bucket sample between 1 to 2 ft.				X	11	2-2-4-5 N=6	23.7 21.2
		2.0		447+/-						
3		2.5	**BEDROCK**	446.5+/-						
			Auger Refusal on Probable Bedrock at 2.5 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.
Samples obtained using a 2-in. O.D. split spoon sampler

Hammer Type: Automatic

Advancement Method:
0-2.5 ft: Continuous flight augers

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations taken from Google Earth

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

201 Hammer Mill Rd
Rocky Hill, CT

Boring Started: 11-18-2020	Boring Completed: 11-18-2020
Drill Rig: CME-75	Driller: P. Michaud
Project No.: J2205052	

BORING LOG NO. IB-4

PROJECT: Islander Solar

SITE: 850 Iron Mine Hill Road, North Smithfield, Rhode Island

CLIENT: Islander Solar LLC, Summit, New Jersey

LOCATION See Exploration Plan

Latitude: 41.9541° Longitude: -71.5191°

Approximate Surface Elev.: 453 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)
1		**SANDY SILT (ML)**, trace gravel, light brown, loose, **(SUBSOIL)**					9	2-1-2-2 N=3	
		2.0	451+/-						
2		**SILTY GRAVEL WITH SAND (GM)**, light brown to gray, very dense, **(GLACIAL TILL)**					4	4-3-50/5"	4.2
				5				50/5"	
		6.0	447+/-						

Auger Refusal on Probable Bedrock at 6 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.
Samples obtained using a 2-in. O.D. split spoon sampler

Hammer Type: Automatic

Advancement Method:
0-6 ft: Continuous flight augers

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations taken from Google Earth

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

Boring Started: 11-18-2020	Boring Completed: 11-18-2020
Drill Rig: CME-75	Driller: P. Michaud
Project No.: J2205052	

BORING LOG NO. IB-5

PROJECT: Islander Solar

CLIENT: Islander Solar LLC
Summit, New Jersey

SITE: 850 Iron Mine Hill Road
North Smithfield, Rhode Island

LOCATION See Exploration Plan

Latitude: 41.9549° Longitude: -71.5197°

Approximate Surface Elev.: 467 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)
1		**SANDY SILT (ML)**, light brown, loose, **(SUBSOIL)** Note: Rock outcrops near borehole.				X	17	2-2-4-5 N=6	
		2.5	464.5+/-			X	6	3-50/1"	
		Auger Refusal on Probable Bedrock at 2.5 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.
Samples obtained using a 2-in. O.D. split spoon sampler

Hammer Type: Automatic

Advancement Method:
0-2.5 ft: Continuous flight augers

Abandonment Method:
Boring backfilled with soil cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations taken from Google Earth

Notes:

WATER LEVEL OBSERVATIONS

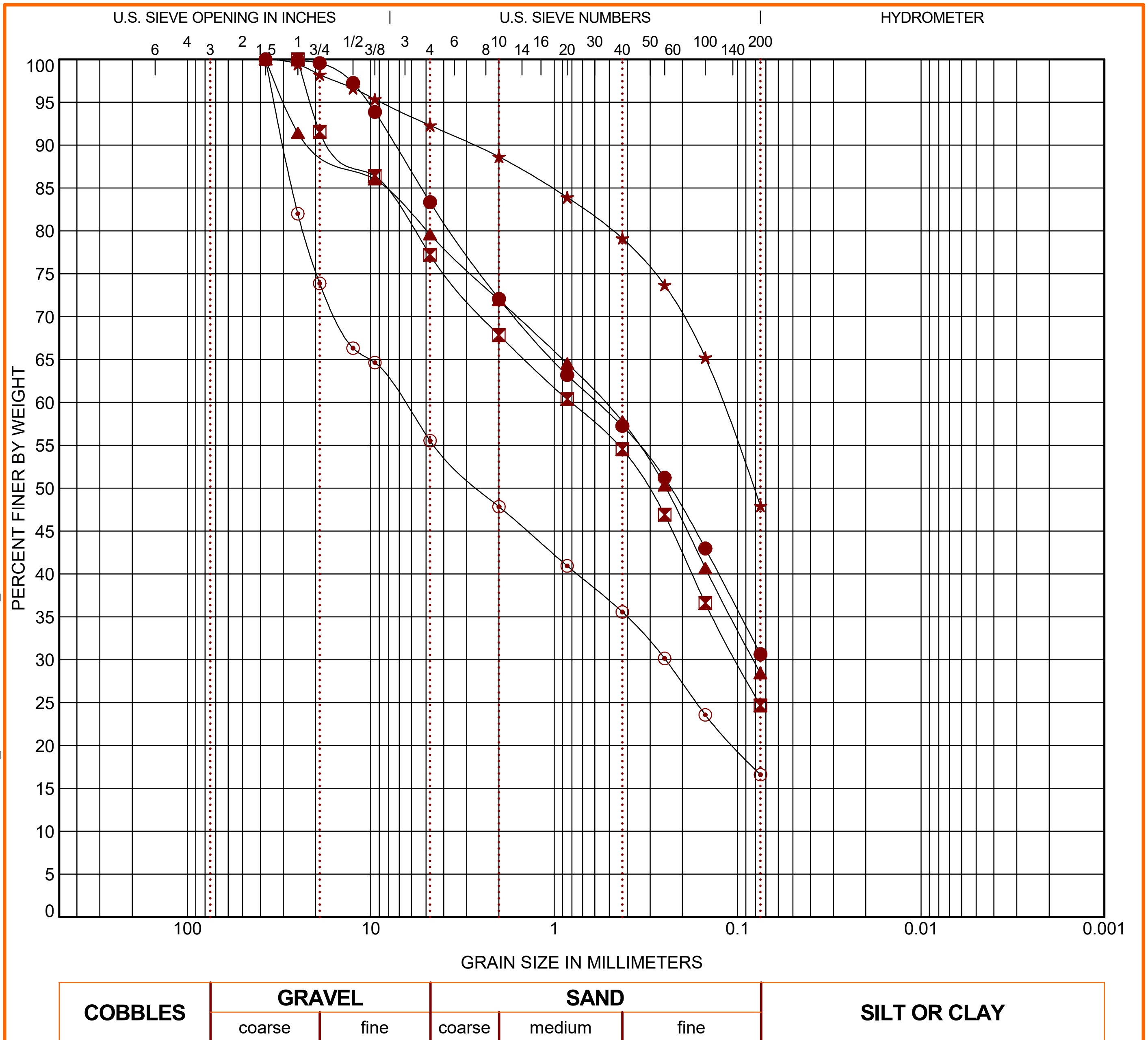
No free water observed

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

Boring Started: 11-18-2020	Boring Completed: 11-18-2020
Drill Rig: CME-75	Driller: P. Michaud
Project No.: J2205052	

