Appendix K

Additional Due Diligence Reports

Geotechnical Engineering Report

Hoffman Solar Project
Franklin, Simpson County, Kentucky
March 2, 2021
Terracon Project No. 57205066

Prepared for:
OPDE Energy C/O
Horus Renewables Corp
Jupiter, FL

Prepared by:
Terracon Consultants, Inc.
Louisville, KY

March 2, 2021

OPDE Energy C/O
Horus Renewables Corp
110 Front Street, Ste#300
Jupiter, FL 33477

Attn: Mr. Jorge E. Medina, Project Development
P: (210) 838-2920
E: jmedina@opdenergy.com

Re: Geotechnical Engineering Report
Hoffman Solar Project
Tyree Chapel Road
Franklin, Simpson County, Kentucky
Terracon Project No. 57205066

Dear Mr. Medina:

Terracon Consultants, Inc. (Terracon) have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with our proposal number 57205066 dated September 11, 2020. This report presents the findings of our exploration and provides geotechnical recommendations concerning site preparation and earthwork, solar panel foundations, and substation foundations. It should be noted that the solar panel foundation recommendations are considered preliminary until a full-scale pile load testing program is completed. We have prepared **Preliminary Karst Assessment** reports for Terracon project number 57195114 delivered separately for the Hoffman and Summer parcels dated August 4, and November 30, 2020.

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

Sincerely,
Terracon Consultants, Inc.

Sadra Javadi, Ph.D.
Geotechnical Engineer

Benjamin W. Taylor, P.E., P.G.
Principal, Regional Manager

This item has been digitally signed and sealed by Benjamin W. Taylor, P.E. on the date adjacent to the seal.
Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Subject Matter Expert (SME) Review By: James M. Jackson, P.E. (FL)

REPORT TOPICS

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

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

Geotechnical Engineering Report

Hoffman Solar Project
Tyree Chapel Road
Franklin, Simpson County, Kentucky
Terracon Project No. 57205066
March 2, 2021

INTRODUCTION

Terracon Consultants, Inc. (Terracon) is pleased to submit this report detailing the geotechnical engineering services performed for the proposed solar farm to be located near Tyree Chapel Road in Franklin, Simpson County, Kentucky. The site location is included in **Site Location and Exploration Plan** section of this report. The purpose of these services is to provide subsurface information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Site preparation and earthwork
- Unpaved access roads
- Groundwater conditions
- Seismic considerations
- Foundation design and construction

Our scope of work for this phase of the project included the following:

- 36 soil borings to depths ranging from about 4 to 23 feet below the existing ground surface (bgs);
- Field Electrical Resistivity testing at 11 locations;
- Laboratory Thermal Resistivity dry-out curve testing conducted on bulk samples obtained at 7 boring locations. The samples were obtained from depths of approximately 1 to 4 feet bgs;
- Corrosion testing performed on bulk samples obtained at 11 locations from approximately 2 to 3 feet bgs;
- Standard Proctor compaction testing on bulk samples obtained at 7 locations from approximately 1 to 4 feet bgs.
- Modified California Bearing Ratio (CBR) testing on bulk samples obtained at 7 locations from approximately 0 to 1 feet bgs.
- Laboratory classification and index property testing of soil samples;

The locations of the borings and field electrical resistivity are shown on the **Exploration Plan**. A log of each boring and results of laboratory testing are included in the **Exploration Results**.

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

SITE CONDITIONS

ITEM	DESCRIPTION
Parcel Information	The project site is approximately 540 acres of land located near Tyree Chapel Road in Franklin, Simpson County, Kentucky. The latitude and longitude for the approximate center of the site is 36.661241°N, -86.534179°W. See **Site Location** and **Exploration Location Plan**.
Existing Improvements	The site is primarily agricultural land. A pond is shown in the northwest portion of the site. Tyree Chapel Road and a transmission line cross the site in a north-south direction.
Current Ground Cover	The project site is covered with crops, grass, isolated stands of trees presenting on the south side of the fields, residential houses, roads/driveways, and ponds.
Existing Topography	Site-specific topographic survey was not available at the time of this report. Based on review of topographic elevation in Google Earth Pro™ and our observation during exploration, the site appears to generally be hilly. The site appears to gently slope away from the south toward north-east and north-west from approximate elevation of +755 to +690 feet.

