<u>Exhibit H</u>

Geotechnical Report

Consulting
Engineers and
Scientists

Geotechnical Report

Bethany Solar

428 Bethmour Road
Bethany, Connecticut

Submitted to:
BL Companies
355 Research Parkway
Meriden, CT 06450

Submitted by:
GEI Consultants, Inc.
455 Winding Brook Drive, Suite 201
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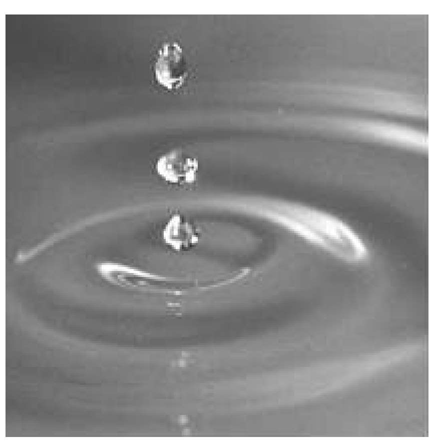
July 5, 2022
Project No. 2201295



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1. Introduction

1.1 Project Summary

GEI Consultants, Inc. (GEI) prepared this report to present the results of a subsurface exploration program and foundation recommendations for the proposed ground-mounted photovoltaic (PV) array in Bethany, Connecticut. On behalf of Tritec, BL Companies has engaged GEI to provide geotechnical engineering services for this project.

1.2 Scope of Services

GEI completed the following scope of services for this report. These services were performed to investigate the subsurface conditions at the Site:

- Marked out test pit locations in preparation for the public utility service mark out (Call Before You Dig).
- Conducted a subsurface exploration program consisting of six (6) test pits.
- Assigned three (3) sieve analyses with hydrometer and moisture content laboratory tests.
- Graphed the grain size distribution test results on the USDA Soil Texture Triangle, obtained the NRCS Hydrologic Soil Group, and estimated a soil infiltration rate.
- Assigned soil resistivity, pH, sulfates, and chlorides testing to one (1) composite soil sample.
- Provided soil corrosivity analysis.
- Developed recommendations for a ballast-supported PV array, should this be evaluated as an option by the design team.
- Developed soil parameters that can be used in the design of a pile-supported PV array.
- Developed frost parameters that can be used in the design of a pile-supported PV array and the solar developer’s risk evaluation.
- Developed recommendations for the access roadway cross section.
- Prepared this *Geotechnical Report* presenting the results of the subsurface explorations and our recommendations.

We performed these services in general accordance with the Connecticut Building Code (Building Code), which is comprised of the 2015 International Building Code (IBC) and a separate package of state-specific amendments.

1.3 Authorization

Our work was performed in general accordance with our proposal dated February 11, 2022, and the resulting Subconsultant Agreement executed March 22, 2022.

1.4 GEI Team

The following GEI personnel performed the services for this report:

- Matthew Glunt, P.E. Project Manager / Technical Review
- Anna Hernberg, P.E. Geotechnical Engineer
- Thomas Rezzani, E.I.T. Geotechnical Professional

1.5 Vertical and Horizontal Reference

Elevations provided in this report are in feet and are referenced to the contours on the plan titled “Sketch Plan”, Sheet No. SK-7, prepared by BL Companies dated January 2022.

Test pit locations were geo-referenced at the site using a handheld GPS unit with accuracy on the order of 5 to 10 feet. These locations were overlaid onto the provided site plan and sketched on Figure 1. Test pit locations shown should be considered approximate.

2. Site and Project Description

2.1 Site Description

The site is a 21.23-acre property located at 428 Bethmour Road. The site is bound by residential properties to the north, east, and south, and Bethmour Road to the west.

The proposed development area is generally brush-covered, with thinner vegetation along previously disturbed areas near Bethmour Road, and woods at the eastern end of the property. A vacant house and associated outbuildings are located at the northeast corner of the site. Several dry-stacked stone walls cross the property.

The grade slopes from El. 625 at the northwest property corner down to El. 590 at the eastern extent of the proposed limit of disturbance. Existing wetlands are located along the east side of the property, beyond the proposed limit of disturbance.

2.2 Proposed Construction

We were provided with a copy of the preliminary Site Plan drawing (SK-7) by BL Companies. We understand an approximate 1.25 MW DC/1.0 MW AC ground-mounted solar array will be sited on the property. Based on the provided preliminary Site Plan, in addition to the PV array, the development will consist of the following:

- One concrete electrical equipment pad and one substation, both located at the northwest corner of the site.
- One stormwater management basin located to the southeast of the PV array.
- A 12-ft wide gravel road ringing the solar array.
- A small gravel parking area for maintenance personnel.
- A new permanent entrance from Bethmour Road.

We understand the preference of the solar developer is to support the array on pile foundations. Recommendations for design and construction of racking pile foundations, as well as a ballast foundation alternative, are provided in Sections 5.3 and 5.4.

We expect that most of the proposed solar array will generally follow the existing contours.

3. Exploration Procedures

3.1 Field Testing Procedures

The test pit locations were laid out within areas of interest on the site based on the provided sketch plan using a handheld GPS unit. Approximate test pit locations relative to the site plan are shown in Figure 1. The appropriate one-call utility location service (Call Before You Dig) was contacted prior to our arrival.

Six (6) test pits were excavated within or near the proposed development footprint on April 7, 2022, using an excavator to depths of 4.5 to 8.7 feet each. Several (5 of 6) test pits were terminated based on excavator refusal. The test pits were logged and photographed by GEI. Test pit logs are attached in Appendix A.

Representative samples were placed in appropriately identified sealed bags and transported to our office for laboratory assignment. Upon completion, each test pit was backfilled with excavated spoils in lifts tamped with the excavator bucket.

3.2 Laboratory Testing

Laboratory testing was conducted on representative soil samples to confirm field identification of the soils and establish engineering characteristics for design. Tests performed by GeoTesting Express, under subcontract to GEI, included the following:

- Three (3) grain-size analyses with standard sieve set and hydrometer (ASTM D6913/D7928)
- Three (3) moisture content tests (ASTM D2216)
- The following corrosion tests on one sample from test pit TP-1, composited from depths ranging from 0.7 to 4.5 feet deep:
    - pH (ASTM D4972)
    - Sulfates (ASTM D516)
    - Chlorides (ASTM D512)
    - Electrical resistivity (ASTM G57).

Results of the laboratory testing program are attached in Appendix B.

4. Subsurface Conditions

4.1 Geologic Setting

Local geologic maps identify that the referenced area is underlain by thick deposits of glacial till (DEP 2009). Glacial till deposits typically overlay the bedrock surface.

Bedrock underlying the site is mapped (Rodgers 1985) as the Beardsley Member of Harrison Gneiss, which is described as gray to dark gray, medium-grained, well-layered and lineated gneiss.

4.2 Subsurface Conditions

The generalized subsurface conditions at the site are described below, in order of increasing depth. The subsurface conditions between exploration locations may differ. The nature and extent of variations between the sampling points will not become evident until construction.

Topsoil – Topsoil in the test pits was measured at 8 to 14 inches thick. These soils were generally characterized as silty sand (SM) or sandy silt (ML) and contained roots and organic fibers. The topsoil in TP-1 contained approximately 10% gravel and small cobbles.

Silty Sand – A 1-foot-thick layer of silty sand (SM) was encountered below the topsoil layer in TP-2. The sand contained approximately 30 percent fines.

Glacial Till – Glacial till was encountered beneath the topsoil and silty sand layers to test pit termination. These soils were characterized as variable proportions of sand, silt, and gravel, and were most often classified as silty sand with gravel (SM), silty gravel with sand (GM), and widely graded sand with silt and gravel (SW-SM). The proportion of silty fines generally ranged from 15 to 35 percent. Interspersed cobbles and boulders were noted below 1.5 feet deep.

Excavator Refusal – Other than TP-5, the test pits were terminated based on excavator refusal at depths ranging from 4.5 to 8.2 feet (El. 620.5 at the northwest corner to El. 594.5 at the southwest corner). Refusal was generally most shallow at the northwest and southeast corners of the proposed development area.

Excavator refusal may have resulted from encountering very dense glacial till, weathered rock, cobbles or boulders, or the upper surface of sound bedrock. Diamond core sampling would be required to determine the character and continuity of material below the refusal of excavator.

4.3 Groundwater Conditions

Groundwater intrusion was observed in five test pits at depths of 2.3 to 3.8 feet. Groundwater intrusion was not observed in TP-4. We note that dense glacial till deposits may exhibit very slow infiltration and recharge rates. Therefore, groundwater may be present within these soils but not observed as free water within test excavations until several hours after the hole is opened. Samples in dense glacial till below groundwater may have been described as "damp" or "moist" due to the compact matrix of the stratum.

Groundwater levels are subject to seasonal and weather-related variations. Groundwater measurements made at different times and different locations may be significantly different than the measurements taken as part of this investigation.

5. Design Recommendations

5.1 Design Load Recommendations

The foundation of the ground mounted PV array should be designed to resist the forces caused by the load combinations in the Building Code for a Risk Category I structure.