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	Boring ID	Depth	USCS Classification	WC (%)	LL	PL	PI	Cc	Cu
●	IB-1	1	Silty sand with gravel (SM)	7.8					
⊠	IB-1	5 - 5.9	Silty sand with gravel (SM)	4.5					
▲	IB-2	2 - 4	Silty sand with gravel (SM)	6.1					
★	IB-3	1	Sandy silt (ML)	21.2					
⊙	IB-4	2 - 3.4	Silty gravel with sand (GM)	4.2					

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Silt	%Fines	%Clay
●	IB-1	1	37.5	0.586			0.0	16.6	52.7		30.6	
⊠	IB-1	5 - 5.9	25	0.812	0.102		0.0	22.8	52.5		24.7	
▲	IB-2	2 - 4	37.5	0.533	0.082		0.0	20.4	51.2		28.4	
★	IB-3	1	37.5	0.122			0.0	7.7	44.4		47.9	
⊙	IB-4	2 - 3.4	37.5	6.671	0.247		0.0	44.5	38.9		16.6	

PROJECT: Islander Solar

PROJECT NUMBER: J2205052

SITE: 850 Iron Mine Hill Road
North Smithfield, Rhode Island

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

CLIENT: Islander Solar LLC
Summit, New Jersey

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

Client
Islander Solar LLC

Project
Islander Solar

Sample Submitted By: Terracon (J2) **Date Received:** 12/3/2020 **Lab No.: 20-1263**

Results of Corrosion Analysis

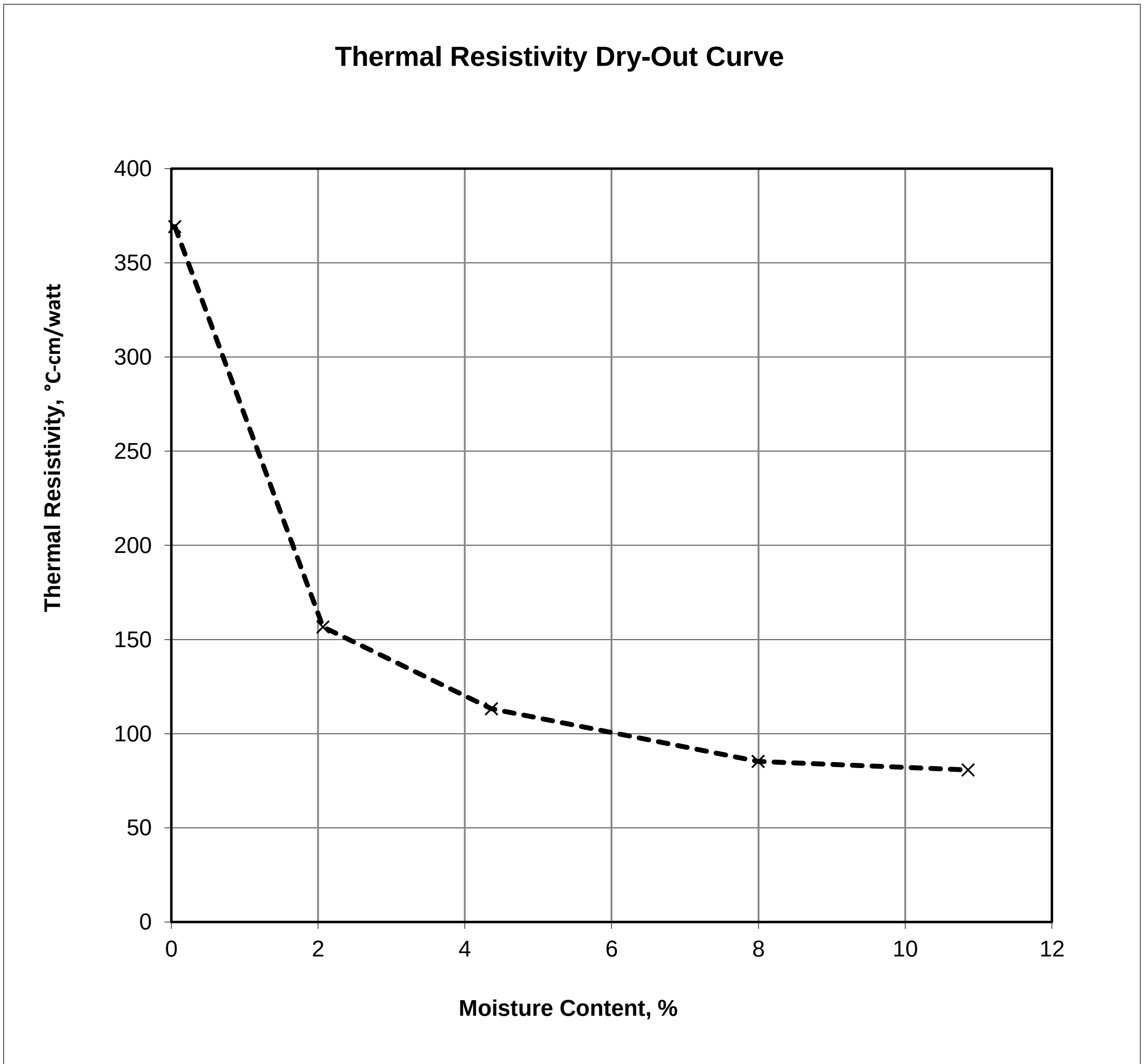
Sample Number	--	--
Sample Location	IB-1	IB-3
Sample Depth (ft.)	1.0-3.0	1.0-2.5
pH Analysis, ASTM G 51	6.16	6.11
Water Soluble Sulfate (SO4), ASTM C 1580 (ppm)	10	36
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil
Chlorides, ASTM D 512, (ppm)	43	50
Red-Ox, ASTM G 200, (mV)	+692	+693
Total Salts, AWWA 2540, (mg/kg)	171	169
Resistivity (Saturated), ASTM G 187, (ohm-cm)	21440	36850

