Geotechnical Engineering Report

Chinook Solar Site
Fitzwilliam, New Hampshire
April 19, 2019
Terracon Project No. J1175115

Prepared for:
NextEra Energy Resources
Juno Beach, FL

Prepared by:
Terracon Consultants, Inc.
Manchester, New Hampshire

April 19, 2019

NextEra Energy Resources
700 Universe Boulevard
Juno Beach, FL 33408

Attn: Mr. Mitchell Thiem
P: (561) 304 5235
E: Mitchell.thiem@nexteraenergy.com

Re: Geotechnical Engineering Report
Chinook Solar Site
State Route 119
Fitzwilliam, New Hampshire
Terracon Project No. J1175115

Dear Mr. Thiem:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PJ1175115 revised June 26, 2018. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

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, P.E.
Geotechnical Dept. Manager

Lawrence J. Dwyer, P.E.
Principal

Reviewed by: James M. Jackson, P.E. (FL)

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Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES
SITE LOCATION AND EXPLORATION PLANS
EXPLORATION RESULTS
SUPPORTING INFORMATION

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

REPORT SUMMARY

Topic [1]	Overview Statement [2]
Project Description	The project consists of a 40 MW DC (30MW AC) photovoltaic (PV) electric power plant situated on approximately 190 acres. The power plant will consist of fixed mount solar panels installed on pile supported steel racking frames, and appurtenances associated with an interconnection substation shown on the plans.
Geotechnical Characterization	11 test borings for the PV arrays and substation were advanced to depths ranging from approximately 14 to 50 feet below existing site grades. Dense to very dense sand and silty sand were encountered throughout the site. The soils at the site are frost susceptible. Groundwater was encountered at depths of 2 to 10 feet, with an average of 4.5 feet. Shallow bedrock may be encountered at the south end of the site, impacting solar pile driveability. Additional site characterization consisted of field electrical resistivity testing, corrosivity, thermal resistivity and mechanical soil laboratory testing.
Earthwork	The site is forested. Site preparation includes tree removal, grubbing, and stripping topsoil from below substation and access roadway areas. Exposed subgrades should be proof-rolled under the observation of the Geotechnical Engineer. Materials and placement/compaction requirements are presented.
Shallow Foundations	Substation transformers, switchgear and other substation components at-grade may be supported on shallow mat/slab type foundations Allowable bearing pressure = 4,000 psf 2-feet of non-frost-susceptible materials required below exterior mat/slab foundations. Detect and remove zones of fill as noted in the **Earthwork** section in the report.
Deep Foundations	Solar arrays are expected to be supported on steel racking frames supported by driven post foundations. Adfreeze uplift force estimated at 6 kips, based on frost penetration depth of 2.5 feet. Transmission line riser structures may be supported on drilled shaft foundations.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics above can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Geotechnical Engineering Report

Chinook Solar Site
State Route 119
Fitzwilliam, New Hampshire
Terracon Project No. J1175115
April 19, 2019

INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed photovoltaic facility to be located near the intersection of State Route 119 and Fullam Hill Road in Fitzwilliam, New Hampshire. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- Excavation considerations
- Foundation design and construction
- Frost considerations
- Seismic Site Classification per IBC
- Access road recommendations

The geotechnical engineering scope of services for this project included the advancement of eleven test borings for PV arrays and substation to depths ranging from approximately 14 to 50 feet below existing site grades.

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

SITE CONDITIONS

Item	Description
Parcel Information	The project is located near the intersection of State Route 119 and Fullam Hill Road in Fitzwilliam, New Hampshire. The site area is approximately 190 acres and is irregularly shaped. Access to the site is via an unpaved road off Fullam Hill Road and the transmission line easement east of the site. The site is broken into sub-section areas designated on the plans provided with the RFP entitled “Chinook 1 North Concept” and “Chinook 1 South Concept” prepared by Tighe & Bond, dated 08/08/2017. The approximate center of the site is located at: ■ Latitude 42.7671° ■ Longitude -72.1014° See **Site Location**
Existing Improvements	The Chinook Solar site is mostly unimproved raw land and forested, with cleared areas. An unpaved road provides access to the southern half of the site. The plans indicate delineated wetland areas with vernal pools in portions of the site.
Existing Topography (from Earth Point Topo Map)	Gently sloping downward from east to west. Based on topographic mapping, site grades vary from approximately Elevation (El) 1,220 feet to El 1,080 feet.

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
Proposed Development	The project consists of a 40 MW DC (30MW AC) photovoltaic (PV) electric power plant constructed on five sub-section parcels as shown on plans provided with the RFP entitled “Chinook 1 North Concept” and “Chinook 1 South Concept” prepared by Tighe & Bond, dated 08/08/2017. The power plant will consist of fixed mount solar panels installed on pile supported steel racking frames. Appurtenances associated with an interconnection substation are also shown on the plans. Interconnection is proposed to tap one of the two115kV transmission lines immediately east of the site.

Item	Description
Maximum Loads	Structural loads were not provided, but have been estimated based on our experience on projects using fixed tilt rack systems: ■ Downward: 4 kips ■ Uplift: 2.5 kips (does not include frost heave load) ■ Lateral: 3.0 kips Substation structure loads were not available.
Grading/Slopes	Minimal changes to existing site grades are anticipated. Arrays are expected to follow existing topography.
Estimated Start of Construction	Unknown at this time.

GEOTECHNICAL CHARACTERIZATION

Geology

Natural Resources Conservation Service (NRCS) Soil Survey Geographic Database (SSURGO) mapping shows soil parent materials as glacial meltout till derived from granite and gneiss. SSURGO mapping shows bedrock as part of the Concord Granite formation, with **exposed bedrock** in the southern portion of the site.

Recorded seismic events date back to a violent earthquake in the St. Lawrence Valley on June 11, 1638. Several earthquakes have been felt throughout New Hampshire since then including an earthquake (estimated MM IV, comparable to approximately Richter Scale 4.0) on October 22, 1905 near Newport, Vermont and a minor earthquake at Berlin, New Hampshire on April 25, 1928. A strong earthquake with estimated Richter Scale magnitude of 5.8 occurred near Lake Ossippee, New Hampshire on December 20, 1940.

The site is located in Cheshire County, New Hampshire within the jurisdiction of the Southwest Region Planning Commission (SWRPC). The SWRPC website did not indicate significant flood hazards in the vicinity of the site. We also checked the Federal Emergency Management Agency (FEMA) website for the Town of Fitzwilliam Flood Insurance Rate Maps. The site was identified as “Area of Minimal Flood Hazard”.

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 the **Exploration Results** section and the GeoModel can be found in the **Figures** section.

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
01	Sand	Poorly Graded Sand (SP), Poorly Graded Sand with Gravel (SP), Poorly Graded Sand with Silt and Gravel (SP-SM)
02	Silty Sand	Silty Sand (SM), Silty Sand with Gravel (SM)
03	Boulders	Boulder zone likely in till
04	Bedrock	Bedrock

Groundwater Conditions

The boreholes were observed while drilling for the presence and level of groundwater. The water levels observed in the boreholes can be found on the boring logs in **Exploration Results** and are summarized below.

Boring Number	Estimated Ground Surface Elevation (feet-NAVD) [2]	Approximate Depth to Groundwater while Drilling (feet) [1]	Approximate Groundwater Elevation While Drilling (feet- NAVD) [2]
B-1	1,106	6	1,100
B-2	1,194	3	1,191
B-3	1,144	5	1,139
B-4	1,203	2	1,201
B-5	1,126	2	1,124
B-6	1,194	5	1,189
B-7	1,191	5	1,186
B-8	1,154	2	1,152
B-9	1,113	3	1,110
B-10	1,162	10	1,152
B-11	1,066	5	1,061

1. Below ground surface
2. Elevations were estimated by interpolating between ground surface contours on the plans provided

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

In-Situ Soil Resistivity

Fourteen in-situ soil resistivity tests were performed for a total of seven pairs of lines. Soil electrical resistivity data were obtained in accordance with ASTM G57 Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method. At each location two perpendicular lines were tested, one running roughly north-south and the other running east-west with "a" spacing of 2, 5, 10, 20, and 50 feet in the array area and 2, 5, 10, 20, 50, 100, 200, and 300 feet in the planned substation area.

We included the field data sheets in the **Exploration Results** section. Locations of the survey lines are shown on the **Exploration Plan**.

Field resistivity values may vary depending upon the season, precipitation, and temperature, and subsurface factors such as variations in soil density and degree of compaction, presence of coarse gravel and cobbles, and soil constituent solubility. Field resistivity values should be evaluated based on the measured data in conjunction with published values for the material.

Corrosion Potential

The table below lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, Red-Ox potential, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground construction materials. Based on American Concrete Institute (ACI) 318-14 Building Code and Commentary Table 19.3.1.1 and Table 19.3.2.1, it is our interpretation the exposure class is S0 and no restriction on cement type is applicable.

Corrosivity Test Results Summary							
Exploration	**Sample Depth (feet)**	**Soil Description**	**Soluble Sulfate (mg/kg)**	**Soluble Chloride (mg/kg)**	**Red-Ox (mV)**	**Electrical Resistivity (Ω-cm)**	**pH**
B-1	2-5	Silty Sand with Gravel	43	43	+672	24,250	7.85
B-6	2-5	Silty Sand	38	53	+682	36,860	7.94

Corrosivity Test Results Summary							
Exploration	**Sample Depth (feet)**	**Soil Description**	**Soluble Sulfate (mg/kg)**	**Soluble Chloride (mg/kg)**	**Red-Ox (mV)**	**Electrical Resistivity (Ω-cm)**	**pH**
B-9	2-5	Silty Sand trace Gravel	63	53	+678	57,230	7.66

It is important to note groundwater was encountered at depths ranging from 2 to 10 feet. Pile foundations for the solar arrays will be located at a depth where portions of the piles may be submerged. The pile designer should take into account the potential for encountering fluctuating groundwater when considering corrosion.

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 American Society of Civil Engineers (ASCE) 7-10.

Description	Value
2015 International Building Code Site Classification	D[2]
Site Latitude	42.7671°
Site Longitude	-72.1014°
S_{DS} Spectral Acceleration for a Short Period [3]	0.286g
S_{D1} Spectral Acceleration for a 1-Second Period [3]	0.110g

1. Seismic Site Classification in general accordance with the *2015 International Building Code*, which refers to ASCE 7-10.
2. The 2015 International Building Code (IBC) uses a site profile extending to a depth of 100 feet for seismic site classification. Borings at this site were extended to a maximum depth of 50 feet. For the purpose of this analysis, it was assumed very dense soil or bedrock extends below the boring depth to 100 feet. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.
3. These values were obtained using online seismic design maps and tools provided by the USGS (http://earthquake.usgs.gov/hazards/designmaps/).

LIQUEFACTION

Based on the depth to groundwater, relative density and composition of site soils, it is our opinion the soils below the site are not susceptible to liquefaction.

FROST CONSIDERATIONS

The soils on this site are frost susceptible, and small amounts of water can affect the performance of the slabs on-grade. Exterior slabs should be anticipated to heave during winter months. If frost action needs to be eliminated in critical areas, we recommend the use of non-frost susceptible (NFS) fill or structural slabs (for instance, structural stoops in front of building doors). 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 the building and slabs, and toward the site storm drainage system
- Install drains around the perimeter of the building stoops, 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 the 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
- Place NFS materials in critical sidewalk areas

As an alternative to extending NFS fill to the full frost depth of 4 feet, consideration can be made to placing extruded polystyrene or cellular concrete under a buffer of at least 2 feet of NFS material.

PV SOLAR ARRAY FIELD

Geotechnical Overview

The site appears suitable for the proposed solar development based upon geotechnical conditions encountered in the borings provided the recommendations provided in this report are implemented during design and construction.

The native silty sand with gravel typically exhibited medium to very dense relative density throughout the exploration depths and contained various amounts of fine-grained soils. The very dense sands with N-values of 40 or more will likely encounter refusal to the driving of the typical steel piles. Additionally, the exploration logs indicate boulders embedded within the deposit at varying depths in several borings. Boring refusal, presumably on bedrock, was encountered in the south end of the site at depths of 10 to 16 feet in B-8 through B-11. B-8 also encountered a boulder at 4 feet. It should be stressed that borings were widely spaced across the site. Bedrock may potentially be encountered at shallower depths in areas between the borings. Groundwater was encountered in the borings at depths of about 2 to 10 feet below the ground surface, while drilling. The silty and with gravel soils encountered at the site are considered frost susceptible.

The near surface silty sand with gravel may become unstable with typical earthwork and construction traffic, especially after precipitation events. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year (typically May to October). If grading is performed during the winter months (typically November to April), an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

Based on the geotechnical engineering analyses, subsurface exploration, and laboratory test results, the proposed arrays may be supported on a driven pile system combined with pre-drilling in areas where very dense sands and bedrock are less than 20 feet.

The **Deep Foundations** section addresses support of the solar arrays using either driven posts or pre-drilled and grouted posts and provides alternative recommendations for ground screws. The **Slab on Grade or Mat** section addresses slab-on-grade/mat support of ancillary structures.

An aggregate surfaced section is recommended for roadways on the site. The **Access Roadways** section addresses the design of aggregate surfaced systems.

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

Deep Foundations

Introduction

The Chinook solar site presents cost considerations for supporting the solar panels on driven pile foundations (typical W6x9 steel sections) due to obstructions. The native silty sand with gravel typically exhibited medium to very dense relative density throughout the exploration depths and the exploration logs indicate boulders embedded within the deposit at several locations. The very dense sands and obstructions in the overburden soil may require pre-drilling post holes to facilitate driving. Pre-drilling will negatively affect the allowable skin friction. Should pre-drilling through boulders be required, the boreholes should be grouted to develop adequate uplift and lateral capacity.

Preliminary soil resistance parameters and anticipated pile embedment lengths are recommended in the following sections. However, we recommend a load testing program to finalize design embedment lengths. The load testing program should test various combinations of embedment conditions.

Preliminary Post Axial Capacity

The panels may be supported on driven steel posts, which should be structurally designed to resist compression, uplift, and bending forces. Capacity in the freeze-thaw zone should be neglected.

The following design parameters have been estimated based on static pile analysis for the W6x9 post pile typically used for solar array support. However, note that conventional pile analyses typically underestimate the capacity of post piles used in solar arrays, and the most effective means of confirming pile capacities for tension, compression, or lateral loads is through pile load tests. Accordingly, full-scale pull-out testing should be performed once the racking system vendor has been selected. The following design parameters may be used for preliminary design.

Array Designation	Embedment Depth (ft.-bgs)	W6x9 Uplift and Compression Unit Skin Friction (psf/ft)	W6x9 End Bearing (kips)
All Arrays	5 to 7	110	1.1
	8 to 12	145	1.5
	13 to 15	145	2.0
	16 to 20	145	2.4

Notes:

1. The pile capacities listed above are considered ultimate skin and end bearing values. We recommend a factor of safety of at least 2 to arrive at allowable values.
2. Skin friction within the frost zone (0 to 4 feet) shall be neglected.
3. Design embedment depths should be based on static pile analysis. The depths provided in this table should be considered preliminary until completion of pile load testing. The results from pile load tests may indicate different embedment depths will be required to accommodate proposed loads.

The above unit skin friction values are to be used in the following equation to obtain the ultimate uplift or compression load capacity of a pile:

$$Q_{ult} = \frac{1}{2} H^2 \times P \times q_s$$

Q_{ult} = Ultimate uplift or compression capacity of post (lbs.)
H = Depth of embedment of pile (ft) minus the frost zone depth
P = Perimeter area/ft. of pile. (i.e. W6x9 = 1.64 ft.)

q_s = Unit skin friction per depth per table above (psf/ft.)

The axial capacity of the steel posts 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 post length is frost depth and resulting adfreeze pressure. The site soils are silty and frost susceptible and therefore should be considered for contributing to adfreeze uplift loads.

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 post and exerts an uplift force due to the adfreeze stress developed around the surface area of the post. The amount of upward force depends on the following:

1. The thickness of ice lenses formed in the seasonal frozen ground
2. The bond between the steel post surface and the frozen ground
3. The surface area of the steel post in the seasonally frozen ground

Research on adfreeze has been performed by the Ontario Ministry of Transport and presented in the Canadian Foundation Engineering Manual (CFEM). The CFEM concludes high values of adfreeze pressure should be applied, particularly for smaller size steel posts and silty soils. Specifically, the CFEM suggests using adfreeze values of approximately 1,500 psf (10.4 psi) for steel piles in clayey soils and may be higher for silty soils due to easier movement of water within the soil. Frost penetration has been estimated at 4 feet. Frost is not typically engaged in the full depth of frost penetration. Therefore, to determine uplift loading due to adfreeze, we suggest using an adfreeze depth of 2.5 feet with an adfreeze pressure of 10.4 psi. For a W6x9 steel post, the estimated uplift force due to adfreeze would be approximately 6.0 kips.

Uplift forces will govern the design and length of the driven or drilled piles, therefore uplift will be the primary factor in foundation costs. 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.

Preliminary Parameters for Lateral Capacity

The parameters in the following table can be used for preliminary analysis of the lateral capacity of either driven or pre-drilled and grouted steel posts for support of solar panel arrays.

Material and Depth (feet-bgs)	LPile (P-y) Curve Soil Model	Effective Unit Weight, γ (pcf)[1]	Friction Angle, ϕ (degrees)	Soil Modulus, k (pci)	Uniaxial Compressive Strength, q_u (psi)
Glacial Till: 0 to 5 feet	Sand (Reese)	120	34	100	Not Applicable
Glacial Till: 5 to 5 feet	Sand (Reese)	63	34	100	Not Applicable
Bedrock – Granite: 15 to 20 feet	Strong Rock (Vuggy Limestone)	103	Not Applicable	Not Applicable	10,000

1. Assumes the groundwater is at a depth of 5 feet. Therefore, the soil's submerged effective unit weight should be used below 5 feet.

The above indicated soil parameters have no factor of safety and may be used to analyze suitability of the proposed section and serviceability requirements. These parameters are based on correlations with SPT results, published values, and our experience with similar soil types. Full-scale pile load testing should be completed to provide parameters for final design.

Pre-Drilled and Grouted Post Considerations

At the south end of the site, where shallow bedrock is encountered, we recommend predrilling minimum 8-inch diameter holes into bedrock and grouting the steel posts in bedrock. For posts grouted in bedrock, uplift capacity will primarily depend on the post to grout bond strength, since the grout to bedrock bond strength will be greater. Design bond strength between grout and steel posts is recommended to be 40 psi (5,760 psf) for grout with a 28-day strength of 2,000 psi. Post uplift capacities should be field verified by load testing piles prior to installing production piles. We anticipate rock sockets of at least 2 feet will be necessary to resist uplift loads. Pile uplift capacities and lengths must be field verified by load testing piles prior to installing production piles.

The frost zone should not be grouted to avoid increased adfreeze loading. Grouted steel posts should be structurally designed to resist compression, uplift, and bending forces. For design purposes, available resistance in the freeze-thaw zone should be neglected, except for lateral capacity. Post uplift capacities and lengths must be field verified by load testing posts prior to installing production posts.

Ground Screw Foundations

Within areas of shallow or exposed bedrock, the photovoltaic panels may be supported on a ground screw system (Terrasmart, or similar) deriving support from the overburden soils or bedrock. The ground screws should be structurally designed to resist vertical loading and uplift and bending forces. The upper 4 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 relative density/consistency of the soil against which the screw is pushed by the horizontal load.

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. Ground screws will need to penetrate deep enough to achieve the required resistance, or as discussed above for steel posts, installed with larger diameter helixes where refusal is too shallow. The designer and contractor should keep these aspects in mind in completing the design and choosing installation methods.