We recommend that wind and snow loading from the Building Code be considered when developing foundation designs as follows:

- Wind load should be calculated in accordance with Chapter 6 of ASCE 7 with the exception of basic wind speed, which is specified in Chapter 16 of the Building Code Table 1604.11. The ultimate wind speed, V_{ult}, for Risk Category I for Bethany is 115 mph.
- Snow load should be calculated in accordance with Chapter 7 of ASCE 7 with the exception of ground snow load, which is specified in Chapter 16 of the Building Code, Table 1604.11. The ground snow load for Bethany is 30 lb/ft^2.

5.2 Allowable Soil Bearing Capacity

The maximum allowable bearing pressures that should be used for the design of equipment pads or PV ballast pads, should they be used, are listed below. Based on the results of this investigation, the equipment pad will likely be founded on glacial till.

Bearing Stratum	Net Allowable Bearing Pressure
Native Glacial Till or Structural Fill	2.0 tons/ft^2

The natural soils may be susceptible to frost heave. We recommend that the proposed equipment pads or other slabs or footings bear on Structural Fill that extends below the frost depth. If some seasonal movement of the equipment pads is acceptable, we recommend all organics, and the top foot of existing frost susceptible material below the slab should be removed and replaced with compacted Structural Fill. At least 18 inches of Structural Fill should be placed below the slab in all areas.

5.3 Pile-supported PV Array Recommendations

We understand that piles will likely be favored by the solar developer to support the PV array in the in-situ soils. Recommended geotechnical parameters for pile design are provided in Table 1.

Dense glacial till containing cobbles and boulders should be expected across the site. Difficulties such as shallow pile refusal on rock and misalignments due to cobble and boulder obstructions should be expected. These conditions may result in misalignments or difficulty reaching depth requirements. Capabilities of foundation products for installation in these difficult conditions will vary by manufacturer, some of which may have proprietary solutions for working in this type of environment. We recommend forwarding the results of this investigation to pile suppliers/designers, who will have a better understanding of the capabilities and limitations of their specific foundation products, as well as potential mitigation options.

Potential pile-support systems include but are not limited to ground screw piles and driven piles. Ground screws have been advertised as a cost-effective solution to rocky soil environments. We understand that pilot holes for the ground screws can be drilled through boulders or into bedrock.

For lateral pile capacity calculations in soil, we recommend using the passive earth pressure coefficients, K_p, for each soil type provided in Table 1. The pile designer must also consider potential lateral pile movements. Movements of several inches may be needed to develop the lateral capacity.

For axial loading, we recommend that piles be designed using an allowable skin friction and allowable end bearing based on the NAVFAC DM 7.02 analysis procedure provided in Appendix C. Alternatively, the pile designer can opt to perform on-site load tests to estimate the allowable loads.

The soil chemical and resistivity test results in Section 5.8 are provided so that the pile designer can perform a corrosivity analysis based on the materials of the pile.

The pile designer should consider the forces caused by frost on the piles, compared to the pile tension capacity. Recommended adfreeze and frost depth consideration are discussed below.

5.4 Ballast-supported PV Array Recommendations

An alternative to the proposed pile foundation is a ballast system. Potential Ballast-Support systems include but are not limited to:

- Precast Concrete Ballast
- Cast-in-Place Concrete Ballast

If the PV array or a portion of the PV array is supported by ballast ground-mount systems, the subgrade should be proof-rolled with a 5-ton vibratory roller before placing the ballast system. Where fill is added, we recommend that Structural Fill, Ordinary Fill, or on-site soils be placed and compacted to at least 92 percent of its maximum dry density determined in accordance with ASTM D1557 (Modified Proctor).

We recommend a maximum allowable soil bearing pressure as shown in the Allowable Soil Bearing Capacity table in Section 5.2.

The details of the surface preparation for the ballast system depend on the system selected. Generally, the bearing surface for each ballast system element should be level.

The natural soils and Ordinary Fill may be susceptible to frost heave. Therefore, some movement of the ballast foundation should be expected.

5.5 Adfreeze/Freezing Conditions

Soil in contact with foundations near the ground surface can freeze to the foundation and develop a substantial adfreeze bond. If the soil in contact with the foundation is frost susceptible, heave can transmit uplift forces to the foundation. Based on the test pit and laboratory results, soils expected to be in contact with racking piles contain up to about 35 percent fines and are therefore potentially frost susceptible.

We recommend using the average value of adfreeze bond stress of 100 kPa (approximately 2,100 lb/ft^2) and 65 kPa (approximately 1,300 lb/ft^2) for fine-grained soils frozen to steel and concrete, respectively, as reported in the Canadian Foundation Engineering Manual 4th Edition.

5.6 Frost Depth

The Connecticut State Building Code specifies a minimum embedment of 42 inches for frost protection of foundations for buildings and structures.

5.7 Seismic Design

The 2018 edition of the Connecticut Building Code document mirrors the 2015 International Building Code, with exception of the revisions and supplemental information provided by state building officials.

Based on the criteria of Building Code Section 1613.3.2 and the conditions observed in the test pits, we recommend the use of Site Class D for seismic design. The Site Class was used in conjunction with the seismic hazard (S_S, S_1) for this location to determine spectral design values, as follows:

Corresponding spectral response design parameters are as follows:

2018 Connecticut Building Code	
Site Class	D
Risk Category	I
Use/Occupancy Group	U
S_S	0.189 g
S_1	0.063 g
S_{DS}	0.202 g
S_{D1}	0.101 g
PGA_M	0.147 g
Seismic Design Category	B

We calculated the spectral response parameters for the Site using general procedures outlined in Building Code Section 1613.3. Peak ground acceleration (PGA_M) is adjusted for Site Class effects, per ASCE 7-10 Section 11.8.3.

Soils present below the site are not judged to be susceptible to liquefaction and this does not need to be accounted for in the design.

5.8 Soil Corrosivity

Electrical resistivity is a broad indicator of soil corrosivity because corrosion reactions are electrochemical in nature and proceed most rapidly when resistivity (i.e., resistance to the flow of ions and electrical current) is low. Specifically, resistivity is a measure of how strongly a given material opposes the flow of electrical current. The composite sample collected from test pit TP-1 at depths 0.7 to 4.5 feet had an electrical resistivity reading of 113,634 Ω-cm, indicating a non-corrosive environment.

Sulfates in soil and groundwater in concentrations greater than 1,000 mg/kg are generally considered to be corrosive to structural elements. The American Concrete Institute recommends that Type II cement be used if sulfate concentrations exceed 1,000 mg/kg. Sample test results indicate sulfates concentrations of less than 10 mg/kg, which is less than 1,000 mg/kg.

Chloride concentrations above 500 mg/kg are generally considered to be corrosive to structural elements. Sample test results indicate chloride concentrations of 12 mg/kg, which is less than 1,000 mg/kg.

We summarized our evaluation of the soil corrosivity to structural elements shown in the table below by comparing the laboratory test results to some available corrosivity references.

Test	Laboratory Results	Reference	Corrosivity to Structural Elements
pH	6.7	Caltrans - Corrosion Guidelines January 2015	Not corrosive
Electrical Resistivity	113,634 Ω-cm	EPRI - Environmental Factors Governing Corrosion Rates, Report 1021854 December 2011	Not corrosive
Chlorides	12 mg/kg	Caltrans - Corrosion Guidelines January 2015	Not corrosive
Sulfates	<10 mg/kg	Caltrans - Corrosion Guidelines January 2015	Not corrosive

5.9 Estimated Infiltration Rate

As currently shown, we expect the bottom of the proposed stormwater basin will be in poorly draining dense glacial tills. We evaluated the USDA soil texture of the sample collected in this region by plotting the grain size analysis results on the USDA Soil Texture Triangle. The soil texture class for this sample is “Sandy Loam.”

We then evaluated the NRCS hydrologic soil group and infiltration rate based on the USDA soil textures. The NRCS hydrologic soil group and estimated infiltration rate for “Sandy Loam” are “B” and 1.0 inches/hour, respectively. NRCS data is summarized in Table 2.

6. Construction Considerations

6.1 Subgrade Preparation

6.1.1 General

To prepare the site for grading operations, topsoil, organic matter, existing pavements, demolished structure remnants, and other deleterious material should be stripped from the site improvement areas. Soft, wet, loose, or otherwise un-suitable soils should be removed and replaced, or potentially re-compacted in-place.

6.1.2 Demolition of Existing Structures and Utilities

All existing structures should be removed in their entirety from within the equipment pad, substation, and solar array footprints. Where existing structures fall at least 10 feet from site improvements, below grade portions of these structures may remain in place.

Existing utilities to remain in use should be rerouted around the proposed structure footprints. Remove or grout existing utilities to be abandoned prior to construction. If not removed, any pipes over 3 inches in diameter should be filled with flowable fill or grout. Otherwise, these pipes may serve as conduits for subsurface erosion resulting in formation of voids below structures. Where existing utilities are left in place and plugged within foundations, it may be necessary to undercut poorly compacted backfill to provide adequate support for foundations.

6.1.3 Equipment Pad

Excavations to final subgrade for the equipment pad should be performed in such a way that limits disturbing or loosening subgrade soils. After stripping and cutting and prior to placing pad base materials, the resulting subgrade should be firm, stable, and unyielding. Stabilization, where required, may consist of removing unsuitable material and replacement with compacted Structural Fill, or where unsuitable soils are relatively thin, drying and compacting in place.