Analyzed By: Trisha Campo
Trisha Campo
Chemist

Project Name: Islander Solar
Project Number: J2205052
Sample ID: IB1@1'-3'
Soil Type: SILTY SAND WITH GRAVEL
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 119.8
Optimum Moisture Content, %: 10.2
Target % Compaction: 85
Sample Dry Density, pcf: 102
Sample % Compaction: 85
As-received Moisture Content, %: 7.8

Thermal Resistivity Test Results

Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.0	369	23.9
2.1	157	22.3
4.4	113	22.7
8.0	85	21.7
10.9	81	21.1

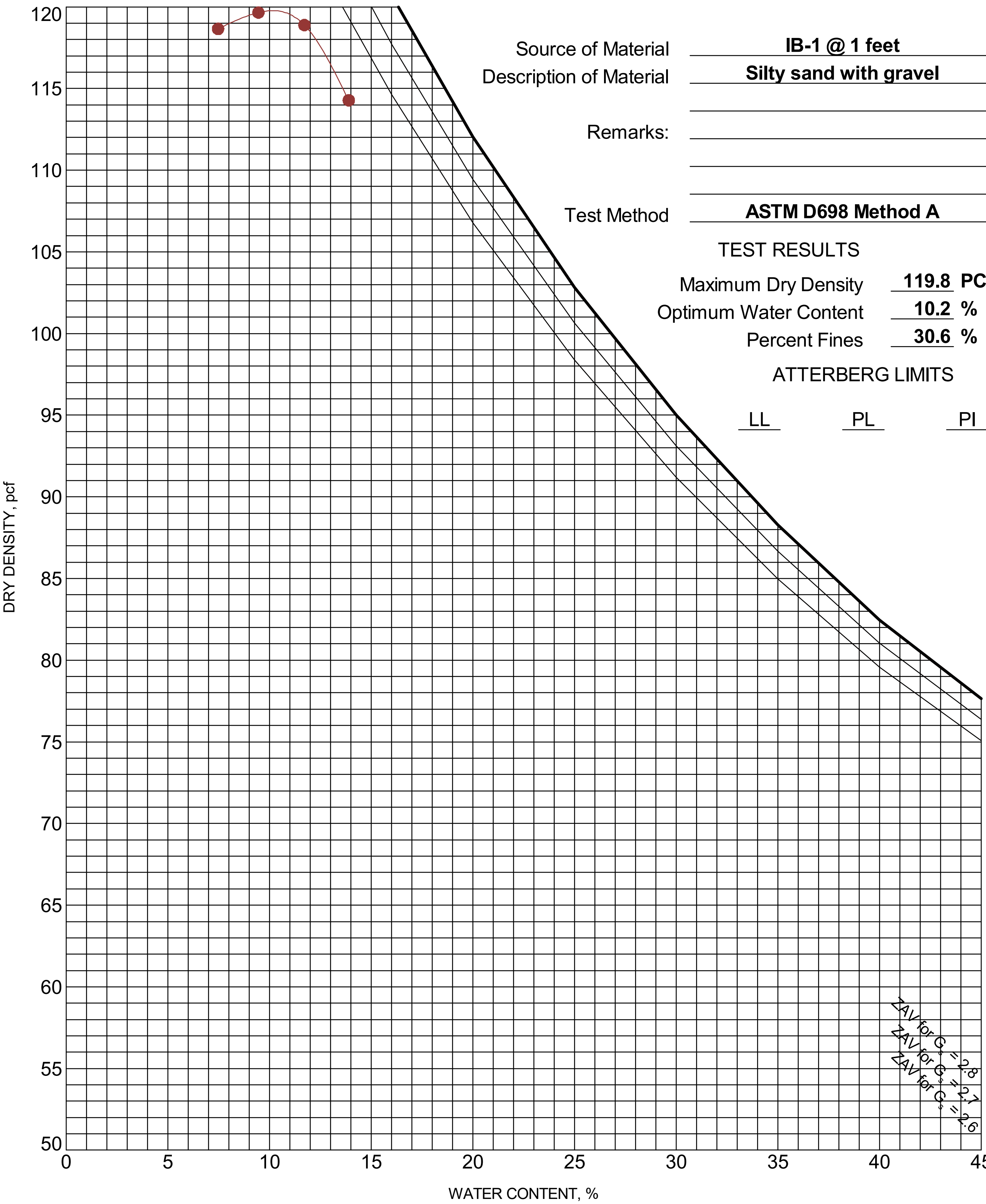


Date: 12/15/2020	Run By: AMM	Reviewed By: BWP

Terracon

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **IB-1 @ 1 feet**

Description of Material: **Silty sand with gravel**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **119.8** **PCF**