Slab on grade or 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 and mats assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure. This also includes the positive drainage of the aggregate base beneath the slab/mat.

Design Parameters

Item	Description
Slab-on-Grade or Mat Support [1]	Minimum 24 inches of NFS Fill compacted to at least 95% of ASTM D 1557
Allowable Bearing Capacity,ksf [2]	4.0
Settlement, (inches)	
Total	< 1.0
Differential	0.5
Estimated Modulus of Subgrade Reaction [3]	200 pounds per square inch per inch (psi/in) for point loads

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 **Earthwork**, and the floor slab support as noted in this table. The modulus recommended is for compacted Structural Fill over very dense glacial till and value may increase with increasing slab size. No adjustment is necessary for mat size.

Saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks that develop should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks that occur beyond the length of the structural dowels. The Structural Engineer should account for this potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

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 will include clearing and grubbing and grading for access roads. Grading plans were not available at the time of this report. Cut and fill depths are unknown. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality control criteria as necessary to prepare the site subsurface conditions consistent with the conditions considered in our geotechnical engineering evaluation for foundations, slabs, and aggregate surfaced roadways.

Site Preparation

The site is partially wooded. Prior to placing fill, existing vegetation, stumps, and root mat should be removed. Complete stripping of the forest mat and topsoil should be performed in the proposed equipment slab areas, access roadways, and staging areas.

Foundation, slab/mat and roadway inorganic subgrades should be proof-rolled 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 proof-roll should be delineated and subsequently addressed by the Geotechnical Engineer.

Based on the outcome of 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 about 1 to 1.5 feet.
- **Granular Fill** - The use of Crushed Stone or Structural Fill could be considered to improve subgrade stability. Typical undercut depths would range from about 0.5 foot to 2 feet. 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 sections **Fill Material Types** and **Fill Compaction Requirements** of this report. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering controlled conditions in accordance with the project specifications.

Fill Material Types

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

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Structural Fill [2]	GW, GW-GM, GP, SW, SW-SM, SP,SP-SM	All locations and elevations. Excavated native material is unsuitable for reuse as Structural Fill.
General Fill [3]	GM, GC, SM, SC	General Fill may be used for general site grading. General Fill should not be used under settlement or frost-sensitive structures. Excavated native material may be reused as General Fill, provided it is free of organic matter and can be adequately compacted.

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Non-Frost Susceptible (NFS) Fill[4]	GW, GP, SW, SP	Under slabs-on-grade, substation pads, or as raise-in-grade fill to reduce potential effects of frost action.
Crushed Stone	GP	For use on wet subgrades, as a replacement for Structural Fill and NFS Fill (if desired). Should be uniform ¾-inch angular Crushed Stone wrapped in a geotextile separation fabric (Mirafi 140N, or similar).
NHDOT 304.33 Crushed Aggregate for Shoulders	GW, GP	For unpaved access road aggregate base.

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 a frozen subgrade.
2. Imported Structural Fill should meet the following gradation:

Percent Passing by Weight

Sieve Size	Structural Fill
6″	100
3″	70 – 100
2″	(100)*
¾″	45 – 95
No. 4	30 – 90
No. 10	25 – 80
No. 40	10 – 50
No. 200	0 – 10

* Maximum 2-inch particle size within 12 inches of concrete elements.

3. General Fill should have a maximum particle size of 6 inches and no more than 25 percent by weight passing the US No. 200 sieve.
4. NFS Fill should consist of Structural Fill, except the percent passing No. 200 sieve should be no more than 5%.

Fill Compaction Requirements

Structural and General Fill should meet the following compaction requirements.

Item	Structural Fill
Maximum Lift Thickness	8 inches or less in loose thickness
Minimum Compaction Requirements [1, 2]	At least 95% of the material's maximum Modified Proctor dry density (ASTM D1557)
Water Content Range [1]	Workable moisture levels

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557, Method C).
2. 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.

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.

Earthwork Construction Considerations

Shallow excavations, for proposed access roads, collector trenches, and incidental earthwork are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of slabs. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over, or adjacent to, construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted, prior to slab construction.

As a minimum, excavations should be performed in accordance with Occupational Safety and Health Administration (OSHA) 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with applicable local, and/or state regulations. In accordance with OSHA, soil is classified as Type B, previously disturbed soil. Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

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 vegetation and topsoil, 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 2,500 square feet of compacted fill in the substation areas and 5,000 square feet in access roads. One density and water content test should be performed for every 100 linear feet of compacted utility trench backfill.

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 Roadways

Approximate axle passes for the project were not provided, however we anticipate traffic to include construction machinery and delivery traffic in the short term and heavy support vehicles throughout the life of the lease. We equate this to be approximately 15,000 18-kip axle passes for the array support roads and 22,000 18-kip axle passes for the construction roads. 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.

Aggregate-surfaced roadway sections below are based on the American Association of State Highway and Transportation Officials (AASHTO) Design of Pavement Structures Manual (1993) for gravel road design with an allowable serviceability loss of 1.0 and rutting of 1.0 inch. We have estimated in-situ California Bearing Ratio (CBR) values based on our experience with similar soil conditions. We have assumed CBR values for the following conditions: CBR of 5 for compacted subgrades consisting of silty sand native soils.

Based on the above assumptions, we have provided the following minimum aggregate thicknesses for the access roadways.

Road Type	Recommended Road Section Thickness (inches)	
	Array Support Roads	**Main/Construction Roads**
Aggregate-Surfaced	6 inches of NHDOT 304.33 Crushed Aggregate for Shoulders over 12 inches of scarified, moisture conditioned, and compacted native soils	8 inches of NHDOT 304.33 Crushed Aggregate for Shoulders over 12 inches of scarified, moisture conditioned, and compacted native soils

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.

In order for the above recommendations to be valid and to maintain roadway performance, surface drainage of the roadway and subgrade should be provided and maintained. Where subgrade conditions are allowed to become wetted or saturated, the subgrade CBR would be less than the estimated value and a reduced performance and possible repair should be expected. The roadway should be sloped to provide surface water drainage at all times. Water should not be allowed to remain within the roadway section and subgrade soils. In addition, the subgrade soils should be prepared in accordance with the **Earthwork** section.

It should be emphasized that all aggregate surfaced roadways, regardless of the thickness or practical subgrade preparation measures, will require on-going maintenance and repairs to keep them in a serviceable condition. It is not practical to design an unpaved aggregate section of sufficient thickness that requires no on-going maintenance. This is due to the porous nature of the NHDOT 304.33 Crushed Aggregate that will allow precipitation and surface water to infiltrate and soften the subgrade soils, and the limited near surface strength of unconfined crushed stone that makes it susceptible to rutting.

When potholes, ruts, depressions or yielding subgrades develop, they must be repaired prior to applying additional traffic loads. Typical repairs could consist of placing additional NHDOT 304.33 Crushed Aggregate in ruts or depressed areas and, in some cases, complete removal of Crushed Aggregate surfacing, repair of unstable subgrade, and replacement of the Crushed Aggregate surfacing. Potholes and depressions should not be filled by blading adjacent ridges or high areas into the depressed areas. New material should be added to the depressed areas as they develop. Failure to make timely repairs will result in more rapid deterioration of the access roadways, making more extensive repairs necessary.

SUBSTATION AND TRANSMISSION STRUCTURES

Geotechnical Overview

The site appears suitable for the proposed interconnection substation based upon geotechnical conditions encountered in the borings provided the recommendations provided in this report are implemented during design and construction.

The native silty sand with gravel typically exhibited medium to very dense relative density throughout the exploration depths and contained various amounts of fine-grained soils. Borings B-6 and B-7 were drilled in the vicinity of the interconnection substation. Both borings were drilled to depths of 50 feet. The exploration logs indicate boulders embedded within the deposit in B-7. Silty sand soils encountered at the site are considered frost susceptible.

The near surface silty sand with gravel soil may become unstable with typical earthwork and construction traffic, especially after precipitation events. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year (typically May to October). If grading is performed during the winter months (typically November to April), an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The **Deep Foundations** section addresses support of the transmission line riser structures. The **Slab on Grade or Mat** section addresses slab-on-grade/mat support of transformers, switchgear, and ancillary structures.

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

Deep Foundations

Drilled Pier Design Recommendations

Transmission line riser structures may be supported on drilled pier foundations. Design recommendations for drilled pier foundations are presented below

Drilled Pier Axial Capacity			
Minimum Pier Embedment Depth Below Ground Surface (feet)	**Material**	**Allowable Skin Friction (psf)**	**Allowable End Bearing Pressure (psf)**
0 to 4	Frost zone	Neglect	Neglect
4 to 10	Glacial Till	900	6,000
>10	Glacial Till	1,200	30,000

Contribution to pier capacity from soils within the active zone frost depth of 4 feet should be ignored. We anticipate the drilled pier will be designed to resist tension loads and, therefore, reinforcing steel should be installed throughout its entire length. Technical specifications requiring material and installation detail submittals, proof of experience in either drilled pier installation, concrete placement methods, and the use and removal of temporary steel casing should be prepared.

Factors of safety of 2 and 3 were included in the allowable skin friction and end bearing values, respectively. Design of the deep foundations should be completed by the Structural Engineer using the geotechnical engineering design criteria provided herein. The required foundation size and depth should be determined based upon analyses for vertical loads and overturning moments.

Drilled Pier Lateral Loading Recommendations

The following table lists input values for use in LPile analyses. LPile estimates values of k based on strength; however, non-default values of k should be used where provided. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters.

Material and Depth (feet-bgs)	LPile (P-y) Curve Soil Model	Effective Unit Weight, γ (pcf)[1]	Friction Angle, ϕ (degrees)	Soil Modulus, k (pci)	Uniaxial Compressive Strength, q_u (psi)
Glacial Till: 0 to 5 feet	Sand (Reese)	120	34	100	Not Applicable
Glacial Till: 5 to 5 feet	Sand (Reese)	63	34	100	Not Applicable
Bedrock – Granite: 15 to 20 feet	Strong Rock (Vuggy Limestone)	103	Not Applicable	Not Applicable	10,000

1. Assumes the groundwater is at a depth of 5 feet. Therefore, the soil's submerged effective unit weight should be used below 5 feet.

The Structural Engineer should evaluate the moment capacity of the piers as part of its structural evaluation. Pier lateral capacities should be reduced due to group effects using reduction factors given below:

Pile Spacing	2.5D	6D
Reduction Factor	0.67	1.0

For intermediate spacing, reduction factor can be calculated using linear interpolation.

All shafts should be reinforced to full-depth for the applied axial, lateral and uplift stresses imposed. For this project, use of a minimum shaft diameter of 18 inches is recommended for the foundations.

Design of the deep foundations should be completed by the structural engineer using the geotechnical engineering design criteria provided herein. The required foundation size and depth should be determined based upon analyses for vertical loads, lateral loads and overturning moments.

Drilled Pier Construction Considerations

Drilled piers should be aligned vertically. The drilling method or combination of methods selected by the contractor should be submitted for review by Terracon, prior to mobilization of drilling equipment. Temporary casing may be required to reduce the likelihood of caving. If piers extend below the groundwater table, drilling mud may also be required to stabilize the hole. Concrete should be placed by directing the concrete down the center of the shaft in order to reduce the likelihood of hitting the reinforcing steel and segregating. Groundwater should be removed prior to placing concrete.

Drilling of foundations to design depths should be possible with conventional drilling equipment using single flight power augers, except in areas with shallow rock. Areas where shallow rock is encountered may require rock drilling equipment. However, if caving soils are encountered, temporary casing or drilling slurry will likely be required in order to advance the drilled shafts to design depth. Temporary casing should also be used whenever shafts are installed adjacent to any existing structures or improvements, to reduce the potential for ground loss and movement due to drilled shaft excavation. Water, if encountered, should be removed from each shaft hole prior to concrete placement. Casing should be installed for the full shaft depth if downhole inspection and clean out is required. Shaft concrete should be placed immediately after completion of drilling and cleaning. If shaft concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes.

Where casing is used for drilled shaft construction, it should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent infiltration of water or the creation of voids in the concrete. The concrete should have a relatively high fluidity when placed in cased holes or through a tremie. Concrete with slump in the range of 6 to 8 inches is recommended.

Free-fall concrete placement in drilled shaft excavations will only be acceptable in dry holes and if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be minimized, is recommended.

Shaft bearing surfaces should be cleaned prior to concrete placement. A representative of the geotechnical engineer should inspect the bearing surface and shaft configuration. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

The drilled shaft installation process should be performed under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil and groundwater conditions encountered, consistency with expected conditions, and details of the installed shaft.

Slab on grade or mat

Transformers, switchgear, and other pieces of equipment for the interconnection substation will be supported on slabs or mats, constructed near the finished grade surface. Design parameters for slabs and mats assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure. This also includes the positive drainage of the aggregate base beneath the slab/mat.

Design Parameters

Item	Description
Slab-on-Grade or Mat Support [1]	Minimum 24 inches of NFS Fill compacted to at least 95% of ASTM D 1557
Allowable Bearing Capacity,ksf [2]	4.0
Settlement, (inches)	
Total	< 1.0
Differential	0.5
Estimated Modulus of Subgrade Reaction [3]	200 pounds per square inch per inch (psi/in) for point loads

4. 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.
5. Allowable bearing capacity developed using factor of safety of 3.0.
6. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the floor slab support as noted in this table. The modulus recommended is for compacted Structural Fill over very dense glacial till and value may increase with increasing slab size. No adjustment is necessary for mat size.

Saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks that develop should be sealed with a water-proof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks that occur beyond the length of the structural dowels. The Structural Engineer should account for this potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

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 will include clearing and grubbing and grading for the substation. Grading plans were not available at the time of this report. Cut and fill depths are unknown. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality control criteria as necessary to prepare the site subsurface conditions consistent with the conditions considered in our geotechnical engineering evaluation for slab/mat foundation slabs.

Site Preparation

The substation site is partially wooded. Prior to placing fill, existing vegetation, stumps, and root mat should be removed. Complete stripping of the forest mat and topsoil should be performed in the proposed equipment slab areas, access roadways, and staging areas.

Foundation slab/mat inorganic subgrades should be proof-rolled 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 proof-roll 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 about 1 to 1.5 feet.
- **Granular Fill** - The use of Crushed Stone or Structural Fill could be considered to improve subgrade stability. Typical undercut depths would range from about 0.5 foot to 2 feet. 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 sections **Fill Material Types** and **Fill Compaction Requirements** of this report. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering controlled conditions in accordance with the project specifications.

Fill Material Types

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

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Structural Fill [2]	GW, GW-GM, GP, SW, SW-SM, SP,SP-SM	All locations and elevations. Excavated native material is unsuitable for reuse as Structural Fill.
General Fill [3]	GM, GC, SM, SC	General Fill may be used for general site grading. General Fill should not be used under settlement or frost-sensitive structures. Excavated native material may be reused as General Fill, provided it is free of organic matter and can be adequately compacted.

Fill Type [1]	USCS Classification	Acceptable Location for Placement
Non-Frost Susceptible (NFS) Fill[4]	GW, GP, SW, SP	Under slabs-on-grade, substation pads, or as raise-in-grade fill to reduce potential effects of frost action.
Crushed Stone	GP	For use on wet subgrades, as a replacement for Structural Fill and NFS Fill (if desired). Should be uniform ¾-inch angular Crushed Stone wrapped in a geotextile separation fabric (Mirafi 140N, or similar).
NHDOT 304.33 Crushed Aggregate for Shoulders	GW, GP	For substation yard aggregate.

5. 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 a frozen subgrade.
6. Imported Structural Fill should meet the following gradation:

Percent Passing by Weight

Sieve Size	Structural Fill
6″	100
3″	70 – 100
2″	(100)*
¾″	45 – 95
No. 4	30 – 90
No. 10	25 – 80
No. 40	10 – 50
No. 200	0 – 10

* Maximum 2-inch particle size within 12 inches of concrete elements.

7. General Fill should have a maximum particle size of 6 inches and no more than 25 percent by weight passing the US No. 200 sieve.
8. NFS Fill should consist of Structural Fill, except the percent passing No. 200 sieve should be no more than 5%.

Fill Compaction Requirements

Structural and General Fill should meet the following compaction requirements.

Item	Structural Fill
Maximum Lift Thickness	8 inches or less in loose thickness
Minimum Compaction Requirements [1, 2]	At least 95% of the material's maximum Modified Proctor dry density (ASTM D1557)
Water Content Range [1]	Workable moisture levels

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557, Method C).
2. 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.

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.

Earthwork Construction Considerations

Shallow excavations, for proposed access roads, collector trenches, and incidental earthwork are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of slabs. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over, or adjacent to, construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted, prior to slab construction.

As a minimum, excavations should be performed in accordance with Occupational Safety and Health Administration (OSHA) 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with applicable local, and/or state regulations. In accordance with OSHA, soil is classified as Type B, previously disturbed soil. Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

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 vegetation and topsoil, 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 2,500 square feet of compacted fill in the substation areas and 5,000 square feet in access roads. One density and water content test should be performed for every 100 linear feet of compacted utility trench backfill.

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.

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

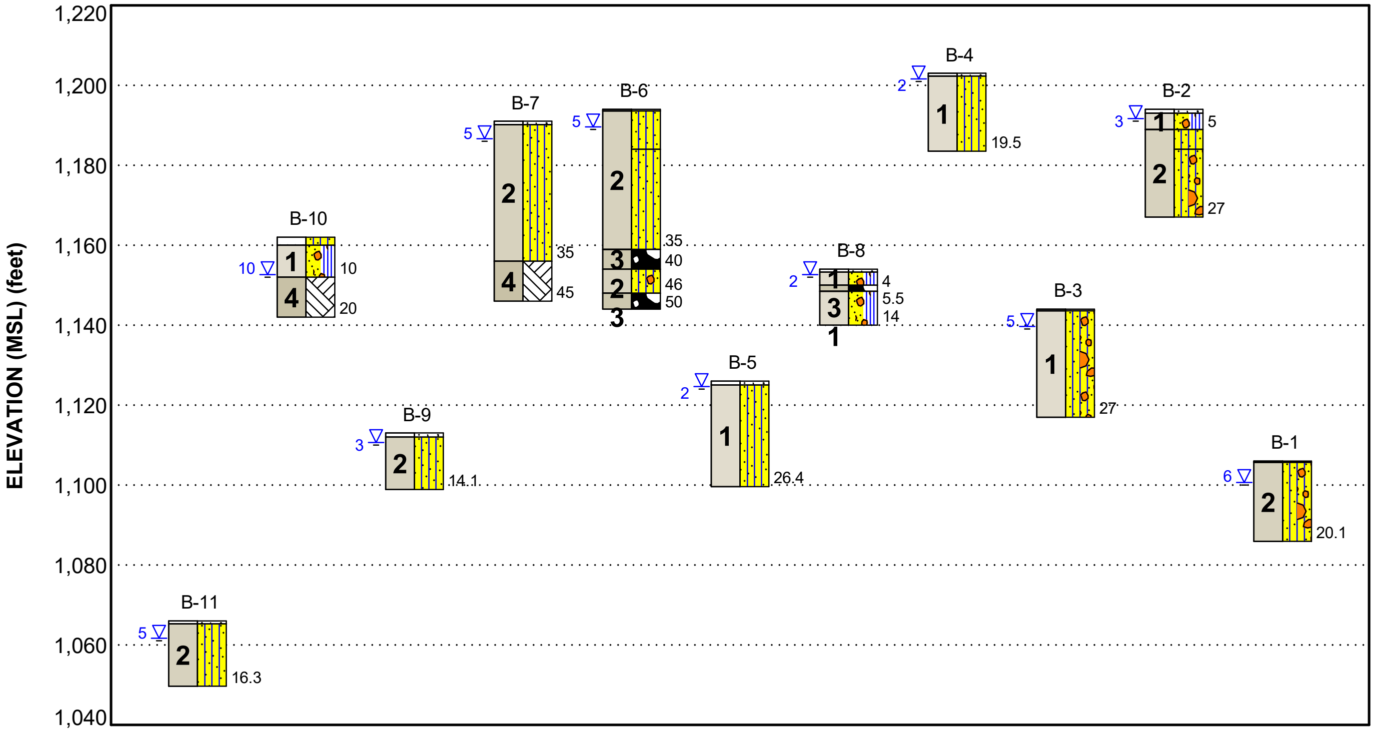
Contents:

GeoModel

GEOMODEL

NextEra Chinook Solar ■ Fitzwilliam, NH
4/19/2019 ■ Terracon Project No. J1175115

Terracon GeoReport



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

Model Layer	Layer Name	General Description
1	**Sand**	Poorly Graded Sand (SP), Poorly Graded Sand with Gravel (SP), Poorly Graded Sand with Silt and Gravel (SP-SM)
2	**Silty Sand**	Silty Sand (SM), Silty Sand with Gravel (SM)
3	**Boulder**	Boulder
4	**Bedrock**	Bedrock

LEGEND

- Topsoil
- Silty Sand
- Silty Sand with Gravel
- Boulders and Cobbles
- Poorly-graded Sand with Silt and Gravel
- Bedrock

- First Water Observation
- Second Water Observation
- Final Water Observation

Groundwater levels are temporal. The levels shown are representative of the date and time of our exploration. Significant changes are possible over time.
Water levels shown are as measured during and/or after drilling. In some cases, boring advancement methods mask the presence/absence of groundwater. See individual logs for details.