PROJECT DESCRIPTON

Item	Description
Proposed Project	The Client intends to develop a of 69.3 Megawatt (AC) photovoltaic (PV) solar facility. The facility will consist of solar panels and various other equipment and appurtenances associated with the substation and O&M Building (e.g. switchgear, transformers, inverters, and overhead and underground electrical conveyance). We understand the panel arrays will cover about 500 acres of the 540-acre site. If the panel array area increases from our assumption, additional field and laboratory testing will be required.
Proposed Construction	We anticipate the project will include the construction of ground-mounted solar panels on steel racks founded on driven W-Section steel beams (W6x9 or similar). Electrical equipment and substation elements are anticipated to be supported on concrete slabs-on-grade, spread footings, or drilled shafts.
Maximum loads	Structural loads were not provided, but the following loads have been estimated based on our experience with similar projects using fixed rack systems: ■ Downward: 4 kips ■ Uplift: 2 kips ■ Lateral: 3 kips ■ Substation Structures: Assuming required contact pressure is about 1,500 psf for a mat foundation with approximate dimension of minimum 50 ft by maximum 150 ft ■ O&M Building: 5 kips per linear foot (klf) ■ Column loads: 150 kips
Grading/Slopes	A site grading plan has not been provided at the time of this report. It is anticipated that the site grading will be within +/- 2 feet of existing grade. Localized high and low areas may require greater cut and/or fill.
Other Improvements	Other improvements associated with this project are not specified at this time, but could include electrical equipment pads to support switchgear, inverters, transformers, and buried utilities.
Access Roadways	We anticipate that access road cross sections used for construction of the project will be the responsibility of the EPC, and that only post construction traffic with an allowable rut depth of 2 inches is what we are to develop recommended aggregate-surfaced thicknesses for in this report. We anticipate low-volume, aggregate-surfaced and native soil access roads will be subject to post-construction maximum vehicle loads of 30,000 lbs. and will travel over the access roads only once per week.

GEOTECHNICAL CHARACTERIZATION

Geology

Formation	Description
Ste. Genevieve Limestone (Msg):	Predominantly oolitic; some crystalline, argillaceous, and fossiliferous, detrital interbeds. Light-gray to almost white, oolitic, medium crystalline, massive to thin-bedded or slightly cross bedded; contains thin shale partings. Gray to white, weathers slightly darker; where exposed to much direct sunlight, weathered rock may be white, commonly speckled red-brown by iron oxide stain; mostly thick bedded and massive but ranges to thin bedded. Upper limestone layers weather to a thick deep-red or maroon clay containing abundant residual chert. Much of residual chert weathers to chalky fragments. Ste. Genevieve grades imperceptibly into underlying St. Louis Limestone.
St. Louis Limestone (Msl):	Limestone, light- to dark-gray, fine- to medium -crystalline; contains blue gray chert nodules, particularly abundant in uppermost part; several light- to medium-gray, oolitic limestone beds in upper part of unit; scattered colonies of corals in middle and lower part; scattered gypsum and anhydrite seams in lower part. Formation weathers to dark-reddish -brown chert residuum. Grades upward into Ste. Genevieve.

The project site is mapped within an area reported by the Kentucky Geological Survey (KGS) to have a very high karst potential. Multiple sinkholes are mapped by the KGS within 1-mile of the site. Further, there are several sinkholes mapped within the site boundaries.

Due to the Karst potential at the site, Terracon completed a preliminary karst survey and assessment consisting of desktop data review, field reconnaissance, Geophysical ERI exploration and confirmatory ATP drilling. The results of these studies are presented by our **Preliminary Karst Assessment** reports for Terracon project number 57195114 delivered separately for the Hoffman and Summer parcels dated August 4, and November 30, 2020.