Equipment pad soil subgrades should be proof-rolled with at least four (4) passes of a minimum 5-ton vibratory roller.

We recommend that a GEI representative observe the final preparation of all subgrades prior to equipment pad construction.

6.1.4 Access Roads

We understand that the access roads at the site will be gravel surface roads. The following roadway sections are suitable for the access roads:

- 12 inches of CTDOT M.02.03 Gravel Surface over a geotextile. Geotextile fabric for roadway underlayment should be a heavy-duty woven product, consisting of GEOTEX 200ST or an approved equivalent.

We recommend that the gravel road section be compacted with at least four (4) passes of a vibratory roller imparting an impact load of at least 10 tons. The resulting subgrade should be firm, stable, and unyielding. Water should be added to materials as needed during compaction. We note that areas of exposed soils will be highly susceptible to disturbance by moisture and equipment movements.

We recommend that the road surface be graded with a minimum cross slope of ½ inch per foot of road width to allow water to drain. Drainage ditches should be provided along the edges of the road to direct surface water and runoff away from the road and subbase.

We recommend that a GEI representative observe the final preparation of all subgrades prior to access road construction.

6.2 Excavation

Excavations will be primarily through dense glacial tills. Cobbles, small boulders, and moderately difficult excavation should be expected within native soils, especially below 4 feet deep. We expect that excavation through soils can be accomplished with conventional earthmoving equipment.

All excavations should be sloped or shored in accordance with the local, state, and federal regulations, including Occupational Safety and Health Agency (OSHA 29 CFR Part 1926) excavation trench safety standards.

Excavation below approximately 2 to 4 feet will require dewatering in most locations. We expect that this can be accomplished using filtered sumps and pumps.

The site soils will be susceptible to moisture intrusion and softening. Therefore, surface water should be controlled during construction.

6.3 Freezing Conditions

The soils at the site are frost susceptible. Therefore, if construction is performed during freezing weather, special precautions will be required to prevent the subgrade soils from freezing. Freezing of the soil beneath the foundation during construction may result in subsequent settlement of the structure.

All subgrades should be free of frost before placement of concrete. Frost-susceptible soils that have frozen should be removed and replaced with compacted Structural Fill. The footing and the soil adjacent to the footing should be insulated until they are backfilled. Soil placed as fill should be free of frost, as should the ground on which it is placed.

If slabs-on-grade or footings are built and left exposed during the winter, precautions should be taken to prevent freezing of the underlying soil.

6.4 Backfilling and Compaction

We recommend that all final cut and fill slopes be constructed at no steeper than 2H:1V grade to allow for the planting and maintenance of grass cover. These slopes should be protected and seeded as soon as practicable after they are completed to reduce the potential for surface erosion.

Recommended specifications for gradation and compaction of backfill soils are provided in the attached recommended Material Specifications (Appendix D).

Existing native glacial till soils can likely be re-used on site as Structural Fill or Ordinary Fill, provided they do not contain oversize, organic, or otherwise deleterious material and can meet the appropriate compaction and moisture requirements. Cobbles and small boulders should be expected within these soils. We caution some of these materials will be difficult to work if allowed to become wet, and placement may be very difficult during certain times of the year.

Fill imported from off site should meet the attached gradation requirements. Fill placed within structural limits, under the access roadway, equipment pad, and substation, and behind any retaining walls should meet the compaction requirements for Structural Fill. Backfill placed in non-structural areas should meet the compaction requirements for Ordinary Fill. Proposed borrow materials that fall slightly outside of these specifications may also be suitable for use, subject to review and approval by GEI.

7. Closure

7.1 Follow-on Services

We recommend that GEI be kept on the project through the final design and construction phases for the following services:

- Review geotechnical-related contractor submittals and assist in developing responses to questions from the contractor (i.e. RFI's).
- Provide periodic site visits during construction to view subgrades and consult on geotechnical-related issues that occur.

7.2 Limitations

This report was prepared for the use of the project team, exclusively. Our recommendations are based on the project information provided to us at the time of this report and may require modification if there are any changes in the nature, design, or location of the proposed PV array. We cannot accept responsibility for designs based on our recommendations unless we are engaged to review the final plans and specifications to determine whether any changes in the project affect the validity of our recommendations, and whether our recommendations have been properly implemented in the design.

Our professional services for this project have been performed in accordance with generally accepted engineering practices. No warranty, express or implied, is made.

Tables

Table 1. Recommended Geotechnical Design Parameters
Bethany Solar
Bethany, Connecticut

Soil Material	Total Unit Weight Above Water Table	Drained Friction Angle	Undrained Strength	Earth Pressure Coefficients[2]		
	γ_t (pcf)	ϕ' (degrees)	C' (ksf)	K_o	K_a	K_p
Ordinary Fill (92% Compaction)[3]	120	32	0	0.47	0.31	3.25
Structural Fill (95% Compaction)[4]	125	35	0	0.43	0.27	3.69
Native Glacial Till	125	36	0	0.41	0.26	3.85

Notes:
1. The values of soil properties in this table are based on empirical correlations using the soil classifications, laboratory index tests, and engineering judgment.
2. K_o = Coefficient of Earth Pressure at Rest K_a = Active Earth Pressure Coefficient (Rankine) K_p = Passive Earth Pressure Coefficient (Rankine).
3. For material compacted to ~92% of Modified Proctor maximum dry density in accordance with ASTM D1557.
4. For material compacted to ~95% of Modified Proctor maximum dry density in accordance with ASTM D1557.

Table 2. USDA Soil Texture, NRCS Soil Group, and Infiltration Rate
Bethany Solar
Bethany, Connecticut

Test Pit ID	Sample Depth (feet)	Percent Sand[1]	Percent Silt[1]	Percent Clay[1]	USDA Soil Texture[2]	NRCS Hydrologic Soil Group[3]	Infiltration Rate (inches/hour)[3]
TP-3 (G4)	4.7-8.7	74.6	20.9	4.5	Loamy Sand / Sandy Loam	B	1.5
TP-4 (G3)	2-5	65.9	27.1	7.1	Sandy Loam	B	1.0
TP-6 (G2)	1-4	67.1	27.0	5.9	Sandy Loam	B	1.0

Notes:
1. USDA classification of soil particle sizes (mm): Sand: 0.05 to 2, Silt: 0.002 to 0.05, Clay: <0.002. Percentage of gravel removed from results to include only sand, silt, and clay proportions.
2. USDA soil texture is based on the soil texture triangle.
3. National Resources Conservation Service (NRCS) Hydrologic Soil Group and Infiltration Rate (referred to as Rawls rate) are based on Soil Texture Class and Table 7-1 of the NRCS Part 630 Hydrology National Engineering Handbook (2009) and Rawls et al 1982 “Estimation of Soil Water Properties.”

Figures

SOURCE:

Base map prepared by BL Companies, “Sketch Plan”, Sheet No. SK-7, dated 1/2022

LEGEND:

Approximate Test Pit Location

GEI Consultants

EXPLORATION LOCATION PLAN
428 BETHMOUR ROAD
BETHANY, CT

GEI PROJECT NO: **2201295**

FIGURE NO.
1

Appendix A

Test Pit Logs

GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	CLIENT: BL Companies	TEST PIT LOG	
	PROJECT: Tritec Bethany Solar	PAGE	TP-1
	CITY/STATE: Bethany, CT	1	
	GEI PROJECT NUMBER: 2201295		

GROUND SURFACE ELEVATION (FT):			625.0	**LOCATION:**	See Plan.
NORTHING:	NM	**EASTING:**	NM	**TOTAL DEPTH:**	4.5 FT
OBSERVED BY:	Tom Rezzani			**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg			**TOTAL WIDTH:**	3.5 FT
EQUIPMENT:	HITACHI 135 G			**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy			**DATE START / END**	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0 – 1	G-1	0 - 0.7	SILTY SAND (SM); ~70% F-C sand, ~20% NP fines, ~10% F-C gravel and cobbles up to 4", brown, moist, contains organic fibers and roots. **TOPSOIL**
1 – 2	G-2	0.7 - 2.7	SANDY SILT (ML); ~50% NP fines, ~30% F-C sand, ~20% F-C gravel and cobbles, olive, damp, few organic fibers. **GLACIAL TILL**
3 – 4	G-3	2.7 - 4.5	Red seam of silt at 3'. East corner of Test Pit contains cobbles up to 6" at 3' deep. WIDELY GRADED SAND WITH GRAVEL (SW); ~80% F-C sand (mostly M-C), ~20% F-C gravel and cobbles, grayish brown, wet. **GLACIAL TILL** *Excavator refusal at 4.5' deep. Possible bedrock*

Bottom of test pit at 4.5 feet. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: Groundwater intrusion at 3.8 FT.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG**	
		PAGE 2	**TP-1**

GROUND SURFACE ELEVATION (FT):	625.0	**LOCATION:**	See Plan
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	4.5 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	3.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG



Bottom of test pit at 4.5 feet.
Pictures showing soil strata at Test Pit 1

NOTES:
IN. = INCHES NM= NOT MEASURED
FT. = FEET

GEI Consultants, Inc.
455 Winding Brook Drive
Glastonbury, CT 06033
(860) 368-5300