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

The following field exploration program was performed for design of solar foundations for this site.

Number of Borings	Exploration Depth (feet) [1]	Location
9 borings	20 or auger refusal	Arrays
2 borings	50	Substation

1. Below ground surface

Boring Layout and Elevations: Borings were laid out using hand-held global position system (GPS). We use a hand-held GPS to locate borings with a 15± foot horizontal accuracy. Approximate ground surface elevations were estimated based on satellite imagery.

Subsurface Exploration Procedures: We advanced soil borings with a track-mounted drill rig using a combination of cased drive and wash and continuous flight augers (solid stem and/or hollow stem, as necessary, depending on soil conditions). Four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. Soil sampling was performed using split-barrel sampling procedures. The samples were placed in appropriate containers, taken to our soil laboratory for testing, and classified by a geotechnical engineer. In addition, we observed and recorded groundwater levels during drilling and sampling.

A geologist or geotechnical engineer prepared field boring logs as part of standard drilling operations, including sampling depths, penetration distances, and other relevant sampling information. Field logs include visual classifications of materials encountered during drilling, and our interpretation of subsurface conditions between samples. Final boring logs, prepared from field logs, represent the geotechnical engineer's interpretation, and include modifications based on observations and laboratory tests.

Soil Electrical Resistivity Testing

Soil electrical resistivity data were obtained in accordance with ASTM G57 Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method. At each location two perpendicular lines were tested, one running north-south and the other running east-west. Locations of the survey lines are shown on the **Exploration Plan**.

Laboratory Testing

The project engineer reviewed the field data and assigned various laboratory tests to better understand the engineering properties of the various soil and rock strata as necessary for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods are applied as a result 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 tests that will be performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils
- ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort
- ASTM D1140 Standard Test Methods for Determining the Amount of Material Finer than 75-μm (No. 200) Sieve in Soils by Washing
- ASTM D4972 Standard Test Method for pH of Soils
- ASTM C1580 Standard Test Method for Water-Soluble Sulfate in Soil
- ASTM D512 Standard Test Methods for Chloride Ion In Water
- ASTM D5334 Standard Test Method for Determination of Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure

The laboratory testing program may often include examination of soil samples by an engineer. Based on the material's texture and plasticity, we describe and classify the soil samples in accordance with the Unified Soil Classification System.

Rock classification was conducted using locally accepted practices for engineering purposes; petrographic analysis may reveal other rock types. Rock core samples typically provide an improved specimen for this classification. Boring log rock classification was determined using the Description of Rock Properties.

SITE LOCATION AND EXPLORATION PLANS

Contents:

Site Location Plan
Exploration Plan

Note: All attachments are one page unless noted above.

SITE LOCATION

NextEra Chinook Solar ■ Fitzwilliam, NH
April 19, 2019 ■ Terracon Project No. J1175115

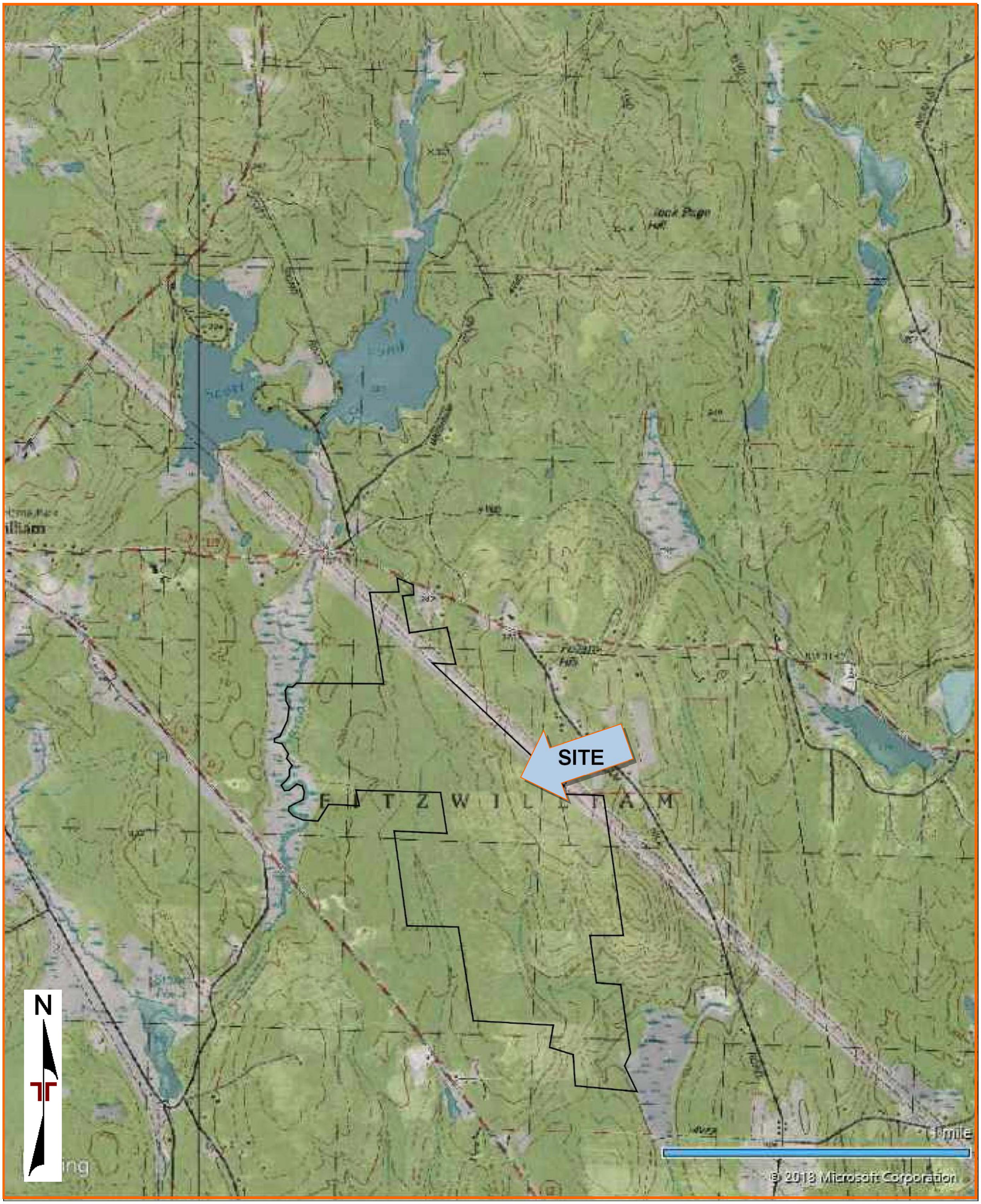


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

TOPOGRAPHIC MAP IMAGE COURTESY OF THE U.S. GEOLOGICAL SURVEY QUADRANGLES INCLUDE: .

EXPLORATION PLAN

NextEra Chinook Solar ■ Fitzwilliam, NH

April 19, 2019 ■ Terracon Project No. J1175115

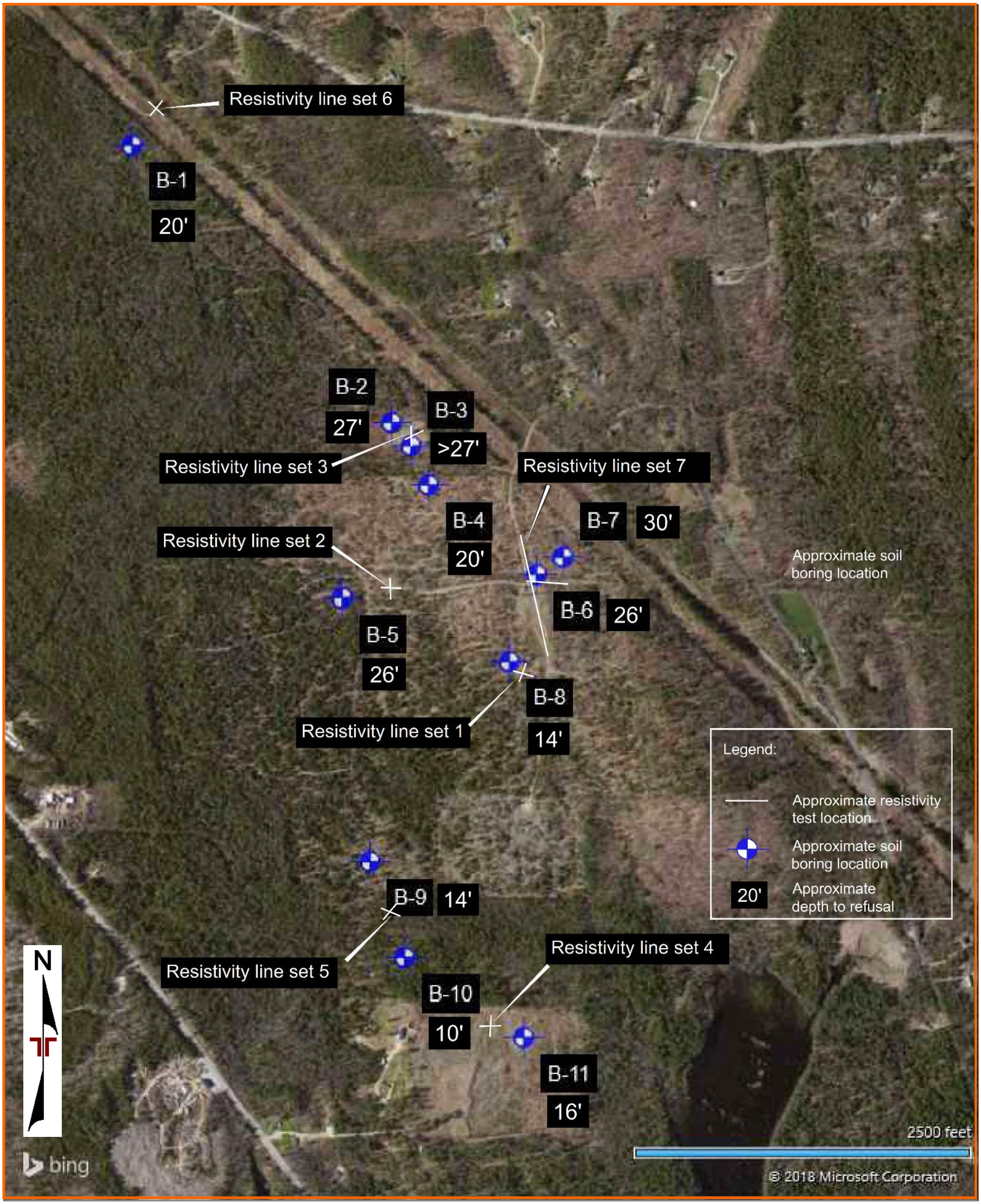


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

AERIAL PHOTOGRAPHY PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

General Notes
Boring Logs (B-1 through B-11)
Grain Size Distribution (2 pages)
Moisture Density Relationship (6 pages)
Field Electrical Resistivity (14 pages)
Corrosivity
Thermal Resistivity (5 pages)

Note: All attachments are one page unless noted above.

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

NextEra Chinook Solar ■ Fitzwilliam, NH

Apr. 19, 2019 ■ Terracon Project No. J1175115

Terracon GeoReport

SAMPLING

Rock Core

Grab Sample

Standard Penetration Test

WATER LEVEL

Water Initially Encountered

Water Level After a Specified Period of Time

Water Level After a Specified Period of Time

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.

FIELD TESTS

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 is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. 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

RELATIVE PROPORTIONS OF SAND AND GRAVEL		RELATIVE PROPORTIONS OF FINES	
Descriptive Term(s) of other constituents	Percent of Dry Weight	**Descriptive Term(s) of other constituents**	**Percent of Dry Weight**
Trace	<15	Trace	<5
With	15-29	With	5-12
Modifier	>30	Modifier	>12

GRAIN SIZE TERMINOLOGY		PLASTICITY DESCRIPTION	
Major Component of Sample	**Particle Size**	**Term**	**Plasticity Index**
Boulders	Over 12 in. (300 mm)	Non-plastic	0
Cobbles	12 in. to 3 in. (300mm to 75mm)	Low	1 - 10
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)	Medium	11 - 30
Sand	#4 to #200 sieve (4.75mm to 0.075mm	High	> 30
Silt or Clay	Passing #200 sieve (0.075mm)		

BORING LOG NO. B-1

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7761° Longitude: -72.1125°

Approximate Surface Elev.: 1106 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.3	4-inches of topsoil, roots, leaf litter, dark brown	1105.5+/-								
2		**SILTY SAND WITH GRAVEL (SM)**, light brown with iron staining, very loose to very dense					4	1-1-2-4 N=3			
							19	25-25-27-36 N=52		21.8	12
		Color change to gray, dense to very dense		5							
					▽		16	12-20-23-30 N=43			
				10			18	11-11-20-25 N=31			
				15			4	50/5"			
	20.1		1086+/-	20			1	50/1"			
		Refusal at 20 Feet									

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *6' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-05-2018

Boring Completed: 10-05-2018

Drill Rig: CME 550

Driller: Terracon/R. Brown

Project No.: J1175115

BORING LOG NO. B-2

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7705° Longitude: -72.1053°

Approximate Surface Elev.: 1194 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)
	1.0	12-inches of topsoil, roots, leaf litter, dark brown	1193+/-
1	5.0	**POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)**, light brown, medium dense to dense	1189+/-
2	10.0	**SILTY SAND (SM)**, trace gravel, olive brown, loose	1184+/-
2	27.0	**SILTY SAND WITH GRAVEL (SM)**, gray, dense to very dense	1167+/-
		Boring Terminated at 27 Feet	

DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
0–2		X	15	2-10-10-3 N=20			
2–4	▽ (3')	X	17	13-22-22-25 N=44			
5–7		X	18	1-1-5-16 N=6			
10–12		X	18	10-71-16-20 N=87			
15–17		X	10	30--38-50/4"			
20–22		X	19	16-18-24-24 N=42			
25–27		X	18	14-14-35-35 N=49			

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *3' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-05-2018	Boring Completed: 10-05-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-3

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.77° Longitude: -72.1047°

Approximate Surface Elev.: 1144 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.4	5-inches of topsoil, roots, leaf litter, dark brown	1143.5+/-								
1		**SILTY SAND WITH GRAVEL (SM)**, olive brown, loose to medium dense				X	15	1-1-3-5 N=4			
						X	10	11-13-15-19 N=28			
				5	▽	X	16	7-7-5-7 N=12			
		Color change to gray at 10 feet, dense to very dense		10		X	20	11-14-19-26 N=33			
				15		X	8	19-35-50/5"			
				20		X	21	12-19-22-20 N=41			
	27.0		1117+/-	25		X	22	9-12-17-15 N=29			

Boring Terminated at 27 Feet

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *5' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-05-2018	Boring Completed: 10-05-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-4

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7692° Longitude: -72.1042°

Approximate Surface Elev.: 1203 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.7	8-inches of topsoil, roots, leaf litter, dark brown	1202.5+/-								
1		**SILTY SAND (SM)**, trace gravel, olive brown, very loose to very dense			▽	X	18	1-1-1-1 N=2			
						X	20	1-13-13-17 N=26			
				5		X	21	12-18-17-17 N=35			
				10		X	12	10-19-30-22 N=49			
				15		X	18	24-30-33-29 N=63			
	19.5		1183.5+/-								
		Refusal at 19.5 Feet									

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *2' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-05-2018	Boring Completed: 10-05-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-5

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7669° Longitude: -72.1067°

Approximate Surface Elev.: 1126 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
		1.0	12-inches of topsoil, roots, leaf litter, dark brown	1125+/-				16	1-1-1-3 N=2			
1			**SILTY SAND (SM)**, trace gravel, olive brown, very loose to very dense			▽ (2')		20	11-10-9-10 N=19		34.9	21
					5			8	5-5-50/4"			
					10			18	10-16-19-14 N=35			
			Color change to gray at 15 feet		15			20	42-40-33-22 N=73			
					20			12	14-18-40-33 N=58			
		26.4		1099.5+/-	25			12	27-32-50/4"			
			Boring Terminated at 26.4 Feet									

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *2' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-04-2018

Boring Completed: 10-04-2018

Drill Rig: CME 550

Driller: Terracon/R. Brown

Project No.: J1175115

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL J1175115 NEXTERA CHINOOK S.GPJ MODELLAYER.GPJ 2/26/19

BORING LOG NO. B-6

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7674° Longitude: -72.1012°

Approximate Surface Elev.: 1194 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.4	5-inches of topsoil, roots, leaf litter, dark brown	1193.5+/-			X	12	1-3-3-5 N=6			
2		**SILTY SAND (SM)**, olive brown, medium dense to dense				X	18	17-18-17-14 N=35		34.2	13
				5	▽	X	12	46-18-10-9 N=28			
	10.0	**SILTY SAND (SM)**, trace gravel, olive brown, dense to very dense	1184+/-	10		X	17	14-15-18-25 N=33			
		Color change to gray at 15 feet		15		X	12	14-15-18-25 N=33			
				20		X	15	14-30-24-50 N=54			
				25		X	21	48-28-50/5"			

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *5' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-03-2018	Boring Completed: 10-03-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-6

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC Juno Beach, FL

SITE: 264 Fullam Hill Rd Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7674° Longitude: -72.1012°

Approximate Surface Elev.: 1194 (Ft.) +/-

MODEL LAYER	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
2	**SILTY SAND (SM)**, trace gravel, olive brown, dense to very dense *(continued)* No recovery in the 30-32 foot sample. Boulder encountered at 34 feet. Advanced roller bit to 35 feet 35.0	1159+/-	30		X	18	21-38-35-50/5"			
3	Cored through a 2.5 foot boulder from 35 to 37.5 feet. Resume split spoon sampling at 40 feet. 40.0	1154+/-	35		X	0	28-39-50/4"			
2	**SILTY SAND WITH GRAVEL (SM)**, gray, very dense Advanced roller bit to 45 feet 46.0	1148+/-	40		X	12	30-38-50/4"			
3	Cored through boulders from 46 to 50 feet. No recovery in core barrel. 50.0	1144+/-	45 / 50							