Optimum Water Content **10.2** **%**

Percent Fines **30.6** **%**

ATTERBERG LIMITS

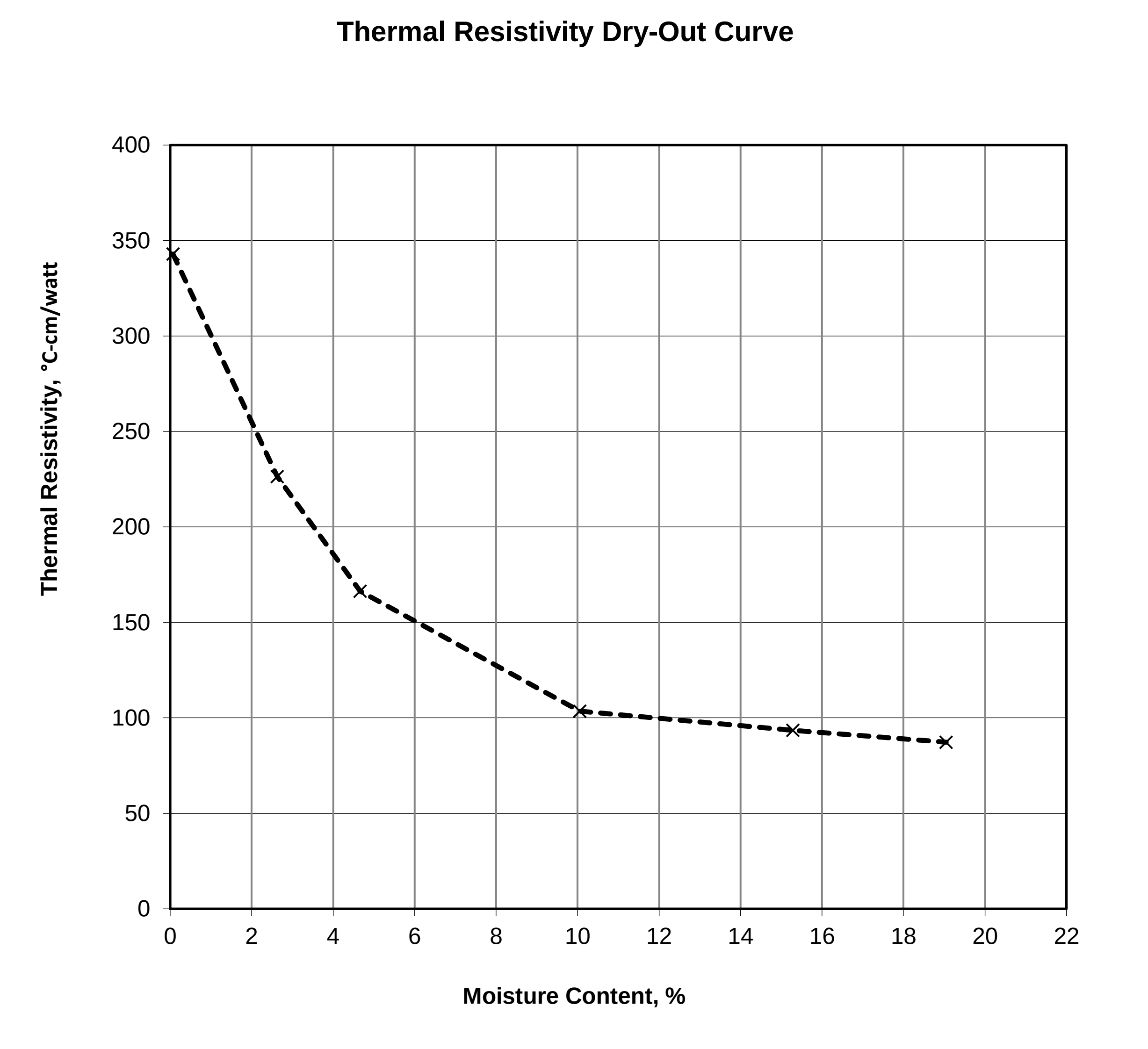
LL	PL	PI

PROJECT: Islander Solar	PROJECT NUMBER: J2205052
SITE: 850 Iron Mine Hill Road North Smithfield, Rhode Island	CLIENT: Islander Solar LLC Summit, New Jersey

Terracon
201 Hammer Mill Rd
Rocky Hill, CT

Project Name: Islander Solar
Project Number: J2205052
Sample ID: IB3@1'-2.5'
Soil Type: SANDY SILT
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 100.3
Optimum Moisture Content, %: 18.5
Target % Compaction: 85
Sample Dry Density, pcf: 85
Sample % Compaction: 85
As-received Moisture Content, %: 21.2

Thermal Resistivity Test Results		
Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.1	343	23.3
2.6	226	22.6
4.7	166	22.4
10.1	103	23.6
15.3	93	21.8
19.0	87	21.6