CLIENT: BL Companies	**TEST PIT LOG**	
PROJECT: Tritec Bethany Solar	**PAGE**	**TP-2**
CITY/STATE: Bethany, CT	1	
GEI PROJECT NUMBER: 2201295		

GROUND SURFACE ELEVATION (FT):	614.0	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	8.2 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10.5 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	5.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0 – 1	G-1	0 - 1.0	SILTY SAND (SM); ~70% F-M sand, ~30% NP fines, brown, some roots and organic fibers, moist. **TOPSOIL**
1 – 2	G-2	1.0 - 2.0	Similar to G-1, reddish brown, absent organic fibers. **SILTY SAND**
2 – 4	G-3	2.0 - 6.0	SILTY GRAVEL WITH SAND (GM); ~35% F-C gravel and cobbles, ~35% NP fines, ~30% F-C sand, gray, damp to wet. **GLACIAL TILL**
4 – 8 #		6.0 - 8.2	SILTY SAND WITH GRAVEL (SM); ~65% F-C sand, ~20% NP fines, ~15% F-C gravel and cobbles, olive to brown, soil mottling, wet. Cobbles and boulders at 6.2' deep. **GLACIAL TILL** *Excavator refusal at 8.2' deep. Possible bedrock*

Bottom of test pit at 8.2 feet. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: Groundwater intrusion at 2.3 FT.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

GEI Consultants, Inc.
455 Winding Brook Drive
Glastonbury, CT 06033
(860) 368-5300

CLIENT: BL Companies		**TEST PIT LOG**	
PROJECT: Tritec Bethany Solar		**PAGE**	**TP-2**
CITY/STATE: Bethany, CT		2	
GEI PROJECT NUMBER: 2201295			

GROUND SURFACE ELEVATION (FT):			614.0	**LOCATION:**	See Plan.
NORTHING:	NM	**EASTING:**	NM	**TOTAL DEPTH:**	8.2 FT
OBSERVED BY:	Tom Rezzani			**TOTAL LENGTH:**	10.5 FT
CHECKED BY:	Anna Hernberg			**TOTAL WIDTH:**	5.5 FT
EQUIPMENT:	HITACHI 135 G			**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy			**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG

Bottom of test pit at 8.2 feet.
Pictures showing soil strata at Test Pit 2

NOTES:

IN. = INCHES NM= NOT MEASURED

FT. = FEET

GEI Consultants	GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG** **PAGE** 1 **TP-3**

GROUND SURFACE ELEVATION (FT):	612.0	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	8.7 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	7 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	4 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0	G-1	0 - 0.8	SILTY SAND (SM); ~70% F-M sand, ~30% NP fines, dark brown, organic fibers, moist. **TOPSOIL**
1 2	G-2	0.8 - 2.5	WIDELY GRADED SAND WITH SILT AND GRAVEL (SW-SM); ~65% F-C sand, ~25% F-C gravel and cobbles, ~10% NP fines, little organic fibers, moist. Increase in cobbles at 2' deep. **GLACIAL TILL**
3 4	G-3	2.5- 4.7	SILTY GRAVEL WITH SAND (GM); ~55% F-C gravel and cobbles up to 6", ~30% F-C sand, ~15% NP fines, gray to grayish brown, moist to damp. **GLACIAL TILL**
5 6 7 8	**G-4**	4.7 - 8.7	WIDELY GRADED SAND WITH GRAVEL (SW); 49.8% F-C sand, 29.7% F-C gravel and cobbles, 20.5% NP fines. Moisture content = 9.7%. **GLACIAL TILL** Increase in boulders and cobbles at 7' deep. *Excavator refusal at 8.7' deep. Possible bedrock*

Bottom of test pit at 8.7 feet. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: Groundwater intrusion at 3.7 FT.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

GEI Consultants	GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG**	
			PAGE 2	**TP-3**

GROUND SURFACE ELEVATION (FT):	612.0	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	8.7 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	7 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	4 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG

Bottom of test pit at 8.7 feet.
Pictures showing soil strata at Test Pit 3

NOTES:
IN. = INCHES NM= NOT MEASURED
FT. = FEET

GEI Consultants	GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG** **PAGE** 1 **TP-4**

GROUND SURFACE ELEVATION (FT):	600.0	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	5.5 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	4.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0	G-1	0 - 0.7	SILTY SAND (SM); ~80% F-C sand, ~20% NP fines, dark brown, moist, organic fibers. **TOPSOIL**
1	G-2	0.7 - 1.8	SILTY SAND WITH GRAVEL (SM); ~70% F-C sand, ~15% NP fines, ~15% F-C gravel, orange-brown, moist, little organic fibers. **GLACIAL TILL**
2 3 4 5	G-3	1.8 - 5.5	SILTY SAND (SM); 56.3% F-C sand, 32.9% NP fines, 10.8% gravel and cobbles, gray-brown, moist. Moisture content = 11.6%. **GLACIAL TILL** Increase in boulders and cobbles at 4.5' *Excavator refusal at 5.5' deep. Possible bedrock*

Bottom of test pit at 5.5 feet. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: No groundwater intrusion observed.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG**	
		PAGE 2	**TP-4**

GROUND SURFACE ELEVATION (FT):	600.0	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	5.5 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	4.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG

Bottom of test pit at 5.5 feet.
Pictures showing soil strata at Test pit 4

NOTES:
IN. = INCHES NM= NOT MEASURED
FT. = FEET

GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	CLIENT: BL Companies PROJECT: Tritec Bethany Solar CITY/STATE: Bethany, CT GEI PROJECT NUMBER: 2201295	TEST PIT LOG PAGE 1	TP-5

GROUND SURFACE ELEVATION (FT):	620.5	**LOCATION:**	See Plan.
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	8.3 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	5.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0 – 1	G-1	0 - 1.2	SILTY SAND (SM); ~65% F-M sand, ~35% NP fines, brown, moist, organic fibers. **TOPSOIL**
2 – 3	G-2	1.2 - 3.2	SILTY GRAVEL WITH SAND (GM); ~65% F-C gravel and cobbles up to 12", ~20% F-C sand, ~15% NP fines, olive, damp to wet. Increase in cobbles at 2.5' deep. **GLACIAL TILL**
4 – 8	G-3	3.2 - 8.3	SILTY SAND WITH GRAVEL (GM); ~50% F-C sand, ~35% NP fines, ~15% F-C gravel and cobbles, brown, wet. **GLACIAL TILL**

Bottom of test pit at 8.3 feet. Planned extent. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: Groundwater intrusion at 2.7 FT.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

	GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG** **PAGE** 2	**TP-5**

GROUND SURFACE ELEVATION (FT):	620.5	**LOCATION:**	See Plan
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	8.3 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	5.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG

Bottom of test pit at 8.3 feet.
Pictures showing soil strata at Test Pit 5.

NOTES:

IN. = INCHES NM= NOT MEASURED

FT. = FEET

GEI Consultants, Inc.
455 Winding Brook Drive
Glastonbury, CT 06033
(860) 368-5300

CLIENT: BL Companies	TEST PIT LOG	
PROJECT: Tritec Bethany Solar	PAGE 1	TP-6
CITY/STATE: Bethany, CT		
GEI PROJECT NUMBER: 2201295		

GROUND SURFACE ELEVATION (FT):	616.0	LOCATION:	See Plan.
NORTHING: NM	EASTING: NM	TOTAL DEPTH:	7.8 FT
OBSERVED BY:	Tom Rezzani	TOTAL LENGTH:	10 FT
CHECKED BY:	Anna Hernberg	TOTAL WIDTH:	4.5 FT
EQUIPMENT:	HITACHI 135 G	DATUM VERT. / HORZ.:	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	DATE START / END	4/7/2022

DEPTH FT.	SAMPLE TYPE & ID	SAMPLE DEPTH (FT)	SOIL DESCRIPTION
0 - 1	G-1	0 - 1.0	SANDY SILT (ML); ~65% NP fines, ~35% F-C sand (mostly F), black, moist, interspersed roots and fibers. **TOPSOIL**
1 - 4.3	G-2	1.0 - 4.3	SILTY SAND WITH GRAVEL (SM); 48.6% F-C sand, 30.3% NP fines, 21.1% F-C gravel and cobbles, light brown to orange-brown , damp to wet. Cobbles at 1.5' deep. Increase in cobbles at 2.7' deep. Moisture content = 12.6%. **GLACIAL TILL**
4.3 - 8	G-3	4.3 - 7.8	WIDELY GRADED SAND WITH SILT AND GRAVEL (SW-SM); ~45% F-C sand, ~45% F-C gravel and cobbles, ~10% NP fines, gray-brown , wet. Large Boulder observed in northeast corner at 6.5' deep. **GLACIAL TILL** *Excavator refusal at 7.8' deep. Possible bedrock*

Bottom of test pit at 7.8 feet. Backfilled with excavated soil placed in lifts and tamped with excavator bucket.

Note: Groundwater intrusion observed at 3.0 FT.