Boring Terminated at 50 Feet

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

5' While drilling

Terracon

77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-03-2018	Boring Completed: 10-03-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-7

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC Juno Beach, FL

SITE: 264 Fullam Hill Rd Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7677° Longitude: -72.1005°

Approximate Surface Elev.: 1191 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.8	10-inches of topsoil, roots, leaf litter, dark brown	1190+/-				18	1-1-1-12 N=2			
2		**SILTY SAND (SM)**, olive brown, very loose to very dense					13	22-15-13-10 N=28		49.1	15
				5	▽		5	5-10-8-11 N=18			
		Trace gravel at 10 feet		10			20	17-21-25-29 N=46			
		Color change to gray at 15 feet		15			20	15-22-25-29 N=47			
				20			10	11-23-25-22 N=48			
				25			10	10-26-40-40 N=66			

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *5' While drilling*

Terracon

77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-03-2018	Boring Completed: 10-04-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-7

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7677° Longitude: -72.1005°

Approximate Surface Elev.: 1191 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
2		**SILTY SAND (SM)**, olive brown, very loose to very dense *(continued)*									
		No recovery, roller bit from 30 to 35 feet. Advanced NX rock core from 35 to 45 feet.		30			0	50/0"			
	35.0		1156+/-	35							
4		Run 1 Hard, fresh, light gray, medium grained GRANITE, with horizontal to moderately dipping, widely spaced, rough, slightly open joints					52		80		
		Run 2 Similar		40			60		96		
	45.0		1146+/-	45							
		Boring Terminated at 45 Feet									

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

5' While drilling

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-03-2018	Boring Completed: 10-04-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-8

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7656° Longitude: -72.102°

Approximate Surface Elev.: 1154 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.7	8-inches of topsoil, roots, leaf litter, dark brown	1153.5+/-								
1	4.0	**POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)**, light brown, medium dense to very dense	1150+/-		▽	X	12	2-9-5-3 N=14			
						X	14	10-36-32-50/4"			
3	5.5	Cored through boulder from 4 to 5.5 feet. Resume split spoon sampling at 10 feet	1148.5+/-	5		X					
1	14.0	**POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)**, dense	1140+/-	10		X	6	4-18-30-30 N=48			

Refusal at 14 Feet

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *2' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-04-2018	Boring Completed: 10-04-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-9

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7615° Longitude: -72.1059°

Approximate Surface Elev.: 1113 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
		1.0	12-inches of topsoil, roots, leaf litter, dark brown	1112+/-				24	1-1-1-1 N=2			
2			**SILTY SAND (SM)**, trace gravel, olive brown, very loose to dense			∇		20	7-10-11-13 N=21		36.7	18
					5			18	8-13-18-22 N=31			
					10			18	11-22-20-20 N=42			
		14.1		1099+/-				0	50/1"			

Refusal at 14.1 Feet

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

∇ *3' While drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-06-2018	Boring Completed: 10-06-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-10

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7595° Longitude: -72.1049°

Approximate Surface Elev.: 1162 (Ft.) +/-

MODEL LAYER	DEPTH	Description	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	2.0	**SILTY SAND (SM)**, reddish brown, loose	1160+/-				8	2-1-3-3 N=4			
1	10.0	**POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)**, light brown, medium dense	1152+/-	5			22	8-10-15-17 N=25			
							20	5-8-12-12 N=20			
4		Run 1 Hard, fresh, slightly weathered, light gray, medium grained GRANITE, with horizontal to moderately dipping, moderately close, rough, slightly open joints		10	▽		1	10-10.1'			
							58		83		
	20.0	Run 2 Similar, except weathering is fresh	1142+/-	15			56		88		
		Boring Terminated at 20 Feet		20							

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *10' After drilling*

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-06-2018	Boring Completed: 10-06-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

BORING LOG NO. B-11

PROJECT: NextEra Chinook Solar

CLIENT: NextEra Energy Resources LLC Juno Beach, FL

SITE: 264 Fullam Hill Rd Fitzwilliam, NH

LOCATION See Exploration Plan

Latitude: 42.7579° Longitude: -72.1016°

Approximate Surface Elev.: 1066 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	RQD (%)	#200 Sieve	WATER CONTENT (%)
	0.7	7-inches of topsoil, roots, leaf litter, dark brown	1065.5+/-								
2		**SILTY SAND (SM)**, trace gravel, gray, very loose to very dense				X	18	1-1-1-1 N=2			
						X	20	18-19-22-29 N=41		35.6	15
				5	▽						
						X	22	4-5-8-14 N=13			
				10		X	20	10-22-38-50/4" N=60			
				15		X	12	38-47-50/3"			
	16.3	*Refusal at 16.25 Feet*	1049.5+/-								

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

Hammer Type: Automatic

Advancement Method:

Abandonment Method:

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.

Notes:

WATER LEVEL OBSERVATIONS

▽ *5' While drilling*

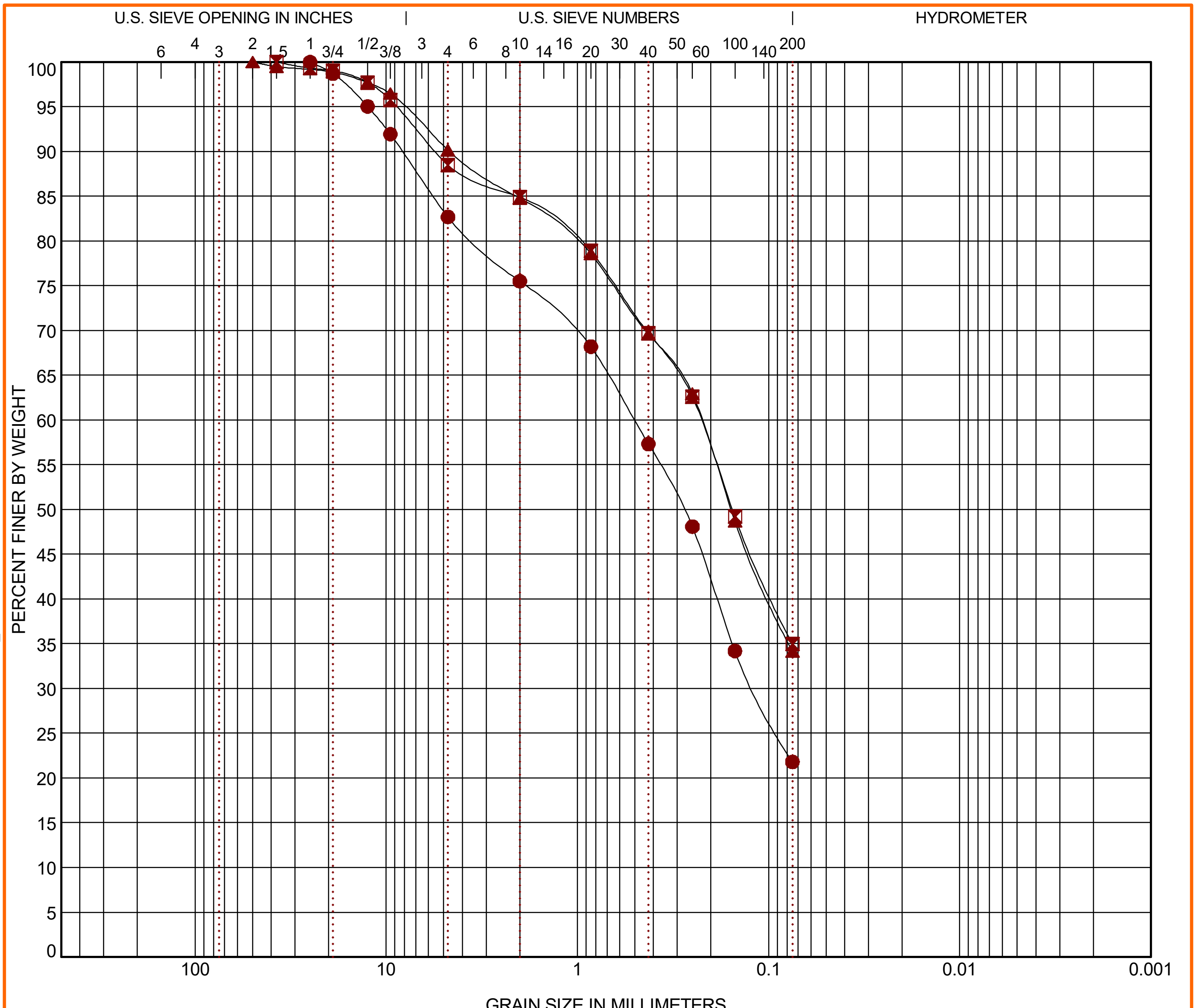
Terracon

77 Sundial Ave, Ste 401W
Manchester, NH

Boring Started: 10-06-2018	Boring Completed: 10-06-2018
Drill Rig: CME 550	Driller: Terracon/R. Brown
Project No.: J1175115	

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring ID	Depth	USCS Classification	WC (%)	LL	PL	PI	Cc	Cu
● B-1	2.1 - 5.1	Silty Sand with Gravel (SM)	12					
⊠ B-5	2.1 - 5.1	Silty Sand (SM)	21					
▲ B-6	2.1 - 5.1	Silty Sand (SM)	13					

Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
● B-1	2.1 - 5.1	25	0.504	0.119		17.3	60.9		21.8	
⊠ B-5	2.1 - 5.1	37.5	0.226			11.5	53.5		35.0	
▲ B-6	2.1 - 5.1	50	0.225			9.9	56.0		34.2	

PROJECT: NextEra Chinook Solar

PROJECT NUMBER: J1175115

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

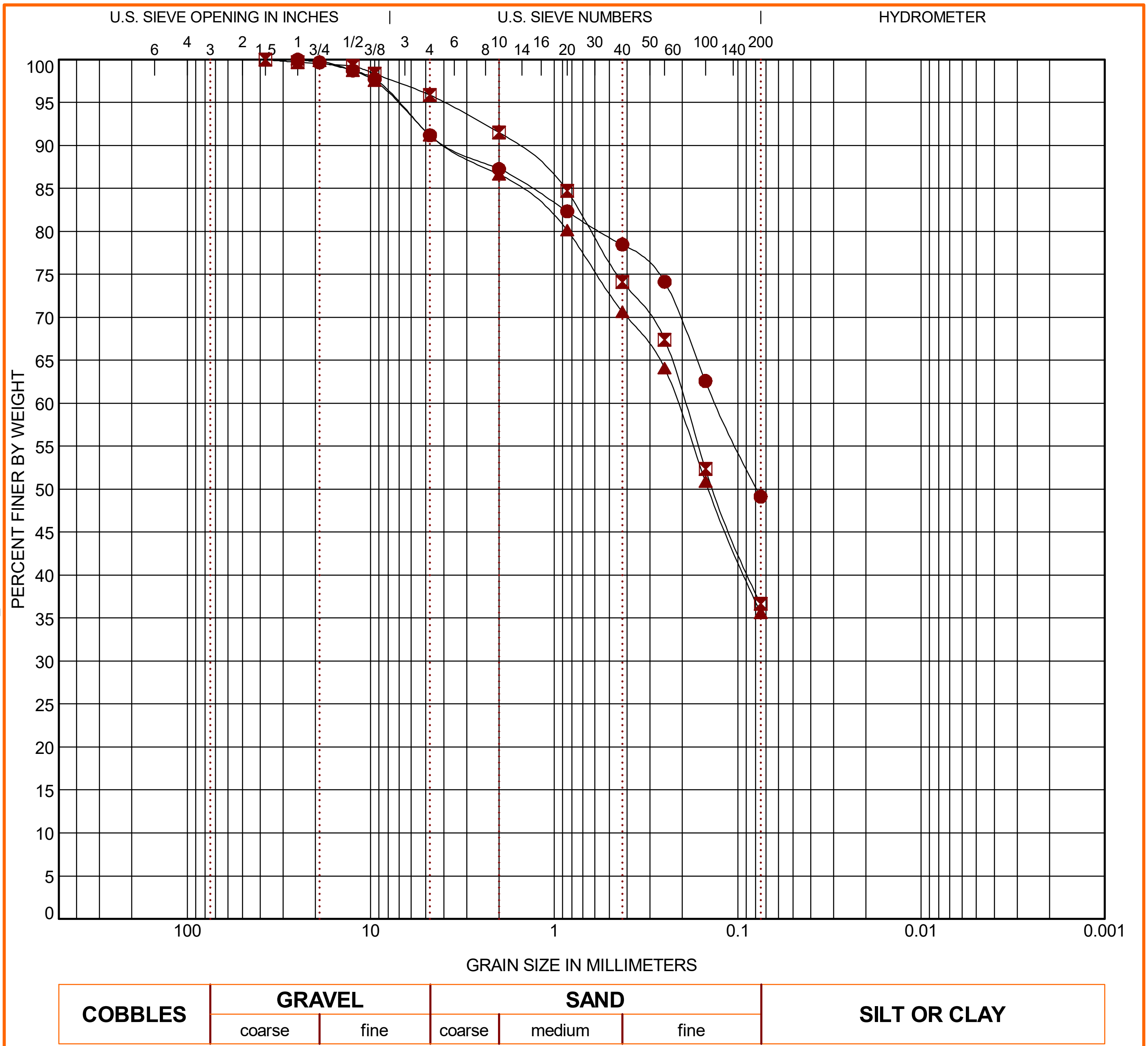
Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS-2 J1175115 NEXTERA CHINOOK S.GPJ TERRACON_DATATEMPLATE.GDT 10/18/18

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



	Boring ID	Depth	USCS Classification	WC (%)	LL	PL	PI	Cc	Cu
●	B-7	2.1 - 5.1	Silty Sand (SM)	15					
⊠	B-9	2.1 - 5.1	Silty Sand (SM)	18					
▲	B-11	2.1 - 5.1	Silty Sand (SM)	15					

	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Fines	%Clay
●	B-7	2.1 - 5.1	25	0.131			8.8	42.1		49.1	
⊠	B-9	2.1 - 5.1	37.5	0.194			4.1	59.2		36.7	
▲	B-11	2.1 - 5.1	25	0.213			8.8	55.5		35.6	

PROJECT: NextEra Chinook Solar

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

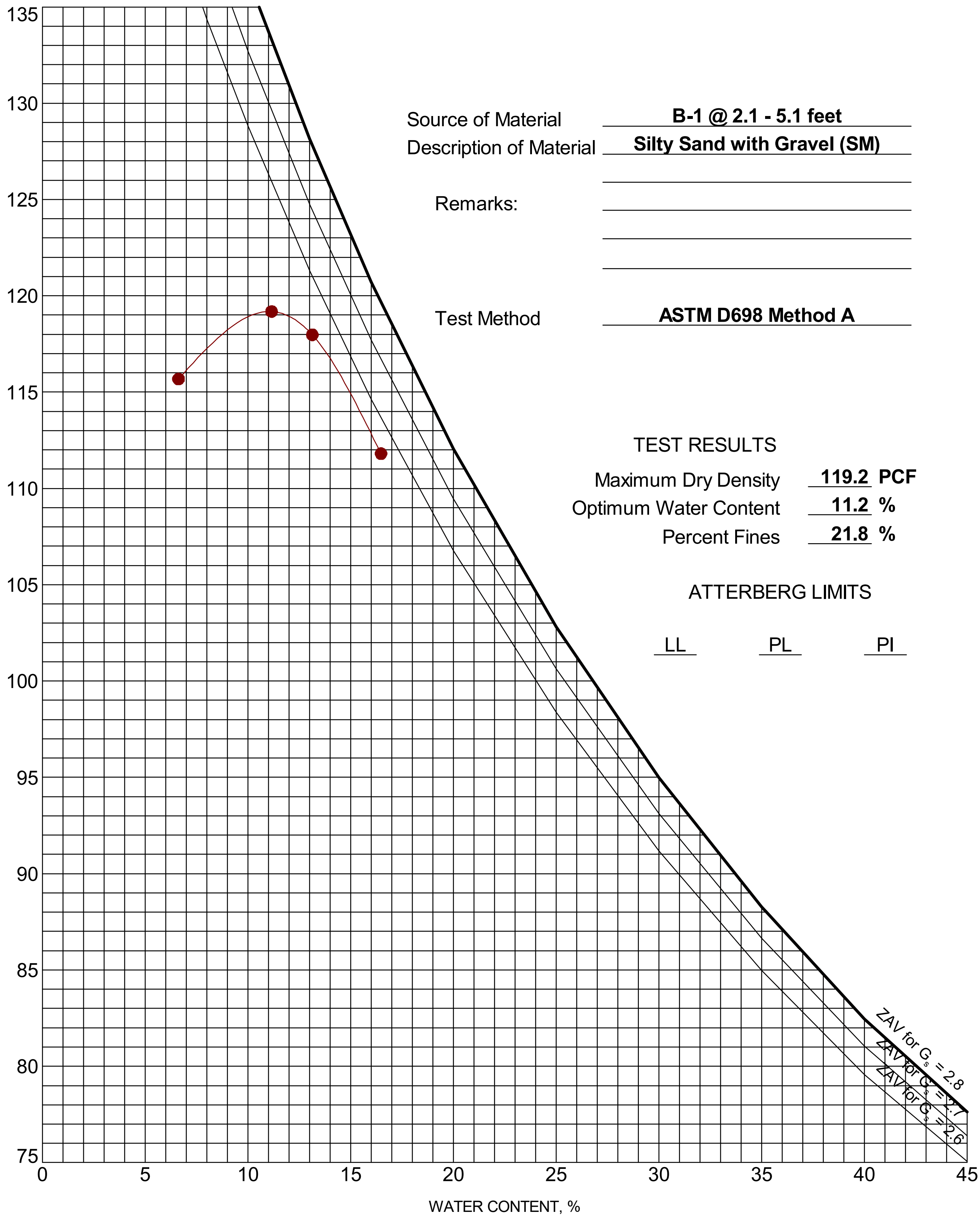
Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

PROJECT NUMBER: J1175115

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **B-1 @ 2.1 - 5.1 feet**

Description of Material **Silty Sand with Gravel (SM)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**119.2**	**PCF**
Optimum Water Content	**11.2**	**%**
Percent Fines	**21.8**	**%**