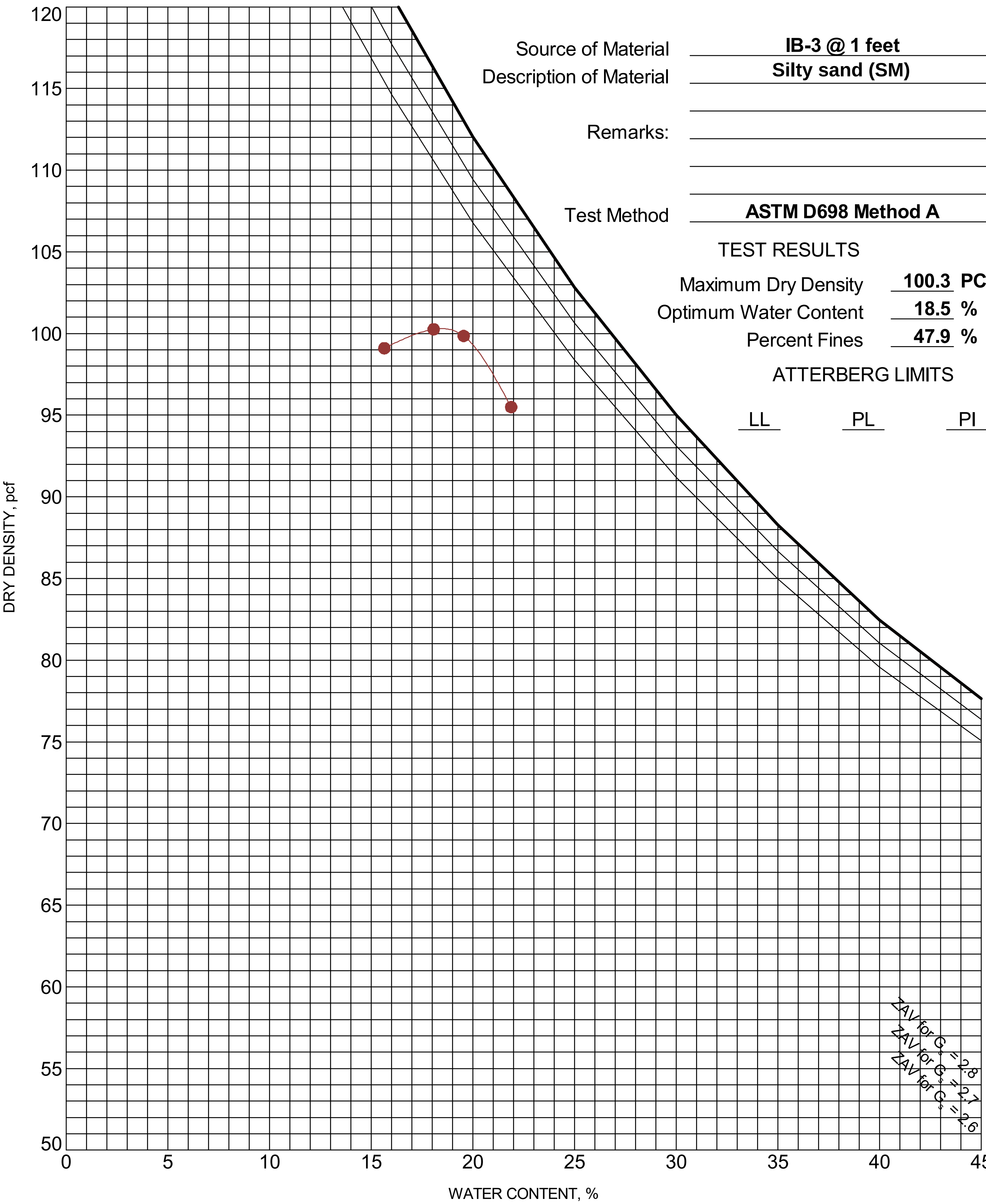


Date: 12/15/2020	Run By: AMM	Reviewed By: BWP

Terracon

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**IB-3 @ 1 feet**
Description of Material	**Silty sand (SM)**
Remarks:	
Test Method	**ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**100.3**	**PCF**
Optimum Water Content	**18.5**	**%**
Percent Fines	**47.9**	**%**

ATTERBERG LIMITS

LL	PL	PI

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V3 J2205052 ISLANDER SOLAR.GPJ TERRACON_DATATEMPLATE.GDT 12/14/20

PROJECT: Islander Solar	Terracon 201 Hammer Mill Rd Rocky Hill, CT	PROJECT NUMBER: J2205052
SITE: 850 Iron Mine Hill Road North Smithfield, Rhode Island		CLIENT: Islander Solar LLC Summit, New Jersey

FIELD ELECTRICAL RESISTIVITY TEST DATA

Islander Solar ■ North Smithfield, Rhode Island

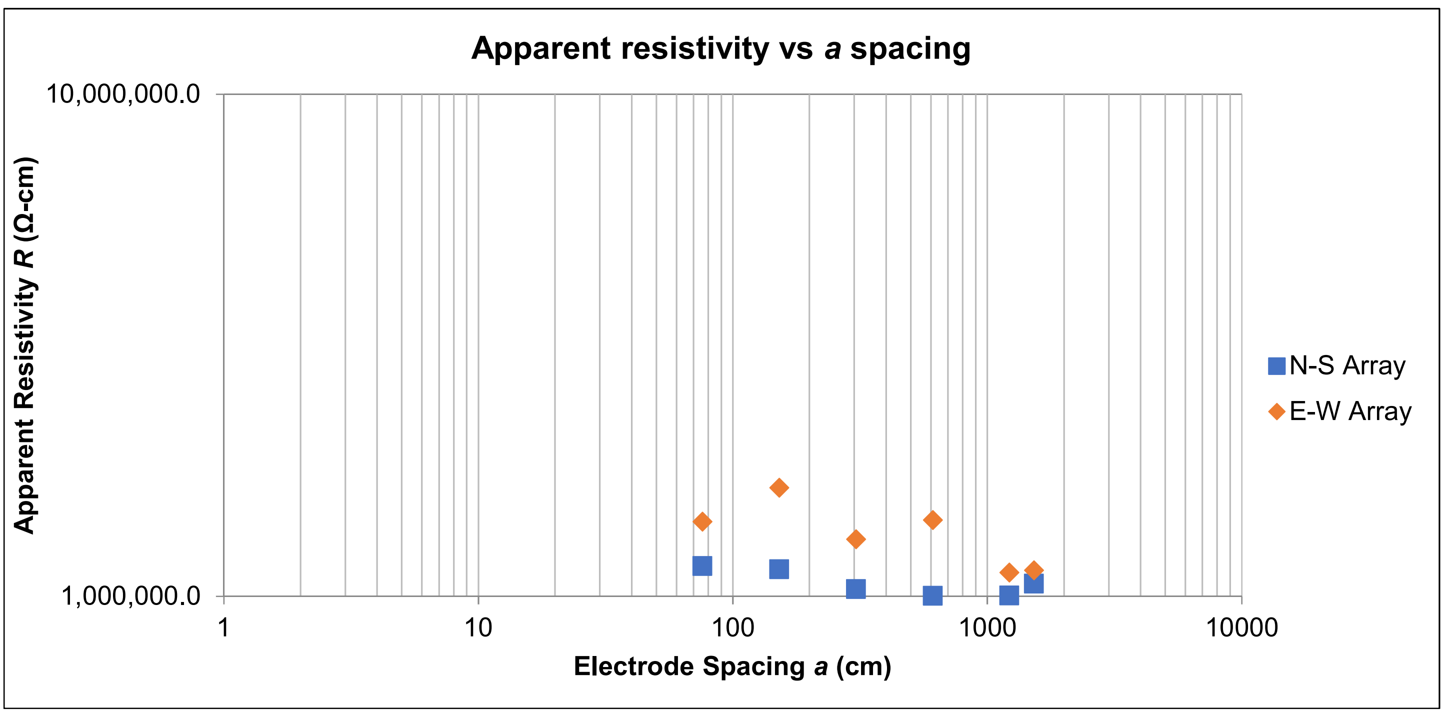
November 19, 2020 ■ Terracon Project No. J2205052