F=FINE M=MEDIUM NP= NONPLASTIC NM= NOT MEASURED
C=COARSE LP=LOW PLASTICITY MP=MEDIUM PLASTICITY

GEI Consultants, Inc. 455 Winding Brook Drive Glastonbury, CT 06033 (860) 368-5300	**CLIENT: BL Companies** **PROJECT: Tritec Bethany Solar** **CITY/STATE: Bethany, CT** **GEI PROJECT NUMBER: 2201295**	**TEST PIT LOG**	
		PAGE 2	**TP-6**

GROUND SURFACE ELEVATION (FT):	616.0	**LOCATION:**	See Plan
NORTHING: NM	**EASTING:** NM	**TOTAL DEPTH:**	7.8 FT
OBSERVED BY:	Tom Rezzani	**TOTAL LENGTH:**	10 FT
CHECKED BY:	Anna Hernberg	**TOTAL WIDTH:**	4.5 FT
EQUIPMENT:	HITACHI 135 G	**DATUM VERT. / HORZ.:**	Per SK-7 / NM
WEATHER:	40-50° F Cloudy	**DATE START / END**	4/7/2022

PHOTOGRAPHIC LOG




Bottom of test pit at 7.8 feet.
Pictures showing soil strata at Test Pit 6

NOTES:
IN. = INCHES NM= NOT MEASURED
FT. = FEET

Appendix B

Laboratory Test Results

Client:	GEI Consultants, Inc.				
Project:	Bethany Solar				
Location:	Bethany, CT			Project No:	GTX-315402
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/11/22	Checked By:	jdt
Depth :	---	Test Id:	665909		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
TP-3	G4	4.67-8.67'	Moist, dark brown gray silty sand with gravel	9.7
TP-4	G3	2-5'	Moist, dark brownish gray silty sand	11.6
TP-6	G2	1-4'	Moist, brown silty sand with gravel	12.6

Notes: Temperature of Drying : 110º Celsius

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Client:	GEI Consultants, Inc.				
Project:	Bethany Solar				
Location:	Bethany, CT			Project No:	GTX-315402
Boring ID:	TP-1	Sample Type:	bag	Tested By:	amp
Sample ID:	Composite-1	Test Date:	05/05/22	Checked By:	jdt
Depth :	0.7-4.5'	Test Id:	665903		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
TP-1	Composite-1	0.7-4.5'	Moist, brown silty sand with gravel	6.7	5.7

Notes: Sample Preparation: screened through #10 sieve

Method A, pH meter used

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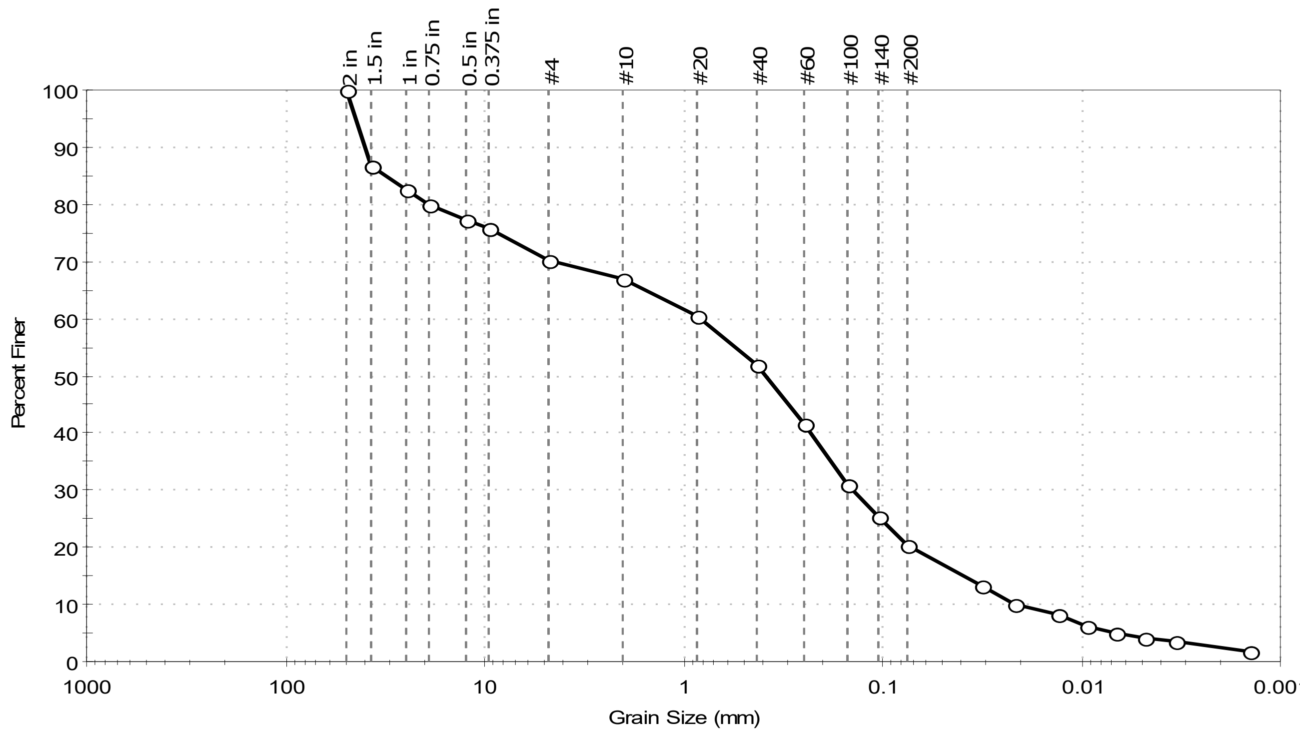
Client: GEI Consultants, Inc.			
Project: Bethany Solar			
Location: Bethany, CT		Project No:	GTX-315402
Boring ID: TP-3	Sample Type: bag	Tested By:	ckg
Sample ID: G4	Test Date: 05/11/22	Checked By:	jdt
Depth : 4.67-8.67'	Test Id: 665905		

Test Comment: ---
Visual Description: Moist, dark brown gray silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	29.7	49.8	20.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
2 in	50.00	100		
1.5 in	37.50	87		
1 in	25.00	83		
0.75 in	19.00	80		
0.5 in	12.50	77		
0.375 in	9.50	76		
#4	4.75	70		
#10	2.00	67		
#20	0.85	61		
#40	0.42	52		
#60	0.25	42		
#100	0.15	31		
#140	0.11	25		
#200	0.075	20		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0320	13		
---	0.0217	10		
---	0.0131	8		
---	0.0094	6		
---	0.0067	5		
---	0.0048	4		
---	0.0034	4		
---	0.0014	2		

Coefficients

D_{85} =31.3644 mm D_{30} =0.1419 mm
D_{60} =0.8143 mm D_{15} =0.0388 mm
D_{50} =0.3855 mm D_{10} =0.0216 mm
C_u =37.699 C_c =1.145

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Est. Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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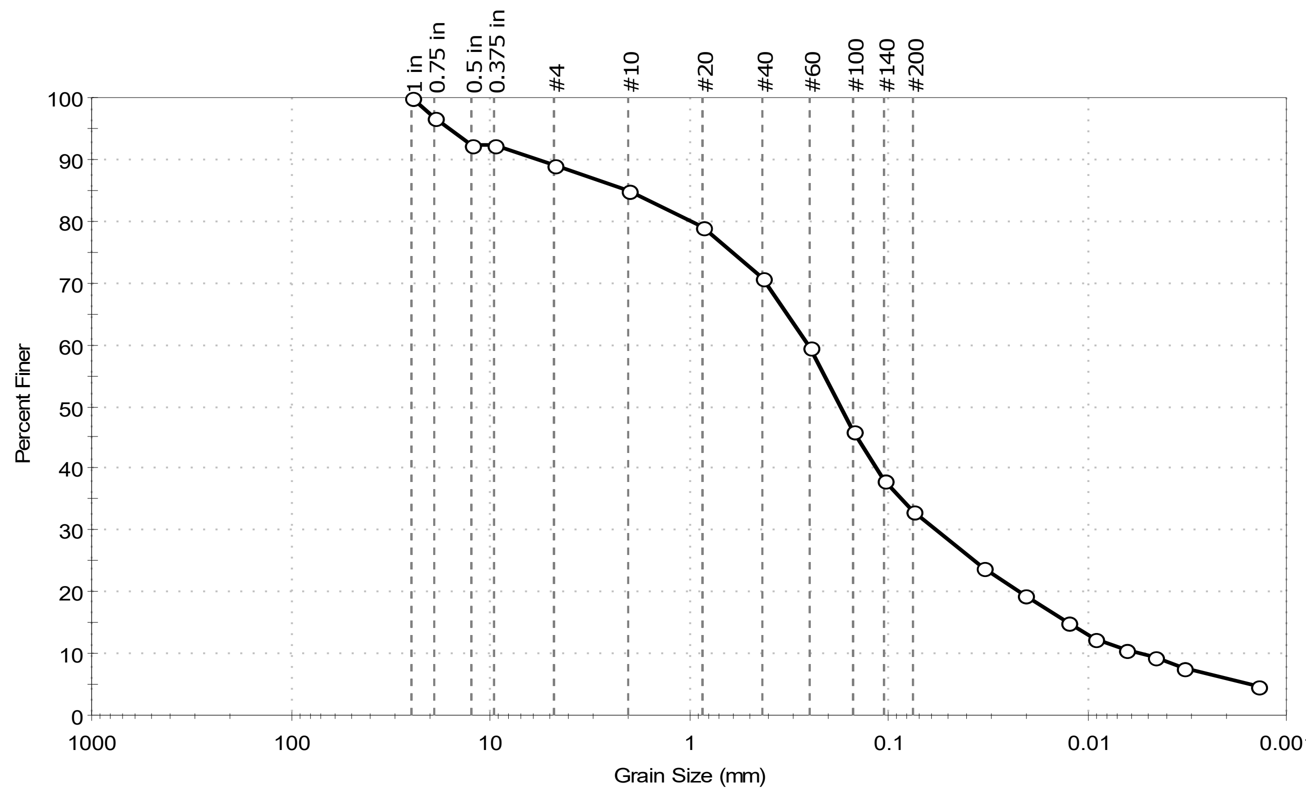
GeoTesting EXPRESS