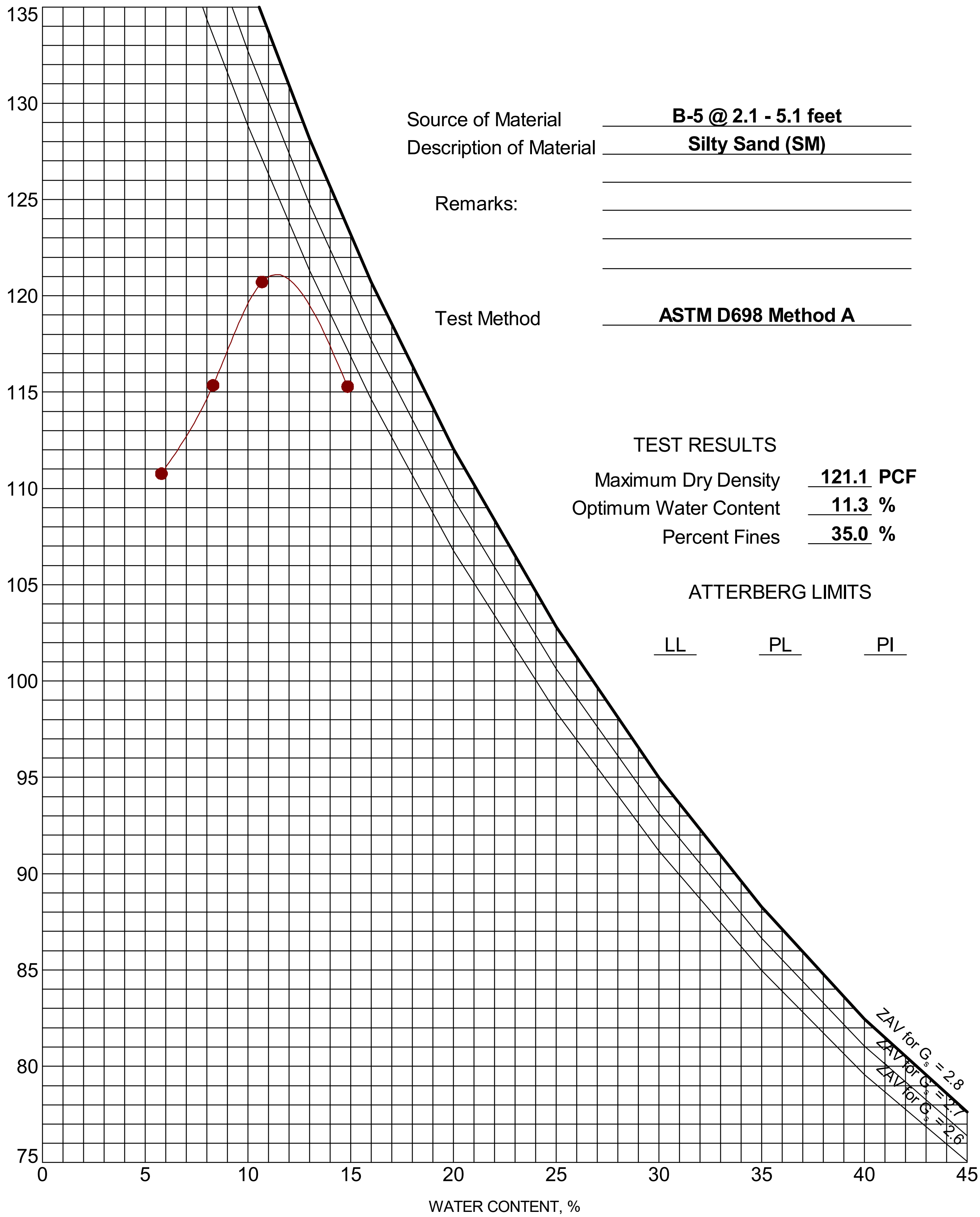
ATTERBERG LIMITS

LL	PL	PI

PROJECT: NextEra Chinook Solar	Terracon 77 Sundial Ave, Ste 401W Manchester, NH	PROJECT NUMBER: J1175115
SITE: 264 Fullam Hill Rd Fitzwilliam, NH		CLIENT: NextEra Energy Resources LLC Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-5 @ 2.1 - 5.1 feet**

Description of Material: **Silty Sand (SM)**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **121.1 PCF**

Optimum Water Content **11.3 %**

Percent Fines **35.0 %**

ATTERBERG LIMITS

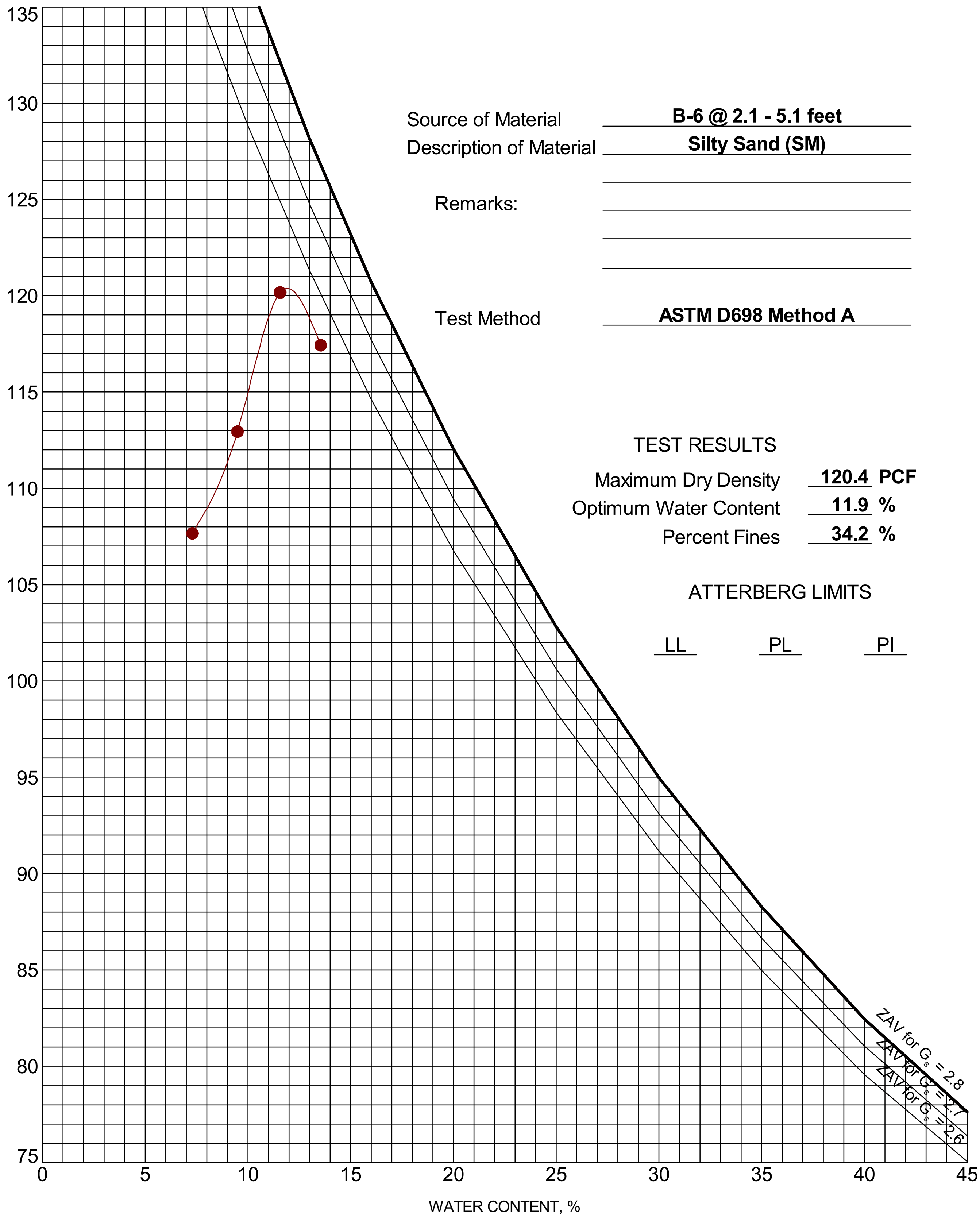
LL	PL	PI

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. COMPACTION - V2 J1175115 NEXTERA CHINOOK S.GPJ TERRACON_DATATEMPLATE.GDT 11/12/18

PROJECT: NextEra Chinook Solar	Terracon 77 Sundial Ave, Ste 401W Manchester, NH	PROJECT NUMBER: J1175115
SITE: 264 Fullam Hill Rd Fitzwilliam, NH		CLIENT: NextEra Energy Resources LLC Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material	**B-6 @ 2.1 - 5.1 feet**
Description of Material	**Silty Sand (SM)**
Remarks:	
Test Method	**ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**120.4**	**PCF**
Optimum Water Content	**11.9**	**%**
Percent Fines	**34.2**	**%**

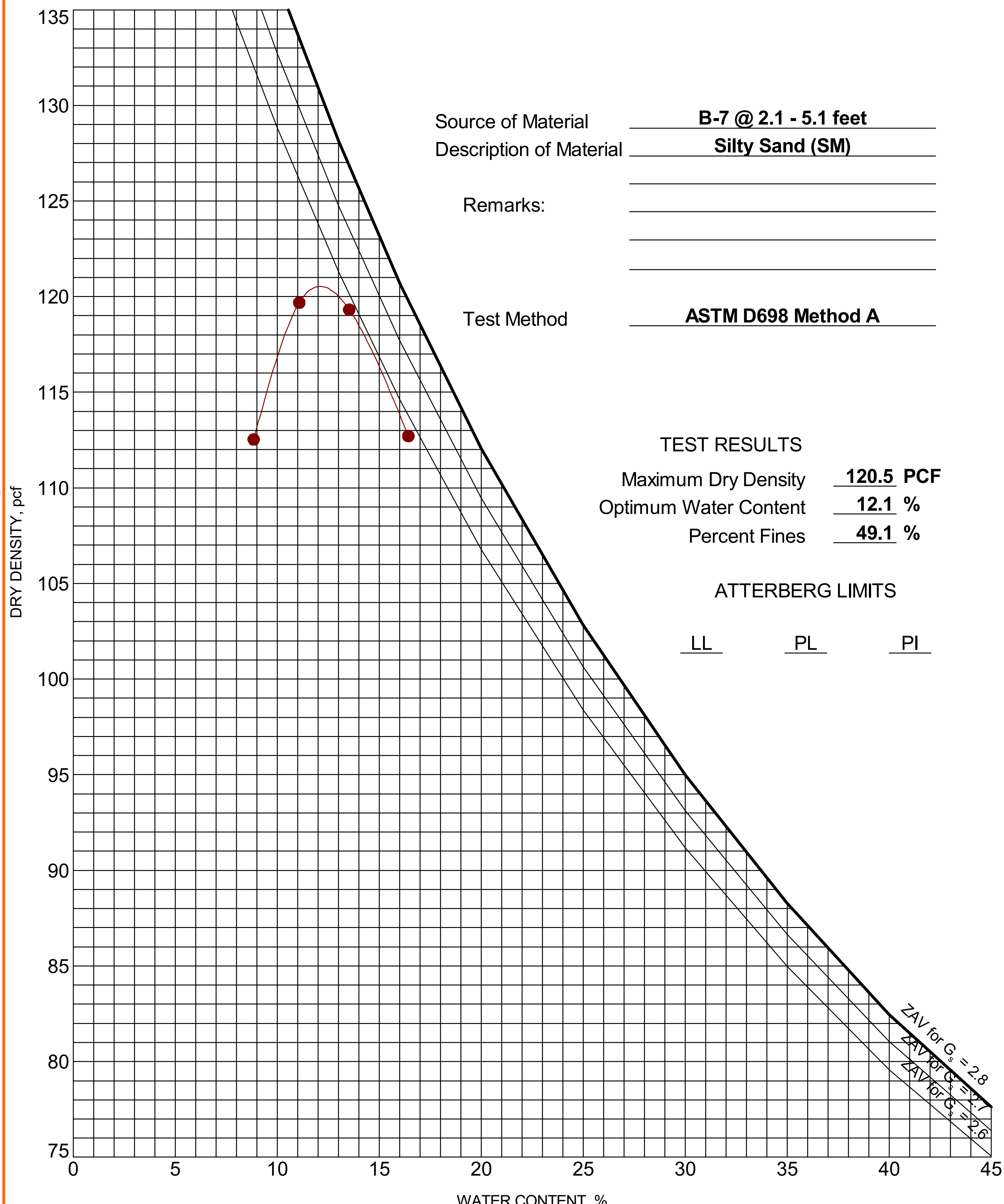
ATTERBERG LIMITS

LL	PL	PI

PROJECT: NextEra Chinook Solar	Terracon 77 Sundial Ave, Ste 401W Manchester, NH	PROJECT NUMBER: J1175115
SITE: 264 Fullam Hill Rd Fitzwilliam, NH		CLIENT: NextEra Energy Resources LLC Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-7 @ 2.1 - 5.1 feet**

Description of Material: **Silty Sand (SM)**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **120.5 PCF**

Optimum Water Content **12.1 %**

Percent Fines **49.1 %**

ATTERBERG LIMITS

LL	PL	PI

PROJECT: NextEra Chinook Solar

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

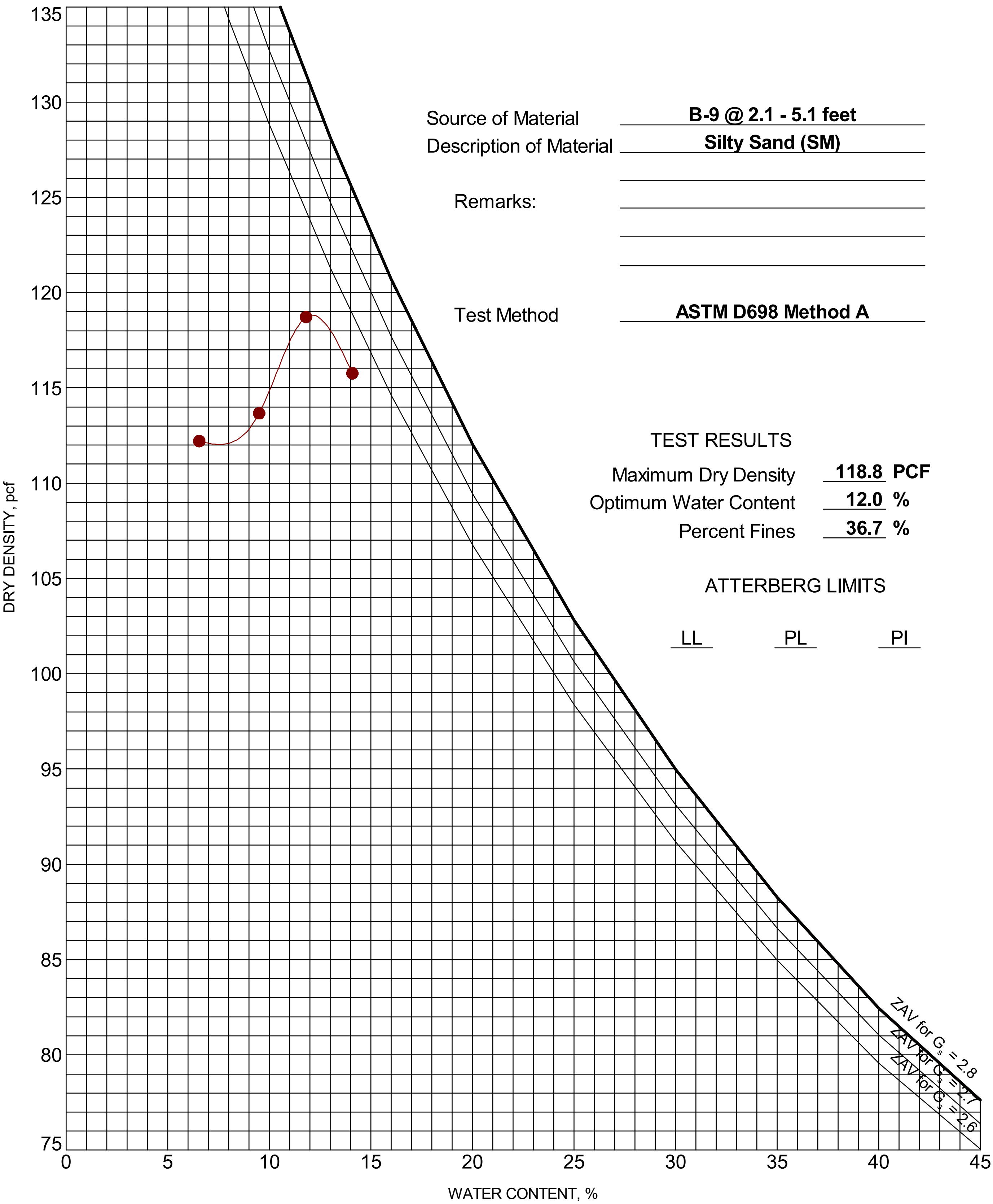
Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

PROJECT NUMBER: J1175115

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material **B-9 @ 2.1 - 5.1 feet**

Description of Material **Silty Sand (SM)**

Remarks:

Test Method **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density	**118.8**	**PCF**
Optimum Water Content	**12.0**	**%**
Percent Fines	**36.7**	**%**

ATTERBERG LIMITS

LL	PL	PI

PROJECT: NextEra Chinook Solar

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

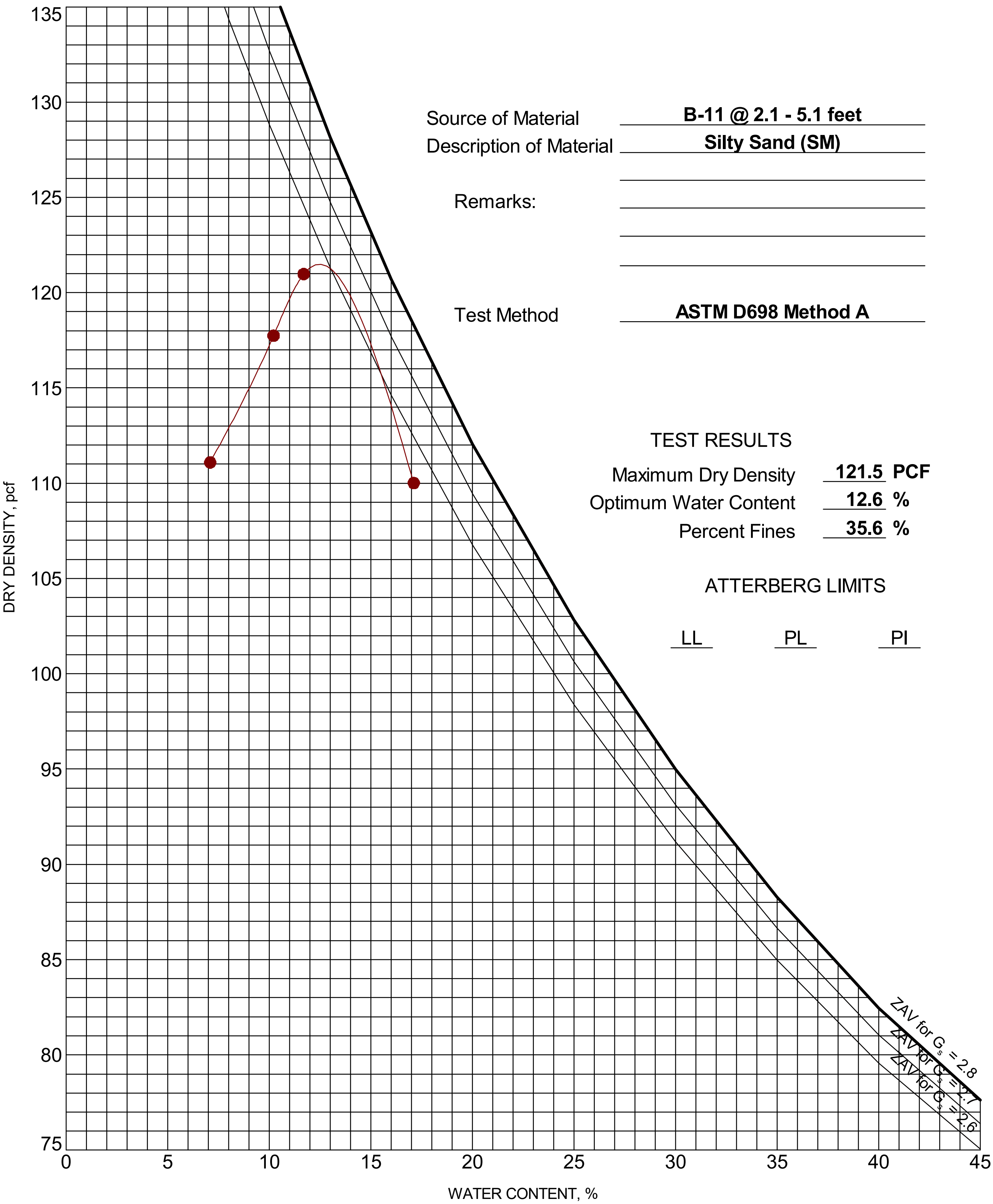
Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

PROJECT NUMBER: J1175115

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

MOISTURE-DENSITY RELATIONSHIP

ASTM D698/D1557



Source of Material: **B-11 @ 2.1 - 5.1 feet**

Description of Material: **Silty Sand (SM)**

Remarks:

Test Method: **ASTM D698 Method A**

TEST RESULTS

Maximum Dry Density **121.5 PCF**

Optimum Water Content **12.6 %**

Percent Fines **35.6 %**

ATTERBERG LIMITS

LL	PL	PI

PROJECT: NextEra Chinook Solar

SITE: 264 Fullam Hill Rd
Fitzwilliam, NH

Terracon
77 Sundial Ave, Ste 401W
Manchester, NH

PROJECT NUMBER: J1175115

CLIENT: NextEra Energy Resources LLC
Juno Beach, FL

R-1
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: Near B-9	Equipment: MiniSting R1
Test Date: 10-3-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	626.9	627.2	627.0	627	2,402
5	402.8	402.0	401.7	402	3,851
10	181.2	181.1	181.1	181	3,469
20	57.3	57.3	57.3	57	2,193
50	18.9	18.9	18.9	19	1,805
Northwest End Point Coordinates: 42.76547310, -72.10193638					
Southeast End Point Coordinates: 42.76537263, -72.10138301					
Center Point Coordinates: 42.76541417, -72.10166257					
Line Orientation: NW-SE					

Line Notes: Heavy vegetation. Some boulders at surface. Uneven terrain

Resistivity Line 2					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	734.6	734.7	734.2	735	2,813
5	428.3	431.8	431.8	431	4,123
10	213.6	213.6	213.6	214	4,090
20	55.2	55.2	55.2	55	2,115
50	19.6	19.6	19.6	20	1,874
Northeast End Point Coordinates: 42.76562171, -72.10157945					
Soutwest End Point Coordinates: 42.76526941, -72.10177398					
Center Point Coordinates: 42.76541417, -72.10166257					
Line Orientation: NE-SW					

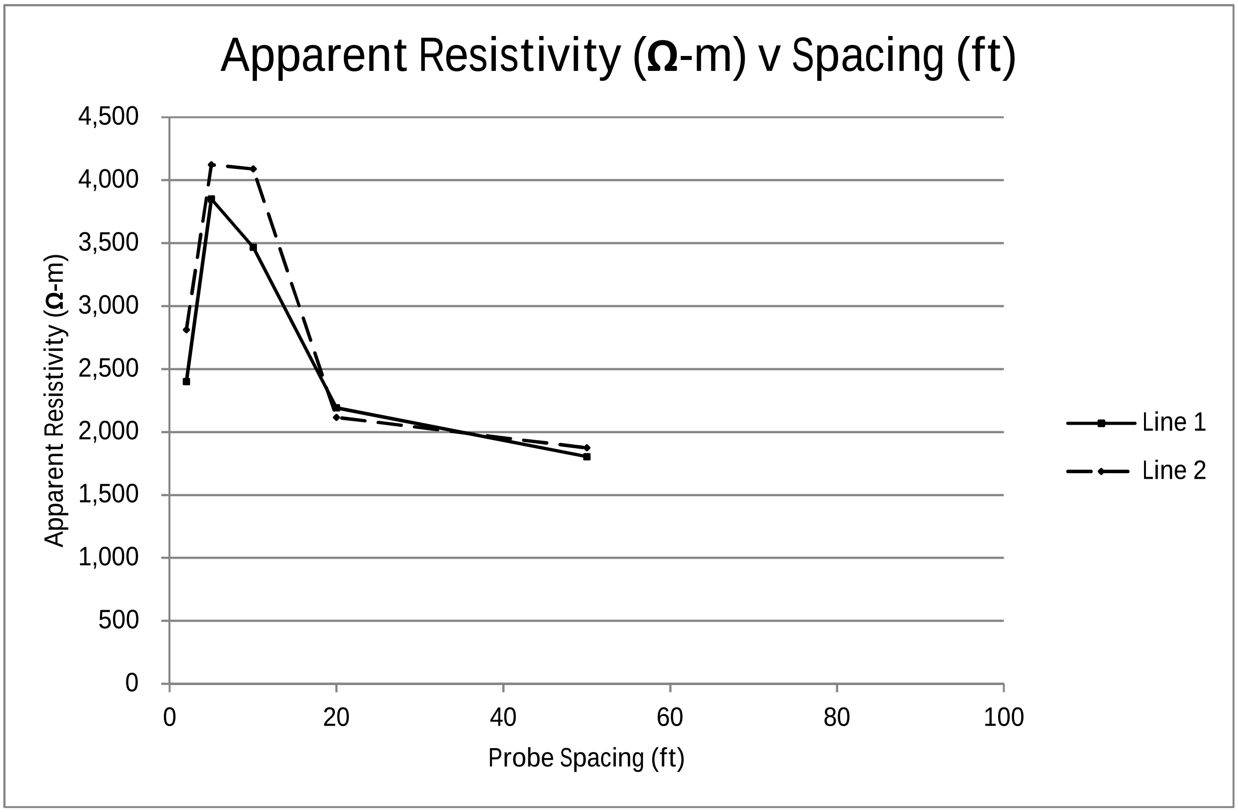
Line Notes: See line 1 notes

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-1
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: Near B-9



R-2
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: East of B-5	Equipment: MiniSting R1
Test Date: 10-4-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	318.1	318.1	318.1	318	1,218
5	157.1	157.1	157.0	157	1,504
10	64.2	64.2	64.2	64	1,229
20	10.3	10.3	10.3	10	393
50	3.6	3.6	3.6	4	342
East End Point Coordinates: 42.76710566, -72.10498248					
West End Point Coordinates: 42.76712340, -72.10553543					
Center Point Coordinates: 42.76711652, -72.10525848					
Line Orientation: E-W					

Line Notes: Rocks below surface. Mostly flat, grassy field.

Resistivity Line 2					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	455.3	455.2	455.2	455	1,744
5	195.8	195.8	195.8	196	1,875
10	47.1	47.1	47.1	47	901
20	10.5	10.5	10.5	11	404
50	3.9	3.9	3.9	4	370
North End Point Coordinates: 42.76729347, -72.10525963					
South End Point Coordinates: 42.76689621, -72.10524872					
Center Point Coordinates: 42.76711652, -72.10525848					
Line Orientation: N-S					

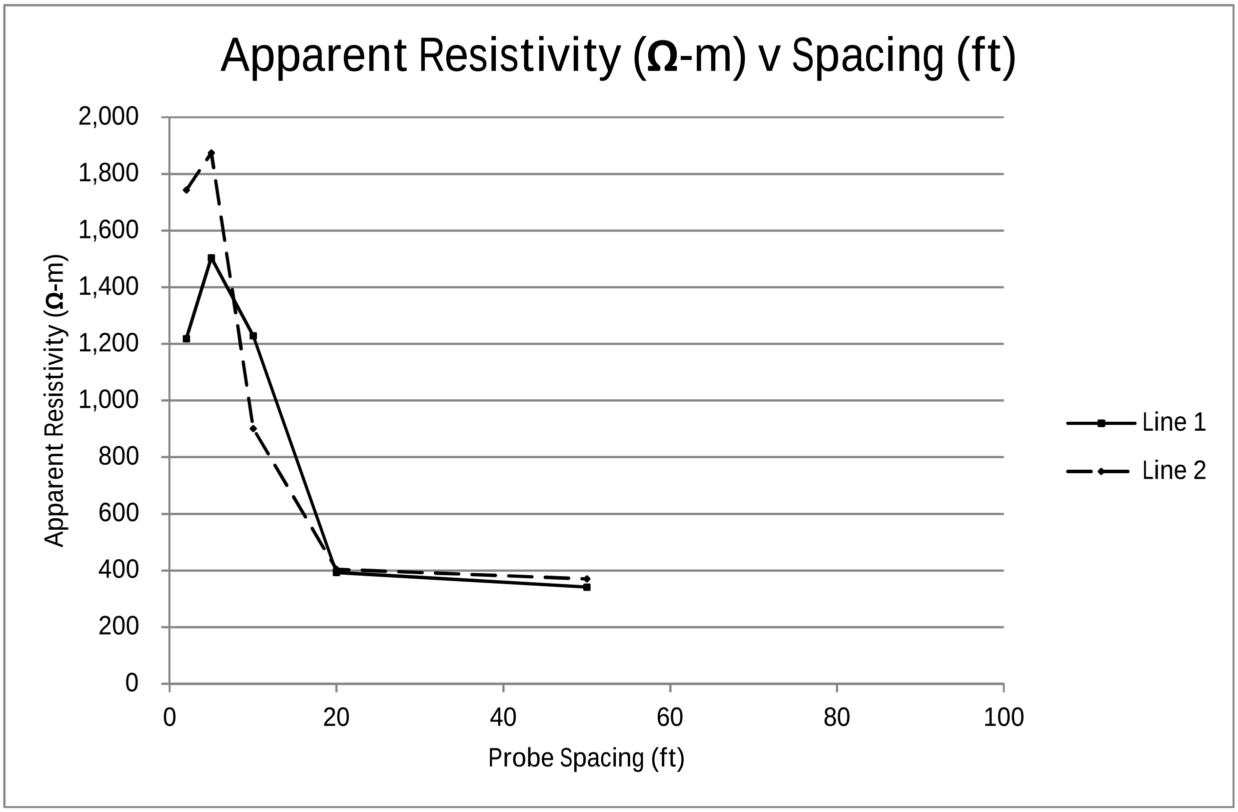
Line Notes: See line 1 notes. Stream to the north of the northern end.

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-2
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: East of B-5



R-3
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: Near B-2/B-3	Equipment: MiniSting R1
Test Date: 10-4-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1

Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	262.1	262.2	262.2	262	1,004
5	95.6	95.6	95.6	96	915
10	40.5	40.5	40.5	41	776
20	15.3	15.3	15.3	15	585
50	3.6	3.6	3.6	4	345

East End Point Coordinates: 42.77019399, -72.10429065
West End Point Coordinates: 42.77003342, -72.10479422
Center Point Coordinates: 42.77011421, -72.10453767
Line Orientation: E-W
Line Notes: Some vegetaion, grassy field

Resistivity Line 2

Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	256.1	256.0	257.8	257	983
5	110.5	110.5	110.5	111	1,058
10	29.5	29.5	29.5	30	565
20	10.8	10.8	10.8	11	412
50	3.9	3.9	3.9	4	378

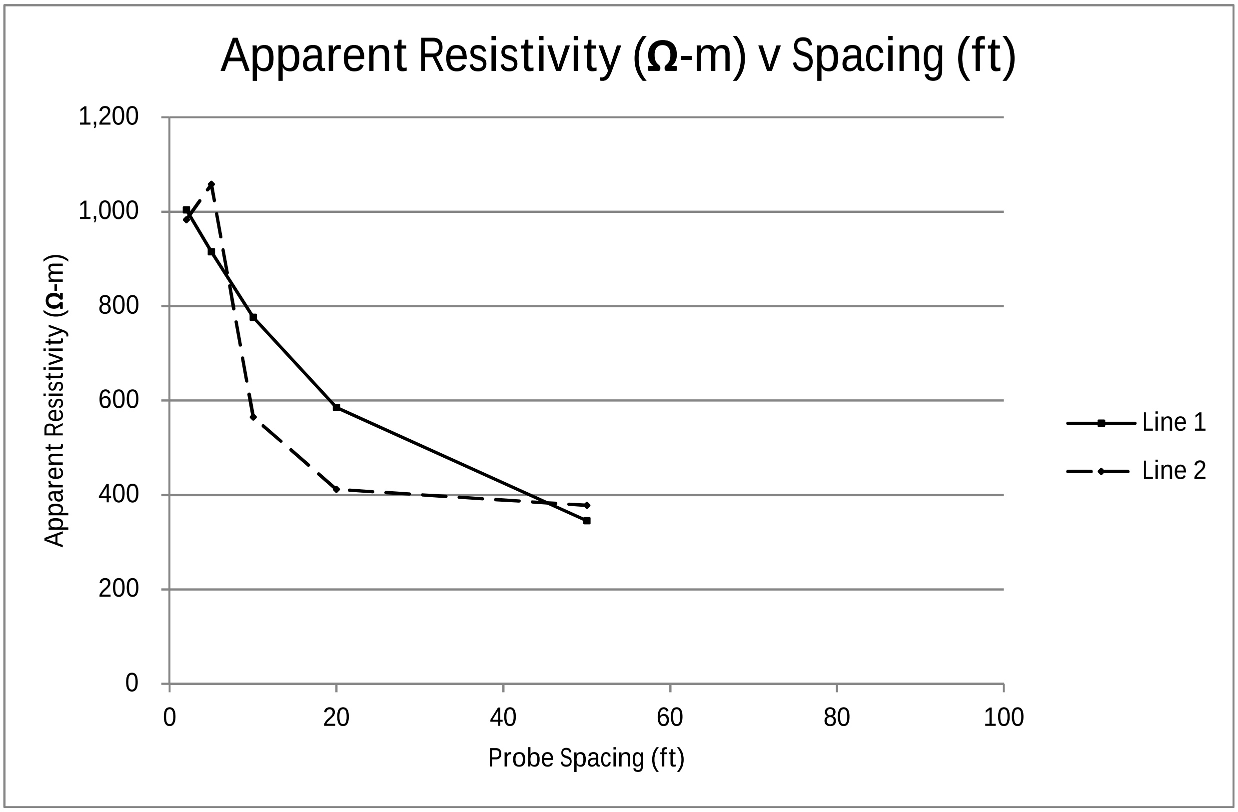
North End Point Coordinates: 42.77029321, -72.10462752
South End Point Coordinates: 42.76990680, -72.10462752
Center Point Coordinates: 42.77011421, -72.10453767
Line Orientation: N-S
Line Notes: See line 1 notes

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-3
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: Near B-2/B-3



R-4
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: Field NW of B-11	Equipment: MiniSting R1
Test Date: 10-5-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1

Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	452.8	452.7	452.6	453	1,734
5	270.7	270.7	270.7	271	2,592
10	177.6	177.6	177.6	178	3,401
20	76.6	76.6	76.6	77	2,935
50	33.3	33.3	33.3	33	3,185

North End Point Coordinates: 42.75854108, -72.10266080
South End Point Coordinates: 42.75812405, -72.10268642
Center Point Coordinates: 42.75833793, -72.10268663
Line Orientation: N-S

Line Notes: Vegetated field. Rocks at/below surface.

Resistivity Line 2

Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	423.3	423.3	423.1	423	1,621
5	254.2	254.2	254.2	254	2,434
10	160.2	160.3	160.3	160	3,069
20	84.8	84.8	84.8	85	3,249
50	27.5	27.5	27.5	27	2,632

East End Point Coordinates: 42.75833750, -72.10240276
West End Point Coordinates: 42.75832967, -72.10295897
Center Point Coordinates: 42.75833793, -72.10268663
Line Orientation: E-W

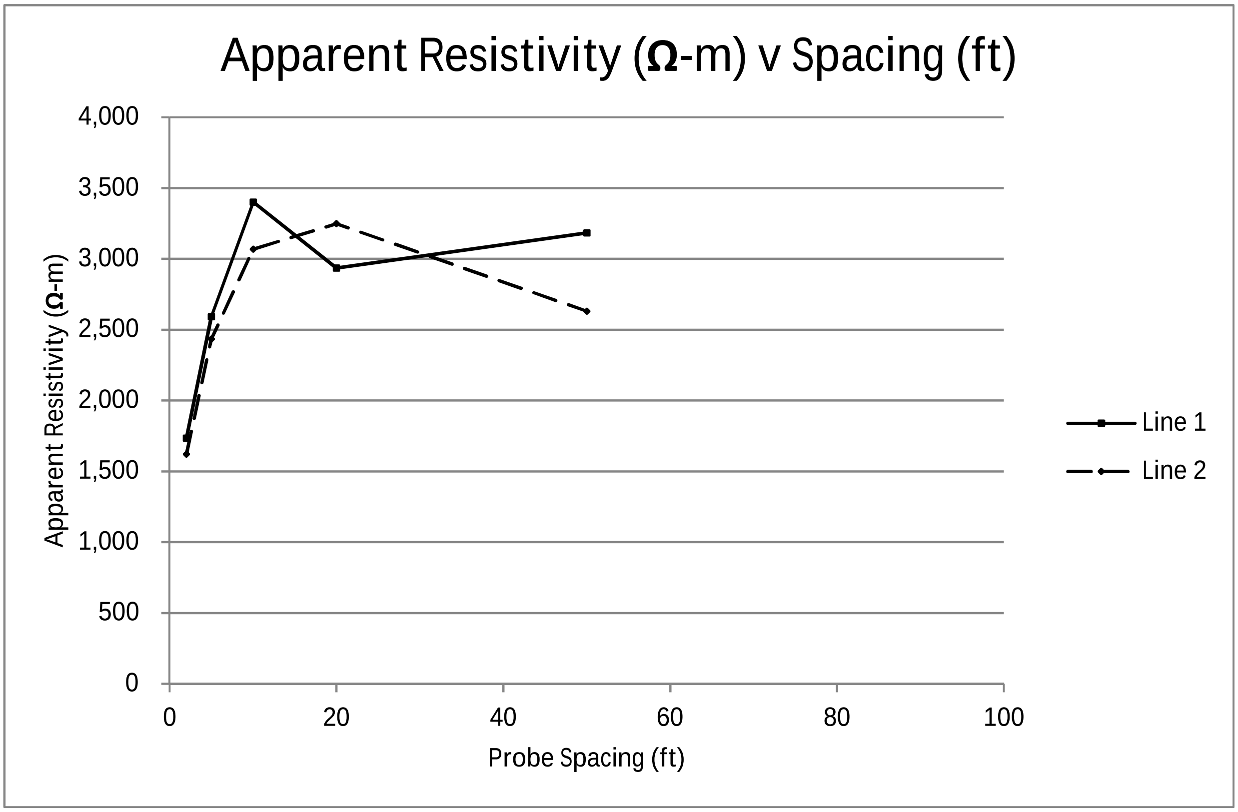
Line Notes: See line 1 notes

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-4
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: Field NW of B-11



R-5
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: SE of B-9	Equipment: MiniSting R1
Test Date: 10-5-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	120.4	120.5	120.5	120	461
5	89.9	89.9	89.9	90	861
10	50.0	50.0	50.0	50	957
20	21.5	21.5	21.5	22	824
50	11.5	11.5	11.5	12	1,104
East End Point Coordinates: 42.76073746, -72.10518394					
West End Point Coordinates: 42.76043945, -72.10552929					
Center Point Coordinates: 42.76059474, -72.10533909					
Line Orientation: E-W					

Line Notes: Many Downed trees, stumps. Pooling water in area, not at line. Rocks beneath surface

Resistivity Line 2					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	132.1	132.2	132.1	132	506
5	86.2	86.1	86.1	86	825
10	54.5	54.5	54.5	54	1,043
20	27.2	27.2	27.2	27	1,043
50	12.3	12.3	12.3	12	1,176
North End Point Coordinates: 42.76066345, -72.10560462					
South End Point Coordinates: 42.76051147, -72.10508437					
Center Point Coordinates: 42.76059474, -72.10533909					
Line Orientation: N-S					