Terracon GeoReport

Array Loc.	IB-1		
Instrument	MiniSting	**Weather**	Sunny 30°F
Serial #	S1507299	**Ground Cond.**	Dry
Calibrated	September 3, 2020	**Tested By**	Ryan Decker
Test Date	November 19, 2020	**Method**	Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)
Notes & Conflicts			

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

Electrode Spacing *a*		Electrode Depth *b*		N-S Test		E-W Test	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)	Measured Resistance *R* Ω	Apparent Resistivity ρ (Ω-cm)
2.5	76	6	15	2,263.00	1150080	2,773.00	1409260
5	152	6	15	1,168.00	1134220	1,696.00	1646940
10	305	6	15	538.20	1035740	675.30	1299580
20	610	6	15	261.60	1003710	370.20	1420380
40	1219	12	30	131.20	1005950	145.50	1115590
50	1524	12	30	110.70	1060730	117.60	1126850



FIELD ELECTRICAL RESISTIVITY TEST DATA

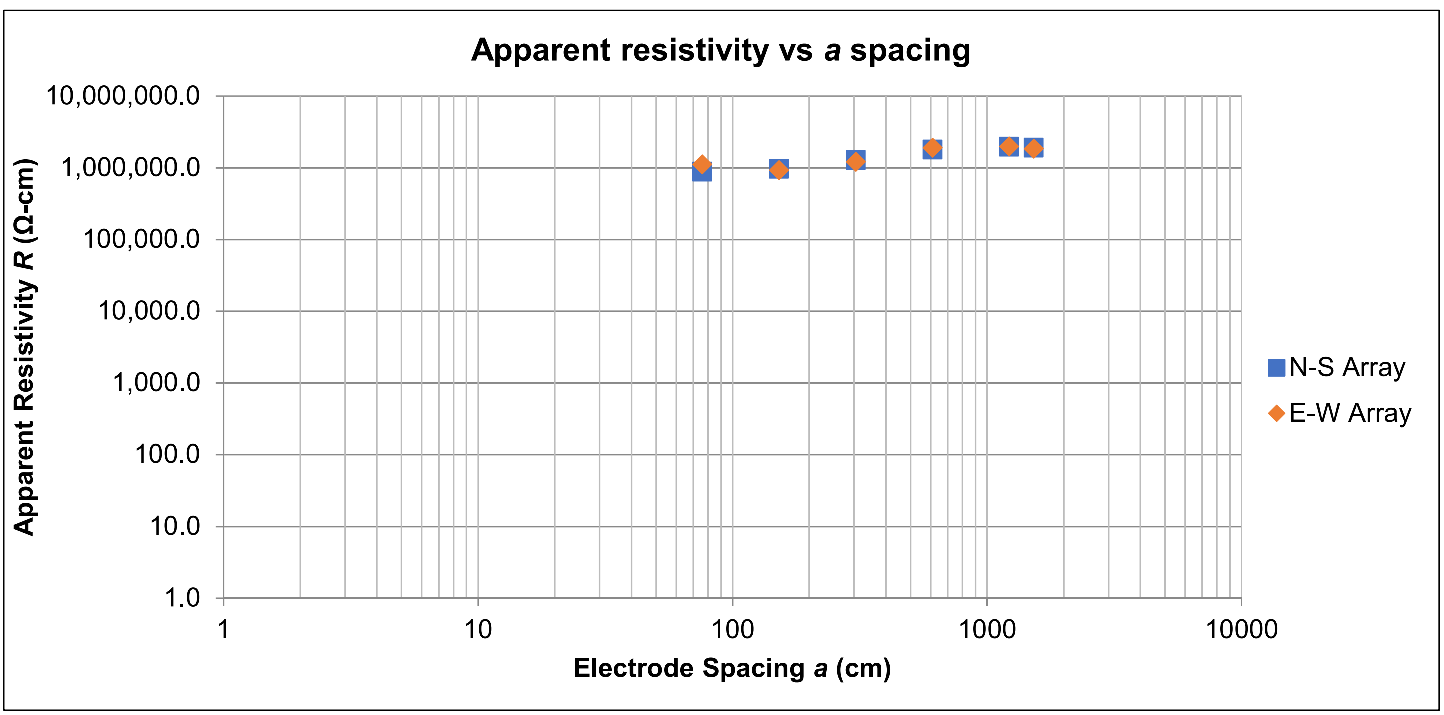
Islander Solar ■ North Smithfield, Rhode Island

November 19, 2020 ■ Terracon Project No. J2205052

Array Loc.	IB-2		
Instrument	MiniSting	**Weather**	Sunny 30°F
Serial #	S1507299	**Ground Cond.**	Dry
Calibrated	September 3, 2020	**Tested By**	Ryan Decker
Test Date	November 19, 2020	**Method**	Wenner 4-pin (ASTM G57-06 (2012); IEEE 81-2012)
Notes & Conflicts			

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

Electrode Spacing *a*		**Electrode Depth *b***		**N-S Test**		**E-W Test**	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance *R* Ω	Apparent Resistivity *ρ* (Ω-cm)	Measured Resistance *R* Ω	Apparent Resistivity *ρ* (Ω-cm)
2.5	76	6	15	1,763.00	895970	2,210.00	1123140
5	152	6	15	1,013.00	983700	959.80	932040
10	305	6	15	672.30	1293810	630.30	1212980
20	610	6	15	474.50	1820560	497.50	1908810
40	1219	12	30	259.70	1991200	260.70	1998870
50	1524	12	30	202.20	1937490	192.40	1843590