The purpose of the preliminary karst survey and assessment was to identify, locate, and characterize existing karst features with particular emphasis on open throat and/or active sinkhole development which could impact the project development infrastructure. Our reports provided a summary of the observed features and our recommendations for the avoidance area and required buffer zone around each feature for the construction purposes.

GeoModel - 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 of this report.

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

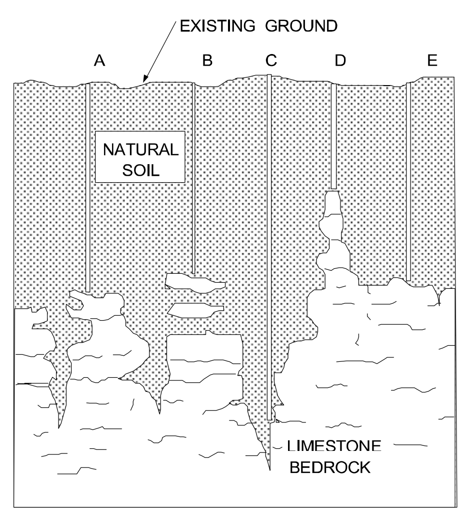
Model Layer	Layer Name	General Description
1	**Cohesive Soil**	Lean Clay (CL) to Fat Clay (CH), brown to reddish brown, medium stiff to stiff
2	**Cohesive Soil**	Lean Clay (CL) to Fat Clay (CH), with rock fragments, brown to reddish brown, very stiff to hard

Borings at 20 of 36 exploration locations were advanced to auger refusal at depths of about 12½ to 23 feet below existing grade. Auger refusal is defined as the depth below the ground surface at which a test boring can no longer be advanced with the soil drilling technique being used. Karst bedrock, such as the limestone formations underlying the site are known for producing several obstructions that can cause the augers to refuse above sound bedrock.

These obstructions can range from floaters to rock pinnacles as illustrated in Examples A, B, C, and D in the figure. Depth to competent bedrock can vary greatly over short distances. The possibility of varying depths to bedrock should be considered when developing the design and construction plans for this project.

Due to the residual nature of the overburden soils, rock fragments, chert, and cobbles should be expected. Therefore, it is possible that piles driven into the overburden soils and weathered rock stratum might encounter difficult driving. We recommend a pile driving and testing program be developed to assess the difficulty of piles penetrating the onsite soils. The pile test program should include pre-drilling.

AUGER REFUSAL ILLUSTRATION



THIS FIGURE IS FOR ILLUSTRATIVE PURPOSES ONLY AND DOES NOT NECESSARILY DEPICT THE SPECIFIC BEDROCK CONDITIONS AT THIS SITE

Groundwater

The boreholes were observed while drilling and after completion for the presence and level of groundwater. Groundwater was not observed in any of the borings while drilling, or for the short duration the borings could remain open. However, this does not necessarily mean the borings terminated above groundwater. Due to the relatively low permeability of the soils encountered in the boring, a relatively long period of time may be necessary for a groundwater level to develop and stabilize in a borehole in these materials. Long-term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

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. In particular, groundwater will tend to perch over the near- surface clayey and hardpan sands during and following periods of prolonged or intense rainfall. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