Client:	GEI Consultants, Inc.				
Project:	Bethany Solar				
Location:	Bethany, CT			Project No:	GTX-315402
Boring ID:	TP-4	Sample Type:	bag	Tested By:	ckg
Sample ID:	G3	Test Date:	05/11/22	Checked By:	jdt
Depth :	2-5'	Test Id:	665904		

Test Comment: ---
Visual Description: Moist, dark brownish gray silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.8	56.3	32.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	92		
0.375 in	9.50	92		
#4	4.75	89		
#10	2.00	85		
#20	0.85	79		
#40	0.42	71		
#60	0.25	60		
#100	0.15	46		
#140	0.11	38		
#200	0.075	33		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0333	24		
---	0.0209	20		
---	0.0126	15		
---	0.0091	12		
---	0.0065	11		
---	0.0046	9		
---	0.0033	8		
---	0.0014	5		

Coefficients

D_{85} =2.0146 mm	D_{30} =0.0575 mm
D_{60} =0.2549 mm	D_{15} =0.0125 mm
D_{50} =0.1749 mm	D_{10} =0.0055 mm
C_u =46.345	C_c =2.358

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Est. Specific Gravity : 2.65
Separation of Sample: Sieve

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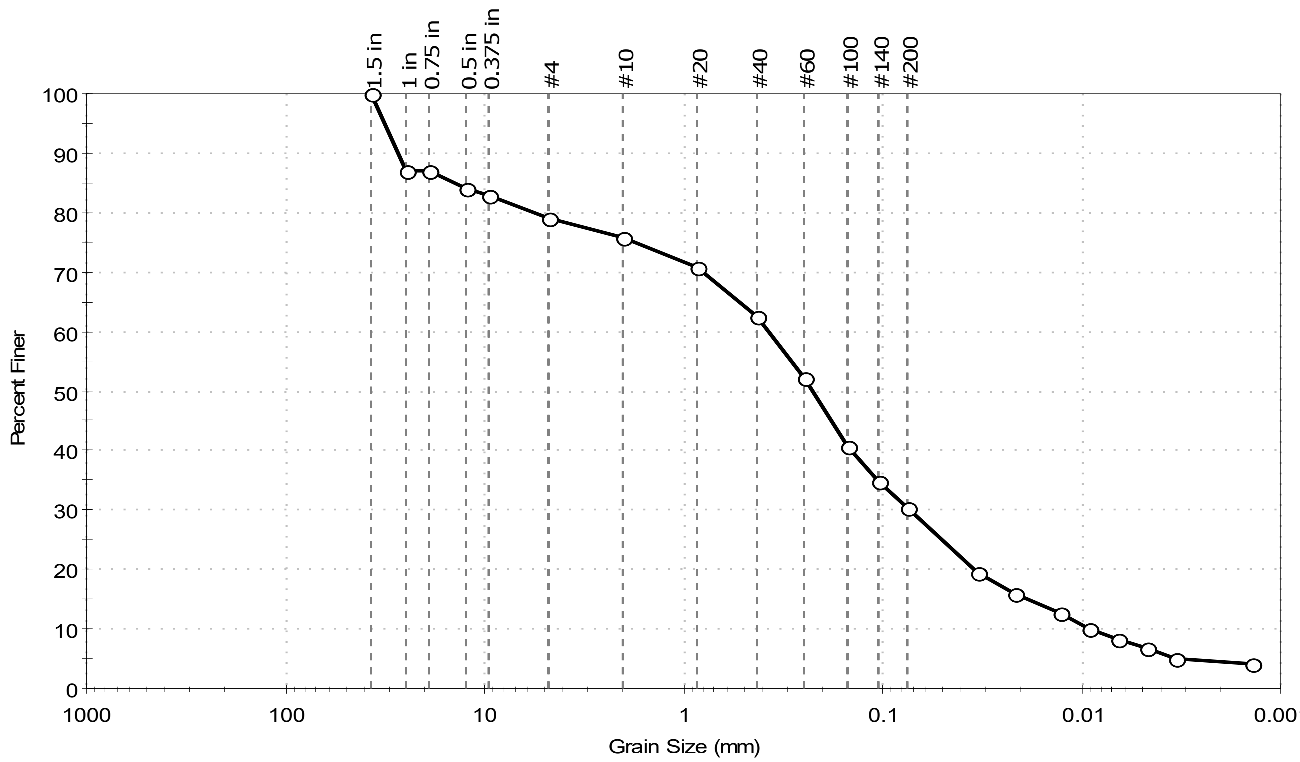
Client:	GEI Consultants, Inc.		
Project:	Bethany Solar		
Location:	Bethany, CT	Project No:	GTX-315402
Boring ID: TP-6	Sample Type: bag	Tested By:	ckg
Sample ID: G2	Test Date: 05/11/22	Checked By:	jdt
Depth : 1-4'	Test Id: 665906		

Test Comment: ---

Visual Description: Moist, brown silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	21.1	48.6	30.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	87		
0.75 in	19.00	87		
0.5 in	12.50	84		
0.375 in	9.50	83		
#4	4.75	79		
#10	2.00	76		
#20	0.85	71		
#40	0.42	63		
#60	0.25	52		
#100	0.15	41		
#140	0.11	35		
#200	0.075	30		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0334	19		
---	0.0216	16		
---	0.0128	13		
---	0.0093	10		
---	0.0066	8		
---	0.0047	7		
---	0.0034	5		
---	0.0014	4		

Coefficients

D_{85} =14.2744 mm — D_{30} =0.0733 mm

D_{60} =0.3721 mm — D_{15} =0.0184 mm

D_{50} =0.2255 mm — D_{10} =0.0092 mm

C_u =40.446 — C_c =1.569

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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Client:	GEI Consultants, Inc.
Project:	Bethany Solar
Location:	Bethany, CT
GTX#:	315402
Test Date:	05/05/22
Tested By:	amp
Checked By:	jdt

Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(\text{ohm-cm})^{-1}$
TP-1	Composite-1	0.7-4.5	Moist, brown silty sand with gravel	113,634	8.80E-06

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
Water added to sample to create a thick slurry prior to testing (saturated condition).
Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
Test conducted in standard laboratory atmosphere: 68-73 F

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GEOTESTING EPXRESS INCORPORATED
125 NAGOG PARK
ACTON MA 01720-3451
USA

Analysis No.	TS-A2210280
Report Date	09 May 2022
Date Sampled	29 April 2022
Date Received	06 May 2022
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Soil: Project: Bethany Solar; Site Location: Bethany, CT; Job Number: GTX-315402

When examined to the applicable requirements of:

ASTM D 512-12*	"Standard Test Methods for Chloride Ion in Water" Method B
ASTM D 516-16	"Standard Test Method for Sulfate Ion in Water"

Results:

ASTM D512 - Chloride Method B

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
TP-1		12.	0.0012	10.
Composite-1	0.7 – 4.5'			

NOTE: [1]Percent by weight after drying and prepared as per the Standard. *Withdrawn 2021 without Replacement

ASTM D 516 – Sulfates (Soluble)

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
TP-1		< 10.	< 0.0010	10.
Composite-1	0.7 – 4.5'			

NOTE: [1]Percent by weight after drying and prepared as per the Standard.