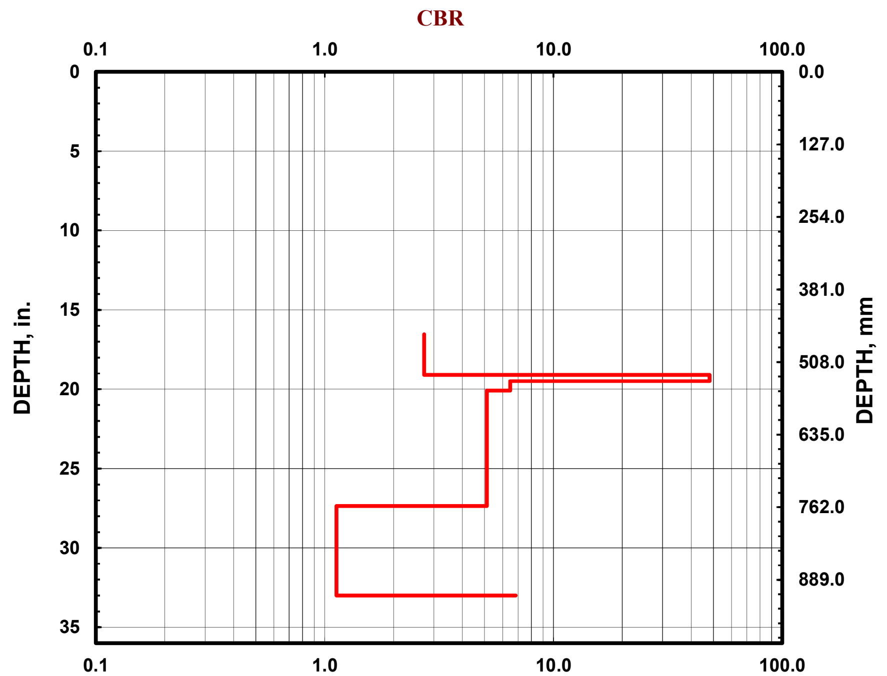
DCP TEST DATA

Project: Islander Solar
Location: East Providence RI
Date: November 18, 2020
Core Location: IB-4-1

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	420.0	0
1	485.0	1
2	495.0	2
1	510.0	1
5	695.0	5
5	838.0	5



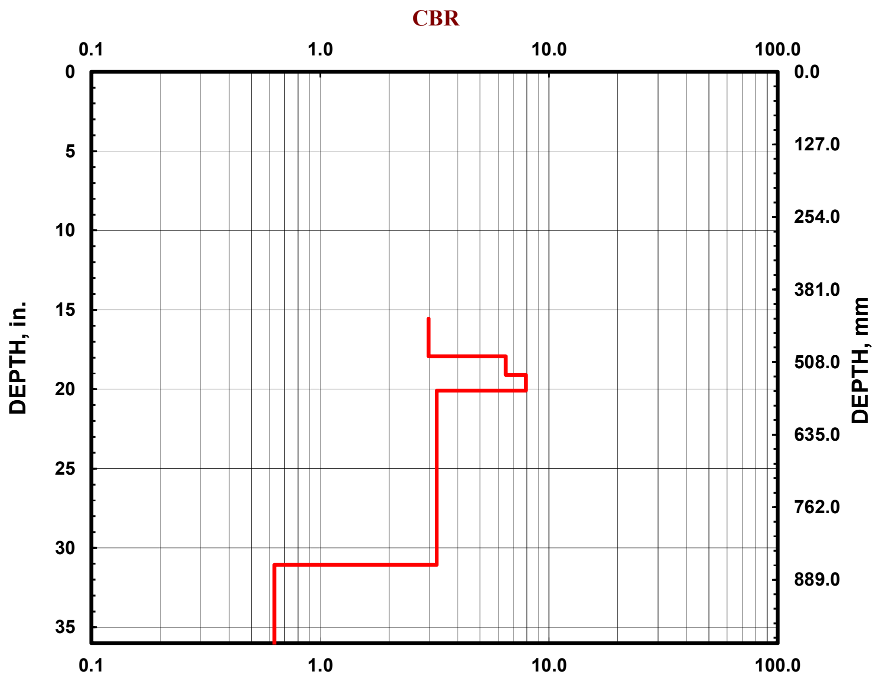
DCP TEST DATA

Project: Islander Solar
Location: East Providence RI
Date: November 18, 2020
Core Location: IB-4-2

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	395.0	0
1	455.0	1
1	485.0	1
1	510.0	1
5	789.0	5
5	1029.0	5
5	1115.0	5



DCP TEST DATA

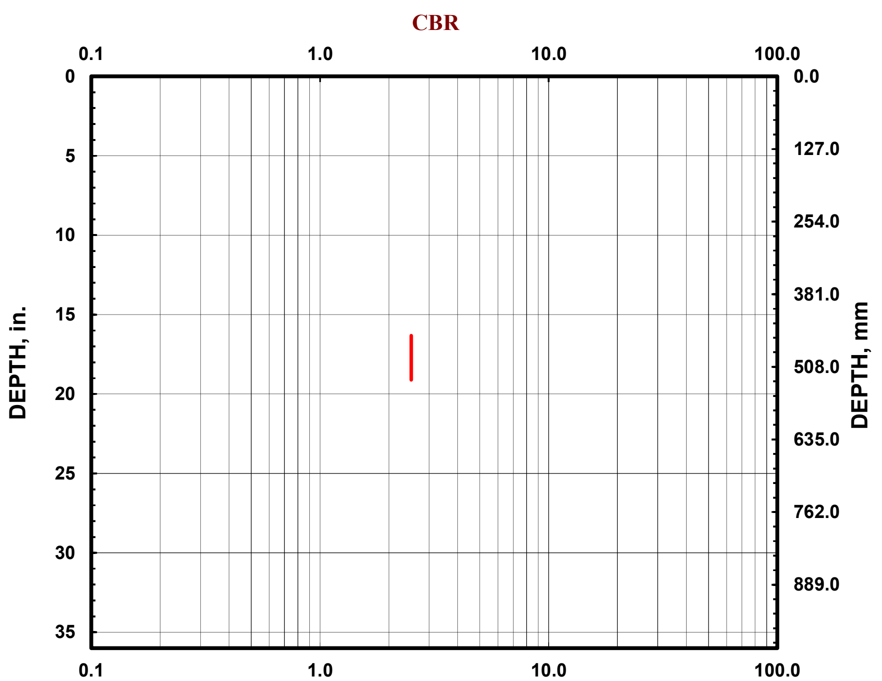
Project: Islander Solar
Location: East Providence RI