GEOTECHNICAL OVERVIEW

Contributory Risk Components

ITEM	DESCRIPTION
Additional Exploration and Testing	A full-scale pile load testing (PLT) program should be considered as the project design progresses. The results of a full scale PLT program in conjunction with soil test boring/test pit results are often successful in reducing the design embedment depth when compared to designs solely based on explorative results and analytical methods.
Suitability Statement	The borings generally encountered medium stiff to hard cohesive soils with varying amounts rock fragments to the maximum termination depth of 23 feet. The proposed site appears suitable for the use of driven steel W-Section steel piles for the support of the proposed solar arrays, however, there is a likelihood of encountering difficulties during pile driving due to rock fragments, floaters, and shallow bedrock.
Soil Conditions	Our borings initially encountered a surface layer with topsoil up to approximately 12 inches thick. The subsurface profile at the project site consists of generally medium stiff to hard, lean and fat clay. Auger refusal was encountered in 20 borings at depths of 12½ to 23 feet on apparent limestone bedrock.
Access	Wet surface conditions, following extended periods of rainfall, could create access issues for rubber-tire vehicles. The site will generally be more accessible in the summer and early fall (the typical dry season in Kentucky) and less accessible in the late fall through early spring (the typical wet season in Kentucky).
Grading	We anticipate minimal site grading will be required. On-site materials appear to generally be suitable for re-use as engineered fill or backfill, except the upper 18 inches of topsoil. Alternatively, these materials could be replaced with imported soils containing an appropriate moisture content. We expect localized areas of unsuitable conditions will be encountered prior to placing fill and within the subgrade for roadways and shallow foundations that are planned. Stabilization measures, such as over-excavation and replacement, should be expected. We assume grading will require less than 2 feet of excavation or fill placement.
Groundwater	Groundwater was not observed within during drilling or for the short duration the borings were allowed to remain open. However, this does not necessarily mean the borings terminated above groundwater. Due to the relatively low permeability of the soils encountered in the boring, a relatively long period of time may be necessary for a groundwater level to develop and stabilize in a borehole in these materials. Long-term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

ITEM	DESCRIPTION
Site Drainage	Final site grading may impact the drainage within the site. A drainage study should be performed once a grading plan has been finalized to review potential drainage or flooding issues.
Karst Potential	The project site is mapped within an area reported by the Kentucky Geological Survey (KGS) to have a very high karst potential. Multiple sinkholes are mapped by the KGS within 1-mile of the site. Further, there are several sinkholes mapped within the site boundaries. Due to the Karst potential at the site, Terracon completed a preliminary karst survey and assessment consisting of desktop data review, field reconnaissance, Geophysical ERI exploration and confirmatory ATP drilling. The results of these studies are presented by our **Preliminary Karst Assessment** reports for Terracon project number 57195114 delivered separately for the Hoffman and Summer parcels dated August 4, and November 30, 2020. The purpose of the preliminary karst survey and assessment was to identify, locate, and characterize existing karst features with particular emphasis on open throat and/or active sinkhole development which could impact the project development infrastructure. Our reports provided a summary of the observed features and our recommendations for the avoidance area and required buffer zone around each feature for the construction purposes.
Frost Heave Potential	Based on the provided information, the solar arrays for this project will be supported by driven piles. The driven piles should be designed to resist design loads including compression, uplift, frost heave action, and lateral forces. The fine-grained materials encountered in the borings consisted primarily of low to high plasticity clay (CL - CH). An active adfreeze depth of 1.1 feet and an adfreeze value of 1,500 psf should be used for uplift load produced by frost heave of the piles in pile embedment analysis. The typical frost protection depth in Franklin, Simpson County, Kentucky for shallow foundation design frost considerations is 24 inches.
Corrosion Hazard / Sulfate Attack	Based on the results of our laboratory chemical testing, the soils have a negligible classification for sulfate exposure according to the criteria in ACI Design Manual 318, Chapter 19, Table 19.3.1.1. The results of our laboratory testing of soil chemical properties are presented in the attachments and are expected to assist a qualified engineer to design corrosion protection for the production piles and other project elements.
Shrink-Swell (Expansive Soil) Hazards	Moderate to high plasticity clays encountered in our borings are susceptible to shrinkage and swell with variations in water contents, and these behaviors could adversely affect lightly loaded grade supported structures. Further impact of high plasticity clay soils should be evaluated with a program of Atterberg limits testing and swell testing in areas that will include grade supported structures. Depending on the final grading plan, remedial measures may be implemented to limit subgrade volume change potential, such as over-