END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

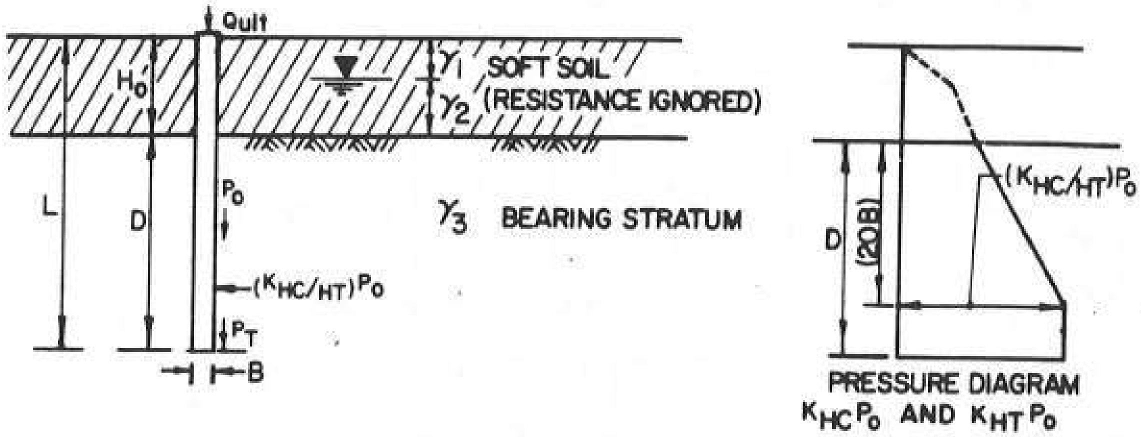
Appendix C

NAVFAC DM 7.02

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, Virginia 22332-2300

Foundations & Earth Structures

DESIGN MANUAL 7.02
REVALIDATED BY CHANGE 1 SEPTEMBER 1986



(A) ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_o}^{H=H_o+D} (K_{HC})(P_o)(\tan\delta)(S)$$

WHERE Q_{ult} = ULTIMATE LOAD CAPACITY IN COMPRESSION

P_T = EFFECTIVE VERTICAL STRESS AT PILE TIP (SEE NOTE I)

N_q = BEARING CAPACITY FACTOR (SEE TABLE, FIGURE I CONTINUED)

A_T = AREA OF PILE TIP

K_{HC} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN COMPRESSION.

P_o = EFFECTIVE VERTICAL STRESS OVER LENGTH OF EMBEDMENT, D (SEE NOTE I)

δ = FRICTION ANGLE BETWEEN PILE AND SOIL (SEE TABLE, FIGURE I CONTINUED)

S = SURFACE AREA OF PILE PER UNIT LENGTH

FOR CALCULATING Q_{all}, USE F_S OF 2 FOR TEMPORARY LOADS, 3 FOR PERMANENT LOADS. (SEE NOTE 2)

(B) ULTIMATE LOAD CAPACITY IN TENSION

$$T_{ult} = \sum_{H=H_o}^{H=H_o+D} (K_{HT})(P_o)(\tan\delta)(S)(H)$$

WHERE: T_{ult} = ULTIMATE LOAD CAPACITY IN TENSION, PULLOUT

K_{HT} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN TENSION

FOR CALCULATING T_{all}, USE $F_S = 3$ ON T_{ult} PLUS THE WEIGHT OF THE PILE (W_P), THUS $T_{all} = \frac{T_{ult}}{3} + W_P$ (SEE NOTE 2)

NOTE-1: EXPERIMENTAL AND FIELD EVIDENCE INDICATE THAT BEARING PRESSURE AND SKIN FRICTION INCREASE WITH VERTICAL EFFECTIVE STRESS P_o UP TO A LIMITING DEPTH OF EMBEDMENT, DEPENDING ON THE RELATIVE DENSITY OF THE GRANULAR SOIL AND POSITION OF THE WATER TABLE. BEYOND THIS LIMITING DEPTH (10B± TO 40B±) THERE IS VERY LITTLE INCREASE IN END BEARING, AND INCREASE IN SIDE FRICTION IS DIRECTLY PROPORTIONAL TO THE SURFACE AREA OF THE PILE. THEREFORE, IF D IS GREATER THAN 20 B, LIMIT P_o AT THE PILE TIP TO THAT VALUE CORRESPONDING TO D = 20B.

NOTE-2: IF BUILDING LOADS AND SUBSURFACE CONDITION ARE WELL DOCUMENTED IN THE OPINION OF THE ENGINEER, A LESSER FACTOR OF SAFETY CAN BE USED BUT NOT LESS THAN 2.0 PROVIDED PILE CAPACITY IS VERIFIED BY LOAD TEST AND SETTLEMENTS ARE ACCEPTABLE.

FIGURE 1
Load Carrying Capacity of Single Pile in Granular Soils

BEARING CAPACITY FACTORS - N_q

ϕ* (DEGREES)	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (DRIVEN PILE DISPLACEMENT)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q ** (DRILLED PIERS)	5	8	10	12	14	17	21	25	30	38	43	60	72

EARTH PRESSURE COEFFICIENTS K_{HC} AND K_{HT}

PILE TYPE	K_{HC}	K_{HT}
DRIVEN SINGLE H-PILE	0.5 – 1.0	0.3 – 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 – 1.5	0.6 – 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 – 2.0	1.0 – 1.3
DRIVEN JETTED PILE	0.4 – 0.9	0.3 – 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

FRICTION ANGLE - δ

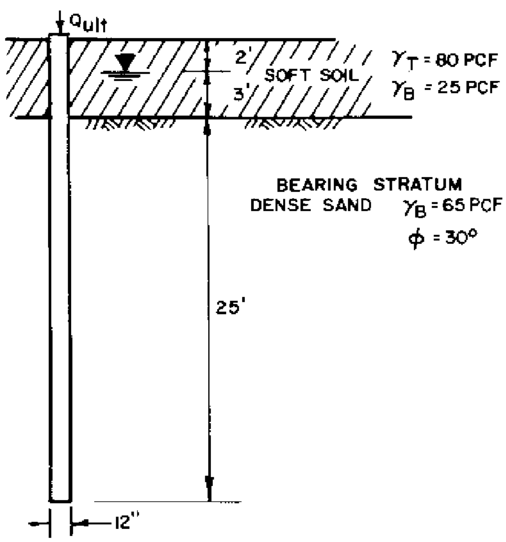
PILE TYPE	δ
STEEL	20°
CONCRETE	3/4 ϕ
TIMBER	3/4 ϕ

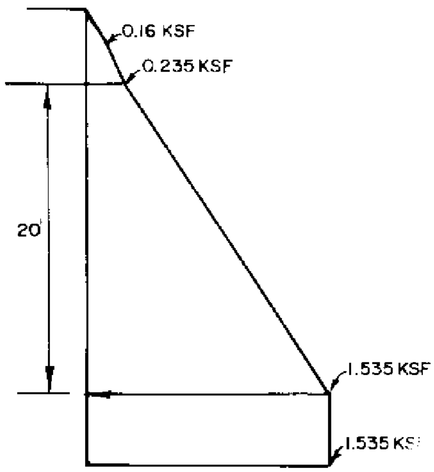
* LIMIT ϕ TO 28° IF JETTING IS USED

** (A) IN CASE A BAILER OR GRAB BUCKET IS USED BELOW GROUNDWATER TABLE, CALCULATE END BEARING BASED ON ϕ NOT EXCEEDING 28°.

(B) FOR PIERS GREATER THAN 24-INCH DIAMETER, SETTLEMENT RATHER THAN BEARING CAPACITY USUALLY CONTROLS THE DESIGN. FOR ESTIMATING SETTLEMENT, TAKE 50% OF THE SETTLEMENT FOR AN EQUIVALENT FOOTING RESTING ON THE SURFACE OF COMPARABLE GRANULAR SOILS. (CHAPTER 5, DM-7.1).

FIGURE 1 (continued)
Load Carrying Capacity of Single Pile in Granular Soils





EFFECTIVE VERTICAL STRESS, P_o, FOR PILE DESIGN

FOR A 12" DIAMETER CLOSED END, DRIVEN PIPE PILE, CONCRETE FILLED, FIND Q_{all} AND T_{all} FOR A 30 FOOT LONG PILE.

P_o MAX OCCURS AT 20B, OR 20' INTO BEARING STRATUM.

$\phi = 30°$, $N_q = 21$

$K_{HC} = 1.5$, $\delta = 20°$

$K_{HT} = 1.0$

$A_T = \pi \times 0.5^2 = 0.78$ SF

CIRCUM. AREA / lf $= 1 \times \pi = 3.14$ SF / lf

$$Q_{ult} = 1.535 \times 21 \times 0.78 + \left[(1.5 \times (\frac{0.235+1.535}{2}) \times \text{TAN } 20 \times 20 \times 3.14) + (1.5 \times 1.535 \times \text{TAN } 20 \times 5 \times 3.14) \right]$$

$$= 25.14 + \left[30.34 + 13.16\right]$$

$$= 68.64 \text{ K}$$

FOR $F_S = 3$, $Q_{all} = \frac{68.64}{3} = 22.9$ K

$$T_{ult} = 1.0 \times (\frac{0.235+1.535}{2}) \times \text{TAN } 20 \times 20 \times 3.14 + 1.0 \times 1.535 \times \text{TAN } 20 \times 5 \times 3.14$$