Date: November 18, 2020
Core Location: IB-4-3

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	415.0	0
1	485.0	1
1	485.0	1
1	485.0	1
1	485.0	1
1	485.0	1



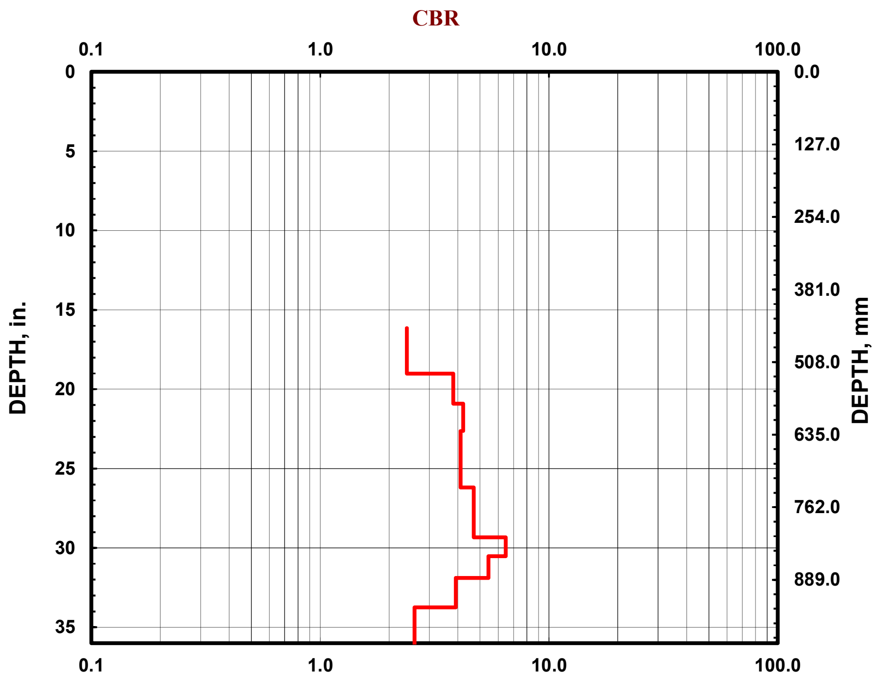
DCP TEST DATA

Project: Islander Solar
Location: East Providence RI
Date: November 18, 2020
Core Location: IB-5-1

Hammer
- ○ 10.1 lbs.
- ● 17.6 lbs.
- ○ Both hammers used

Soil Type
- ○ CH
- ○ CL
- ● All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	410.0	0
1	483.0	1
1	531.0	1
1	575.0	1
1	620.0	1
1	665.0	1
1	705.0	1
1	745.0	1
1	775.0	1
1	810.0	1
1	857.0	1
1	925.0	1
1	925.0	1
1	925.0	1
1	925.0	1
1	925.0	1



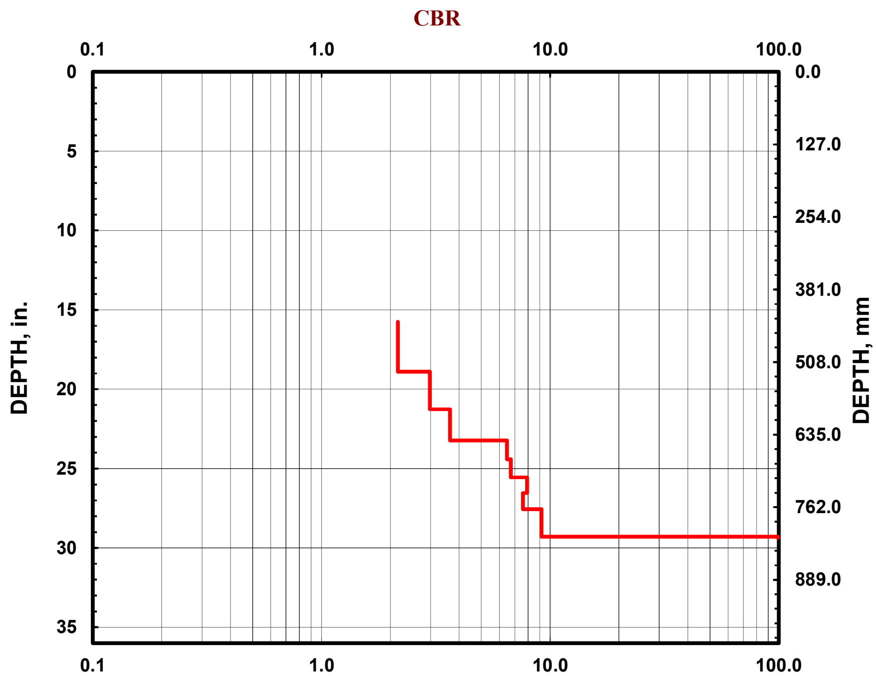
DCP TEST DATA

Project: Islander Solar
Location: East Providence RI
Date: November 18, 2020
Core Location: IB-5-2

Hammer
- ◯ 10.1 lbs.
- ◉ 17.6 lbs.
- ◯ Both hammers used

Soil Type
- ◯ CH
- ◯ CL
- ◉ All other soils

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	400.0	1
1	480.0	1
1	540.0	1
1	590.0	1
1	620.0	1
1	649.0	1
1	674.0	1
1	700.0	1
1	722.0	1
1	744.0	1
1	746.0	1
1	746.0	1
1	746.0	1
1	746.0	1
1	746.0	1



SUPPORTING INFORMATION

Contents:

Note: All attachments are one page unless noted above.

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

Islander Solar ■ North Smithfield, Rhode Island

Terracon Project No. J2205052

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

DESCRIPTIVE SOIL CLASSIFICATION

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

LOCATION AND ELEVATION NOTES

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

STRENGTH TERMS

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

RELEVANCE OF SOIL BORING LOG

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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

[O] PI < 4 or plots below “A” line.

[P] PI plots on or above “A” line.

[Q] PI plots below “A” line.