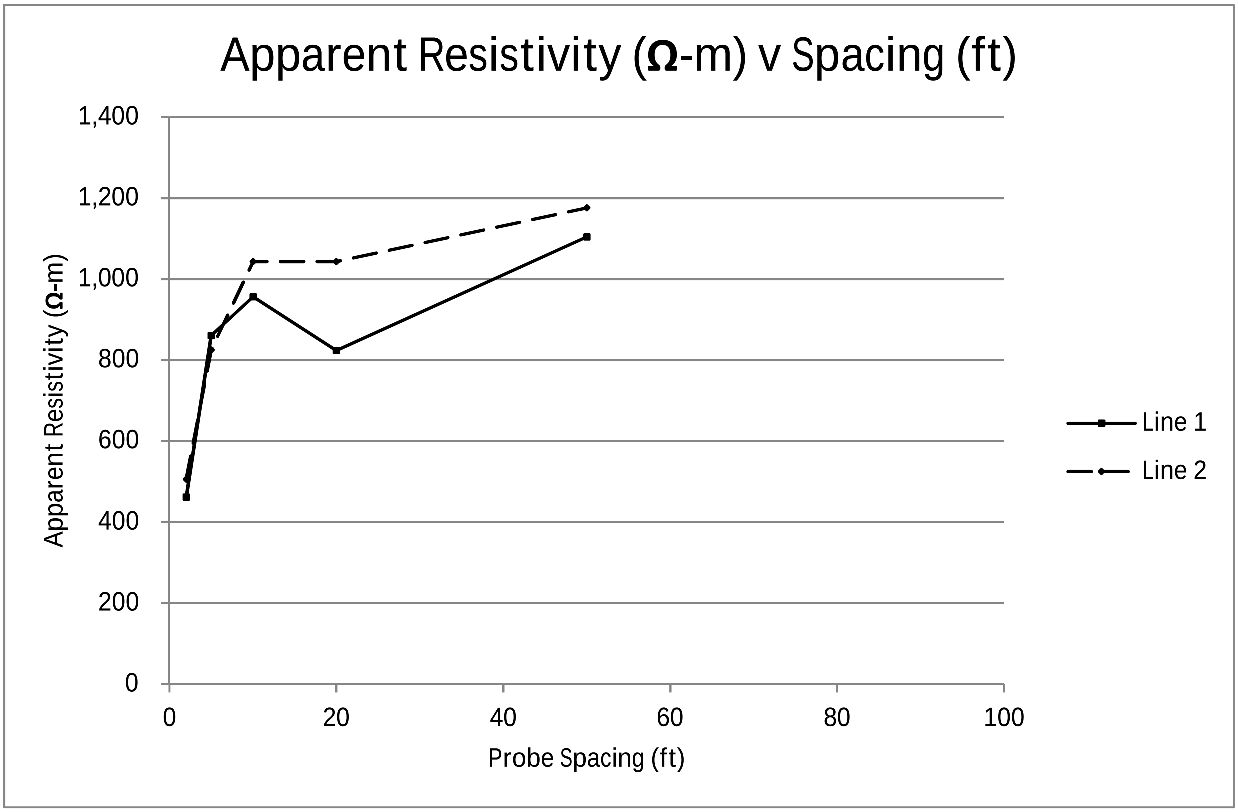
Line Notes: See line 1 notes

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-5
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: SE of B-9



R-6
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: NE of B-1	Equipment: MiniSting R1
Test Date: 10-5-18	Tested by: Andrew Michaud
Weather: Cloudy	Temperature: 60s

Resistivity Line 1					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	369.9	369.8	369.8	370	1,416
5	305.5	305.4	305.4	305	2,925
10	153.7	153.7	153.7	154	2,943
20	74.7	74.7	74.6	75	2,859
50	17.2	17.2	17.2	17	1,643
Northeast End Point Coordinates: 42.77702252, -72.11169769					
Southwest End Point Coordinates: 42.77671471, -72.11204680					
Center Point Coordinates: 42.77687662, -72.11186767					
Line Orientation: NE-SW					

Line Notes: In a powerline easement. Many rocks, stumps. Grassy eneven terrain.

Resistivity Line 2					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	397.8	397.8	397.8	398	1,524
5	270.1	270.2	270.2	270	2,587
10	140.7	140.8	140.8	141	2,696
20	59.3	59.3	59.3	59	2,269
50	14.8	14.8	14.8	15	1,415
Northwest End Point Coordinates: 42.77701348, -72.11208717					
Southeast End Point Coordinates: 42.77672606, -72.1167927					
Center Point Coordinates: 42.77687662, -72.11186767					
Line Orientation: NW-SE					

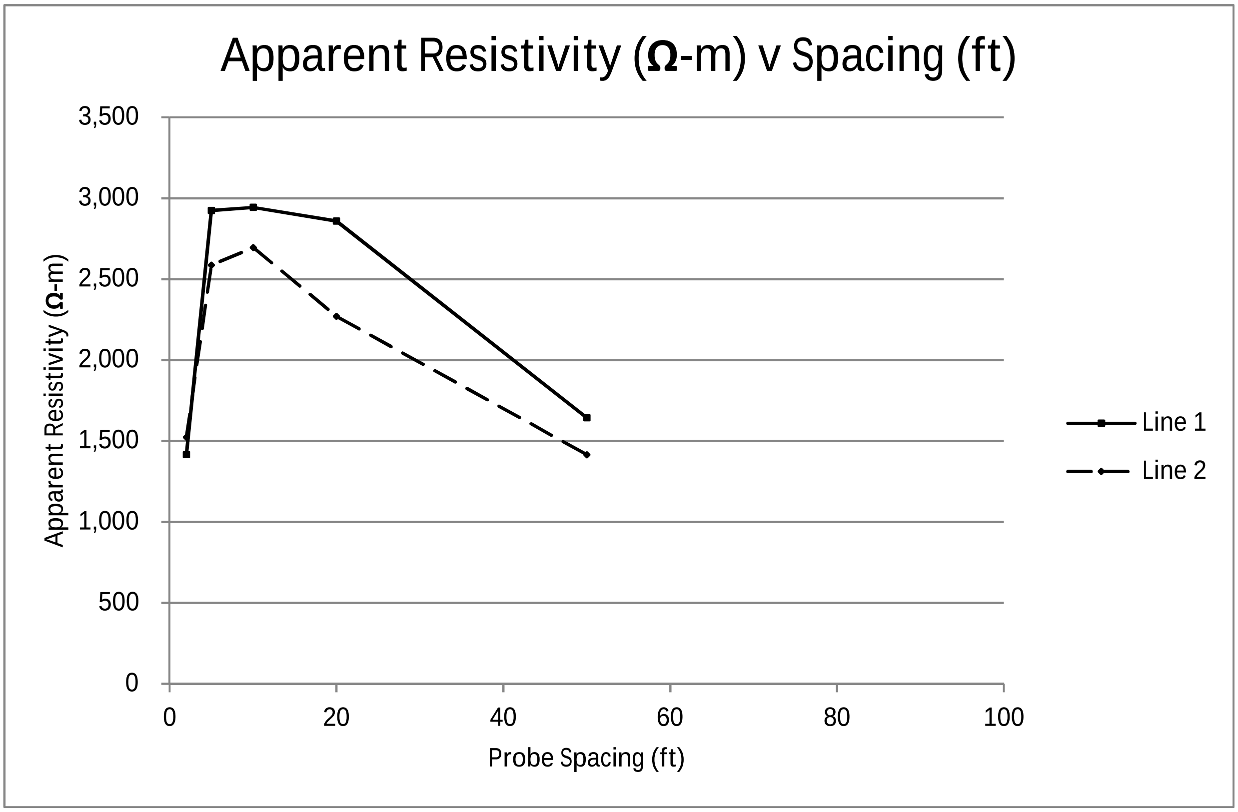
Line Notes: See line 1 notes

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-6
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: NE of B-1



R-7
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Terracon

Test Location: Near B-6/B-7	Equipment: MiniSting R1
Test Date: 10-17-18	Tested by: Andrew Michaud
Weather: Sunny	Temperature: 40-50s

Resistivity Line 1					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	495.5	495.4	495.3	495	1,897
5	156.3	156.3	156.2	156	1,496
10	47.2	47.2	47.2	47	903
20	12.2	12.2	12.2	12	466
50	4.5	4.5	4.5	4	429
100	3.4	3.4	3.4	3	643
200	2.7	2.7	2.7	3	1,019
300	2.3	2.3	2.3	2	1,306
North End Point Coordinates: 42.76816020, -72.10170422					
South End Point Coordinates: 42.76576505, -72.10098029					
Center Point Coordinates: 42.76697360, -72.10137778					
Line Orientation: N-S					

Line Notes: Grassy field on south end of line, hard-packed roadway on northern half.

Resistivity Line 2					
Probe Spacing (ft)	Resistance Reading (Ω)				Apparent Resistivity (Ω-m)
	R1	R2	R3	Average	
2	313.8	313.3	314.4	314	1,202
5	58.2	58.2	58.2	58	558
10	18.9	18.9	18.9	19	361
20	9.9	9.9	9.9	10	378
50	5.4	5.4	5.4	5	515
100	4.1	4.1	4.1	4	793
East End Point Coordinates: 42.76718533, -72.10045991					
West End Point Coordinates: 42.76725909, -72.10155216					
Center Point Coordinates: 42.76721562, -72.10101388					
Line Orientation: E-W					

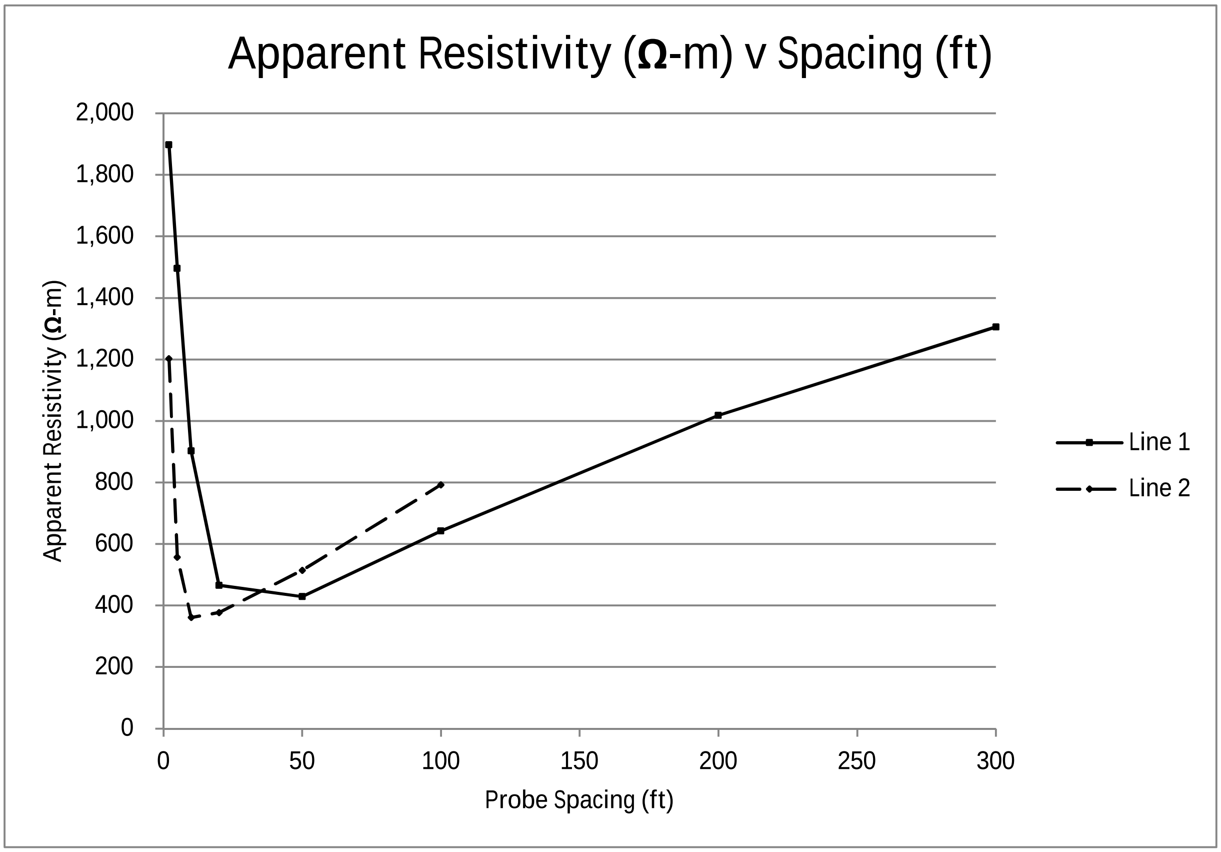
Line Notes: Grassy logging road. Soil slightly damp. Cobbles in soil.

Comments:

1. Field electrical resistivity testing performed per ASTM G57-06, "Standard Test Method for Field Measurement of Electrical Resistivity Using the Wenner Four-Electrode Method"

R-7
Field Electrical Resistivity Test Results
Chinook Solar
Fitzwilliam,NH
J1175115

Test Location: Near B-6/B-7



CHEMICAL LABORATORY TEST REPORT

Project Number: J1175115
Service Date: 11/07/18
Report Date: 11/19/18
Task:

Terracon

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

Client

NextEra Energy Resources LLC
Juno Beach, FL

Project

NextEra Chinook Solar

Sample Submitted By: Terracon (J1) **Date Received:** 11/1/2018 **Lab No.: 18-1359**

Results of Corrosion Analysis

Sample Number			
Sample Location	B-1	B-6	B-9
Sample Depth (ft.)	2.1-5.1	2.1-5.1	2.1-5.1
pH Analysis, AWWA 4500 H	7.85	7.94	7.66
Water Soluble Sulfate (SO4), ASTM C 1580 (mg/kg)	43	38	63
Sulfides, AWWA 4500-S D, (mg/kg)	Nil	Nil	Nil
Chlorides, ASTM D 512, (mg/kg)	43	53	53
Red-Ox, AWWA 2580, (mV)	+672	+682	+678
Total Salts, AWWA 2520 B, (mg/kg)	257	119	122
Resistivity, ASTM G 57, (ohm-cm)	24250	36860	57230

Analyzed By: Trisha Campo

Trisha Campo
Chemist

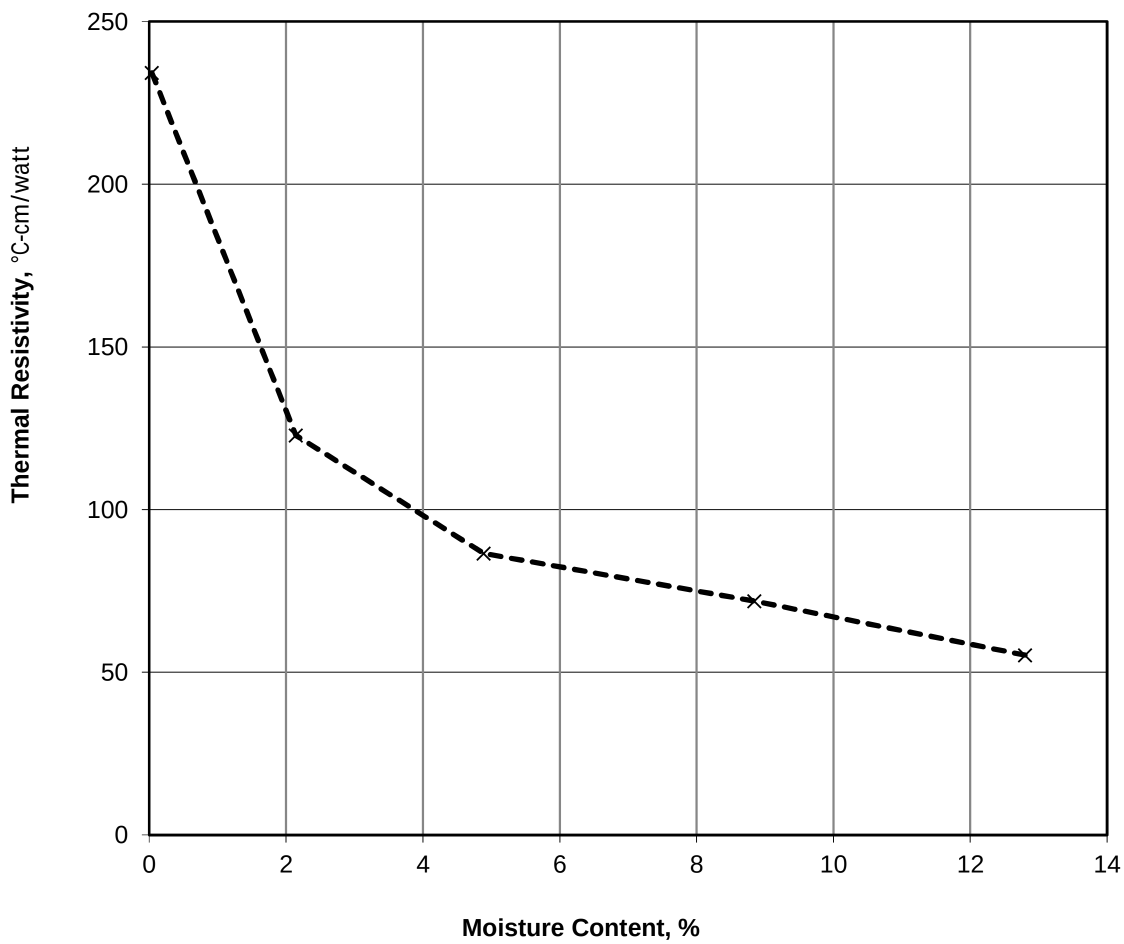
The tests were performed in general accordance with applicable ASTM, AASHTO, or DOT test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent of our company. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.