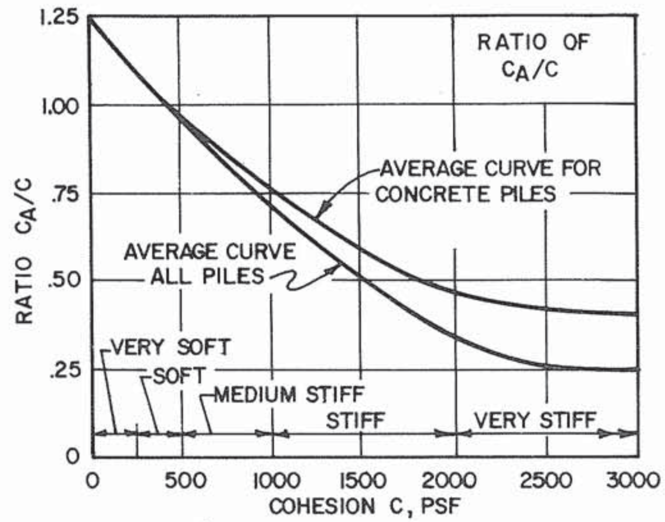
$$= 20.23 + 8.77$$

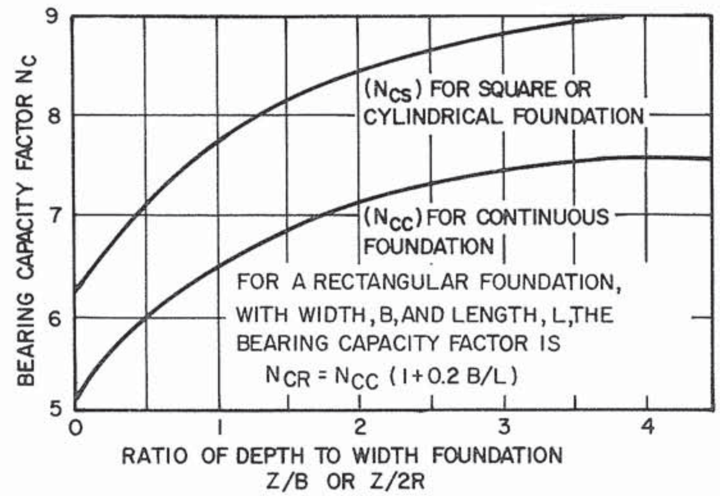
$$= 29.0 \text{ K}$$

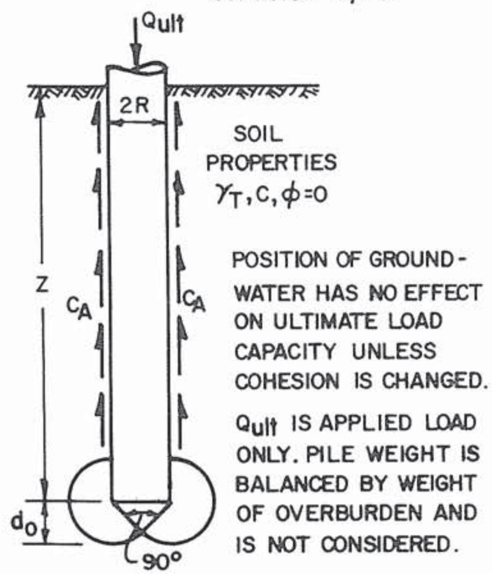
$W_P \approx .117$ K/lf $\times 30' = 3.5$ K

$T_{all} = \frac{29.0}{3} + 3.5 = 13.2$ K

FIGURE 1 (continued)
Load Carrying Capacity of Single Pile in Granular Soils







ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = C \, \underset{(N_{CC})}{(N_{CS})} \, \pi R^2 + C_A \, 2 \pi R Z$$

RECOMMENDED VALUES OF ADHESION

PILE TYPE	CONSISTENCY OF SOIL	COHESION, C PSF	ADHESION, C_A PSF
TIMBER AND CONCRETE	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 480
	MED. STIFF	500 - 1000	480 - 750
	STIFF	1000 - 2000	750 - 950
	VERY STIFF	2000 - 4000	950 - 1300
STEEL	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 460
	MED. STIFF	500 - 1000	460 - 700
	STIFF	1000 - 2000	700 - 720
	VERY STIFF	2000 - 4000	720 - 750

ULTIMATE LOAD CAPACITY IN TENSION

$T_{ult} = C_A \, 2 \pi R Z$

T_{ult} UNDER SUSTAINED LOAD MAY BE LIMITED BY OTHER FACTORS, SEE TEXT.

FIGURE 2
Ultimate Load Capacity of Single Pile or Pier in Cohesive Soils

(3) Drilled Piers. For drilled piers greater than 24 inches in diameter settlement rather than bearing capacity may control. A reduced end bearing resistance may result from entrapment of bentonite slurry if used to maintain an open excavation to the pier's tip. Bells, or enlarged bases, are usually not stable in granular soils.

(4) Piles and Drilled Piers in Cohesive Soils. See Figure 2 and Table 3. Experience demonstrates that pile driving permanently alters surface adhesion of clays having a shear strength greater than 500 psf (see Figure 2). In softer clays the remolded material consolidates with time, regaining adhesion approximately equal to original strength. Shear strength for point-bearing resistance is essentially unchanged by pile driving. For drilled piers, use Table 3 from Reference 4, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures, by the Departments of Army and Air Force, for determining side friction. Ultimate resistance to pullout cannot exceed the total resistance of reduced adhesion acting over the pile surface or the effective weight of the soil mass which is available to react against pullout. The allowable sustained pullout load usually is limited by the tendency for the pile to move upward gradually while mobilizing an adhesion less than the failure value.

Adhesion factors in Figure 2 may be very conservative for evaluating piles driven into stiff but normally consolidated clays. Available data suggests that for piles driven into normally to slightly overconsolidated clays, the side friction is about 0.25 to 0.4 times the effective overburden.

(5) Piles Penetrating Multi-layered Soil Profile. Where piles penetrate several different strata, a simple approach is to add supporting capacity of the individual layers, except where a soft layer may consolidate and relieve load or cause drag on the pile. For further guidance on bearing capacity when a pile penetrates layered soil and terminates in granular strata see Reference 5, Ultimate Bearing Capacity of Foundations on Layered Soils Under Inclined Loads, by Meyerhoff and Hanna, which considers the ultimate bearing capacity of a deep member in sand underlying a clay layer and for the case of a sand bearing stratum overlying a weak clay layer.

(6) Pile Buckling. For fully embedded piles, buckling usually is not a problem. For a fully embedded, free headed pile with length equal to or greater than 4T, the critical load for buckling is as follows (after Reference 6, Design of Pile Foundations, by Vesic):

$$P_{crit} = 0.78\ T^3 f \quad \text{for } L \geq 4T$$

where: P_{crit} = critical load for buckling

f = coefficient of variation of lateral subgrade reaction (see Figure 10)

T = relative stiffness factor (see Figure 10)

L = length of pile.

TABLE 3
Design Parameters for Side Friction for Drilled Piers in Cohesive Soils

Design Category	Side Resistance		Remarks
	C_A/C	Limit on side shear - tsf	
A. Straight-sided shafts in either homogeneous or layered soil with no soil of exceptional stiffness below the base			
1. Shafts installed dry or by the slurry displacement method	0.6	2.0	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.3(a)	0.5(a)	(a) C_A/C may be increased to 0.6 and side shear increased to 2.0 tsf for segments drilled dry
B. Belled shafts in either homogeneous or layered clays with no soil of exceptional stiffness below the base			
1. Shafts installed dry or by the slurry displacement methods	0.3	0.5	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.15(b)	0.3(b)	(b) C_A/C may be increased to 0.3 and side shear increased to 0.5 tsf for segments drilled dry

Appendix D

Recommended Material Specifications

Recommended Material Specifications
Bethany Solar
428 Bethmour Road
Bethany, CT

Structural Fill and Ordinary Fill shall consist of hard, durable sand and gravel, free of clay, organic matter, surface coatings, and other deleterious materials. Soil finer than the No. 200 sieve (the "fines") should be nonplastic. On-site materials can be re-used as Structural Fill or Ordinary Fill, provided they can meet the appropriate compaction and moisture requirements indicated below and do not contain deleterious materials. Soils to be used as fill imported from off site should also meet the gradation requirements given below.

Structural Fill

Structural Fill should consist of hard, durable sand and gravel. It should be free of clay, organic matter, surface coatings, and other deleterious materials. Soil finer than the No. 200 sieve (the "fines") should be nonplastic. Structural Fill shall meet the following gradation requirements:

Sieve Size	Percent Passing by Weight
3 inches	100
1 - ½ inch	55 – 100
No. 4	35 – 85
No. 16	20 – 65
No. 50	5 – 40
No. 200 (fines)	0 – 10

Structural Fill should be compacted in maximum 12-inch-thick, loose lifts to at least 95 percent of the maximum dry density determined in accordance with ASTM D1557 (Modified AASHTO Compaction). The moisture content should be held to within +/- 3 percent of optimum moisture content (as determined by ASTM D1557).

Ordinary Fill

Ordinary fill should consist of hard, durable sand and gravel, free of clay, organic matter, surface coatings, and other deleterious materials. Soil finer than the No. 200 sieve (the “fines”) should be nonplastic. Ordinary Fill shall meet the following gradation requirements:

Sieve Size	Percent Passing by Weight
6 inches	100
3 inches	80 – 100
No. 4	20 – 100
No. 200 (fines)	0 – 20

Ordinary fill should be compacted in maximum 12-inch-thick, loose lifts to at least 92 percent of the maximum dry density determined in accordance with ASTM D1557 (Modified AASHTO Compaction). The moisture content should be held to within +/- 3 percent of optimum moisture content (as determined by ASTM D1557).

Geotextile Fabric

Geotextile fabric for roadway underlayment (if used, refer to Section 6.1.4) should be a heavy-duty woven fabric, consisting of GEOTEX 200ST or an approved equivalent product.