Project Name: NexEra Chinook Solar
Project Number: J1175115
Sample ID: B-1@2.1-5.1 feet
Soil Type: Silty Sand with Gravel
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 119.2
Optimum Moisture Content, %: 11.2
Target % Compaction: 90
Sample Dry Density, pcf: 107
Sample % Compaction: 90
As-received Moisture Content, %: 12.7

Thermal Resistivity Test Results

Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.0	234	23.8
2.1	123	23.0
4.9	87	19.2
8.8	72	19.3
12.8	55	17.6

Thermal Resistivity Dry-Out Curve



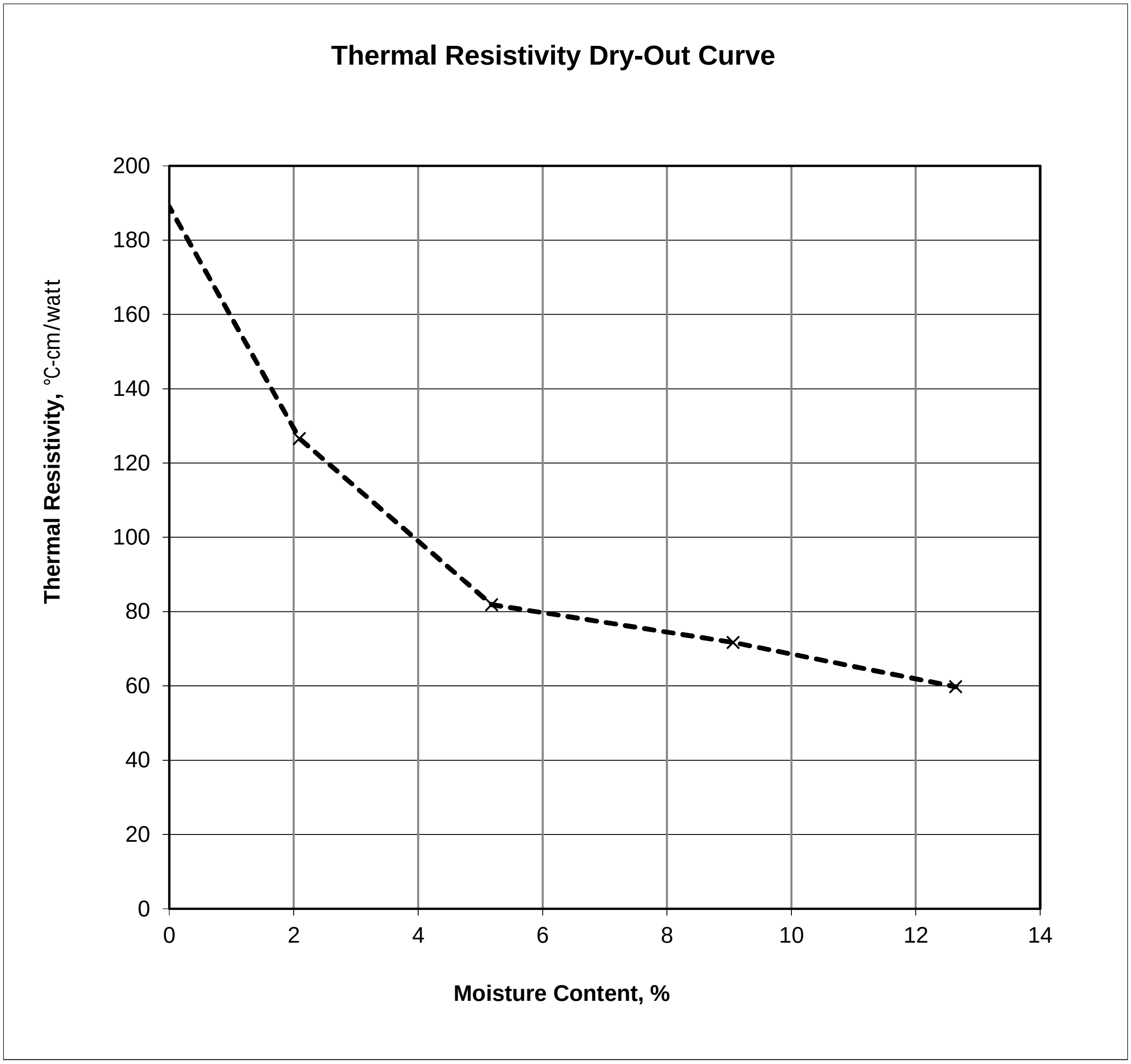
Date: 11/12/18	Run By: AMM	Reviewed By: BWP

Terracon

Project Name: NexEra Chinook Solar
Project Number: J1175115
Sample ID: B-5
Soil Type: Silty Sand
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 121.1
Optimum Moisture Content, %: 11.3
Target % Compaction: 90
Sample Dry Density, pcf: 109
Sample % Compaction: 90
As-received Moisture Content, %: 20.3

Thermal Resistivity Test Results

Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.0	190	23.0
2.1	127	19.0
5.2	82	18.1
9.1	72	18.1
12.6	60	18.5



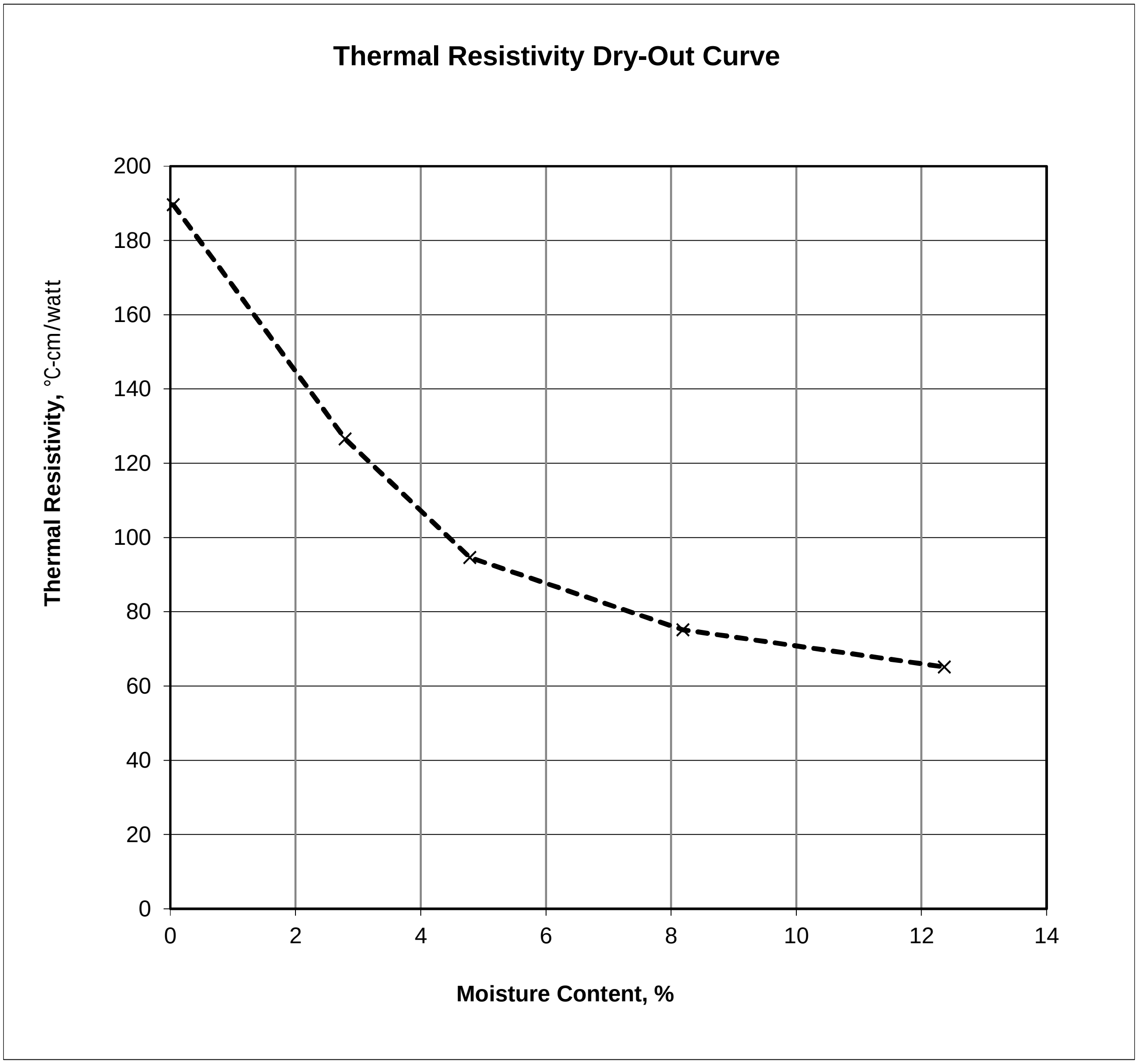
Date: 11/12/18	Run By: AMM	Reviewed By: BWP

Terracon

Project Name: NexEra Chinook Solar
Project Number: J1175115
Sample ID: B-6
Soil Type: Silty Sand
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 120.4
Optimum Moisture Content, %: 11.9
Target % Compaction: 90
Sample Dry Density, pcf: 109
Sample % Compaction: 90
As-received Moisture Content, %: 12.8

Thermal Resistivity Test Results

Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.0	190	22.7
2.8	127	16.9
4.8	95	16.8
8.2	75	17.8
12.4	65	19.2



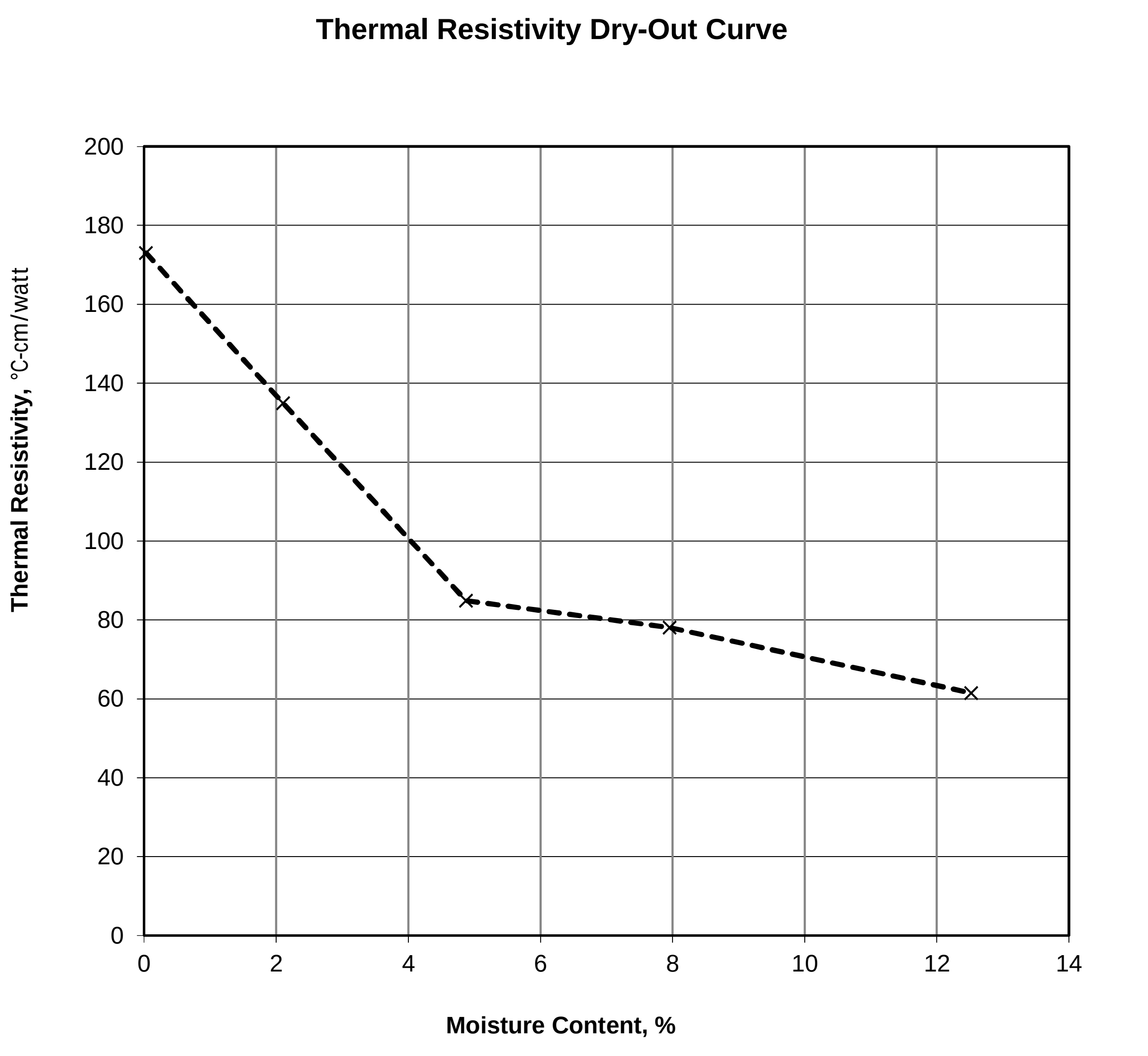
Date: 11/12/18	Run By: AMM	Reviewed By: BWP

Terracon

Project Name: NexEra Chinook Solar
Project Number: J1175115
Sample ID: B-7
Soil Type: Silty Sand
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 120.5
Optimum Moisture Content, %: 12.1
Target % Compaction: 90
Sample Dry Density, pcf: 109
Sample % Compaction: 90
As-received Moisture Content, %: 15.4

Thermal Resistivity Test Results

Moisture Content (%)	Thermal Resistivity (°C-cm/watt)	Temperature (°C)
0.0	173	23.4
2.1	135	23.2
4.9	85	22.7
8.0	78	19.5
12.5	61	18.8

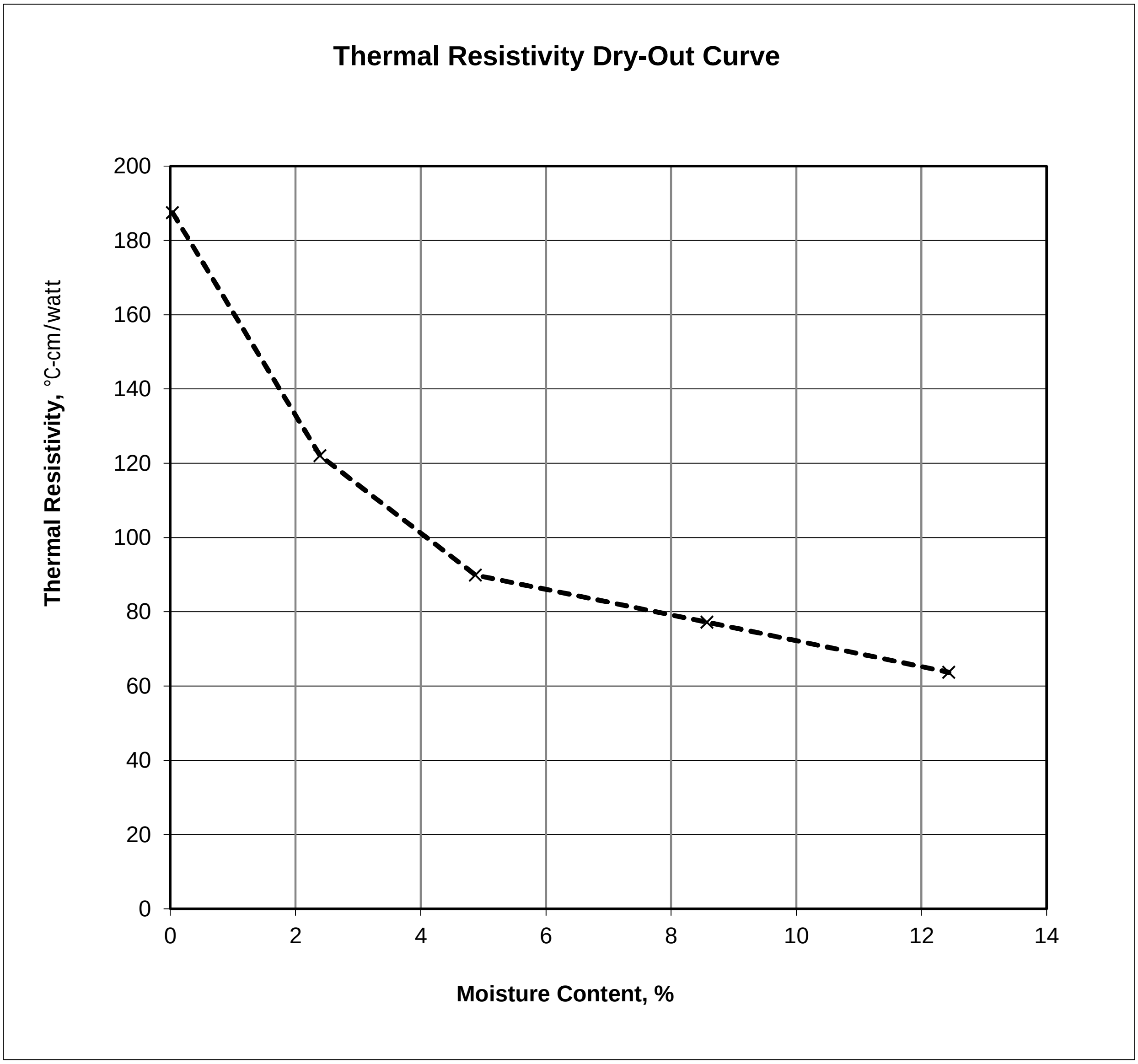


Date: 11/12/18	Run By: AMM	Reviewed By BWP

Terracon

Project Name: NexEra Chinook Solar
Project Number: J1175115
Sample ID: B-11
Soil Type: Silty Sand
Standard/Modified Proctor: ASTM D 698-A
Max Dry Density, pcf: 121.5
Optimum Moisture Content, %: 12.6
Target % Compaction: 90
Sample Dry Density, pcf: 110
Sample % Compaction: 90
As-received Moisture Content, %: 15.3

Thermal Resistivity Test Results		
Moisture Content (%)	**Thermal Resistivity (°C-cm/watt)**	**Temperature (°C)**
0.0	187	24.5
2.4	122	18.1
4.9	90	18.6
8.6	77	18.2
12.4	64	18.4



Date: 11/12/18	Run By: AMM	Reviewed By: BWP

Terracon

SUPPORTING INFORMATION

Contents:

Unified Soil Classification System
Description of Rock Properties

Note: All attachments are one page unless noted above.

UNIFIED SOIL CLASSIFICATION SYSTEM

Terracon GeoReport

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]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [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]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]		SW	Well-graded sand [I]
			$Cu < 6$ and/or [Cc<1 or Cc>3.0] [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G, H, I]
			Fines classify as CL or CH		SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above “A” line [J]		CL	Lean clay [K, L, M]
			PI < 4 or plots below “A” line [J]		ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay [K, L, M, N]
			Liquid limit - not dried			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	< 0.75	OH	Organic clay [K, L, M, P]
			Liquid limit - not dried			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] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[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.

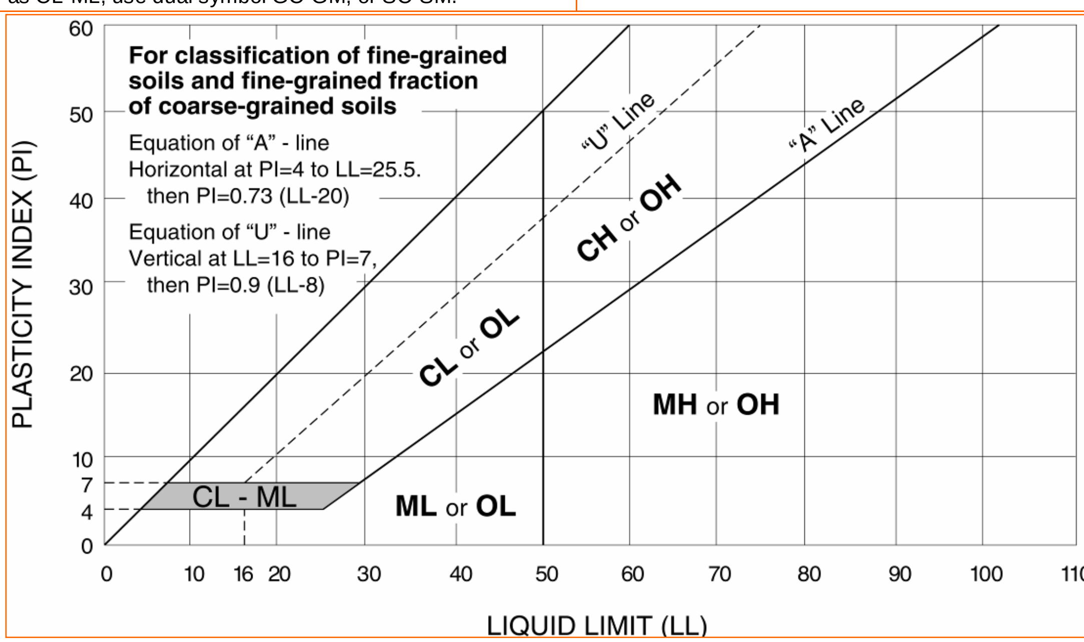
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add “gravelly” to group name.

[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.



DESCRIPTION OF ROCK PROPERTIES

WEATHERING	
Term	**Description**
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi (MPa)**
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Extremely close	< ¾ in (<19 mm)	**Laminated**	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	**Very thin**	½ in – 2 in (12 – 50 mm)
Close	2-1/2 in – 8 in (60 – 200 mm)	**Thin**	2 in – 1 ft. (50 – 300 mm)
Moderate	8 in – 2 ft. (200 – 600 mm)	**Medium**	1 ft. – 3 ft. (300 – 900 mm)
Wide	2 ft. – 6 ft. (600 mm – 2.0 m)	**Thick**	3 ft. – 10 ft. (900 mm – 3 m)
Very Wide	6 ft. – 20 ft. (2.0 – 6 m)	**Massive**	> 10 ft. (3 m)

Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0-degree angle.

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009
Technical Manual for Design and Construction of Road Tunnels – Civil Elements

DESCRIPTION OF ROCK PROPERTIES

Terracon GeoReport

WEATHERING	
Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" no discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)	
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [1]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

1. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) [1]

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

1. RQD (given as a percentage) = length of core in pieces 4 inches and longer / length of run

Joint Openness Descriptors

Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.