ITEM	DESCRIPTION
	excavation and replacement with 2-feet of low volume change (LVC) materials, treatment with a chemical admixture, etc.
Excavation Hazards	Based on the results of our borings, we expect general instability in the form of caving, sloughing, and raveling to be encountered in excavations. Excavations will likely require bracing, sloping, and/or other means to create safe and stable working conditions.
Slope Hazards	Based on review of Topographical Mapping, the parcels are lying generally on a flat area with gentle slopes. As such, slope stability hazards are expected to be minimal. However, depending on any cut-slopes or fill embankments that are part of planned grading, localized areas may require stabilization and should be reviewed as part of a final geotechnical evaluation.
Anticipated Pile Drivability	Based on the subsurface conditions encountered, the soils appear to be suitable for pile installation and support of planned solar panels. There is a likelihood of encountering difficulties during pile driving due to rock fragments, floaters, and shallow bedrock. Auger refusal on apparent limestone bedrock was encountered in 20 of the borings at depths of about 12½ to 23 feet below existing ground surface. Drivability in the weathered and fragments portions of limestone bedrock would be difficult or result in driving refusal, and pre-drilling could be required. The design phase pile load test program should evaluate areas with differing depths of bedrock and characterize the drivability with depth, particularly zones in apparent weather bedrock strata.
General Construction Considerations	The near-surface soils are moderately moisture sensitive and subject to degradation with exposure to moisture. To the extent practical, earthwork should be performed during warmer and drier periods of weather to reduce the amount of necessary subgrade remedial measures for soft and unsuitable conditions beneath access roadways, equipment pads, etc.

PILE FOUNDATIONS TO SUPPORT PANEL RACKING

We understand that driven pile foundations are the preferred foundation type for support of the panel racking system. In our opinion, traditional driven pile foundations could be used for the panel racking system. We have provided preliminary geotechnical parameters for foundation evaluation based on the results of our subsurface characterization.

We have performed preliminary geotechnical analyses for driven pile foundations to support typical PV panel racking system. The actual foundation design should be performed with consideration of a preconstruction full-scale pile load testing (PLT) program. The results of a PLT program performed in conjunction with subsurface site characterization are usually successful in reducing the design embedment depth and pile sections when compared to design based on site characterization and analytical methods alone.

The recommended PLT program should be performed when more details regarding foundation type and loads become available. If possible, the PLT program should also be performed using the intended production pile properties including section size; section shape; surface texture (e.g. bare steel, 3mil galvanized, etc.); and installation methods, equipment, and procedures. The final structural design for pile foundations should consider anticipated steel loss due to corrosion and the design loads provided by the racking manufacturer.

Adfreeze and Frost Considerations for Foundations

Based on the provided information, the solar arrays for this project are anticipated to be supported by driven piles. The driven piles should be designed to resist design loads including compression, uplift, frost heave action and lateral forces. The majority of the soils on this site are frost susceptible. The typical frost depth in the Franklin, KY area for foundation design frost considerations is 24 inches. However, due to relatively deep groundwater level (greater than 10 feet), the frost heave potential is considered to be negligible.

If frost action needs to be eliminated in critical grade supported slab or mat foundation areas, we recommend the use of non-frost susceptible (**NFS**) fill in all or portions of the conventional frost depths of 2 feet 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 for grade supported structures:

- 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 access roadways, and connect them to the storm drainage system.
- Grade clayey subgrades, so groundwater potentially perched in overlying more permeable subgrades, such as sand or aggregate base, slope toward a site drainage system.

- Place NFS fill as backfill beneath slabs and access roadways 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.

Preliminary Axial Capacities

The recommended capacities are based on the results of our widely-spaced soil explorations and other noted assumptions. Subsequent analyses will be required once PLT is completed and other design considerations are more fully defined. Therefore, the results of the analyses described below should not be used for design. Rather, these capacities are intended to assist in roughly evaluating construction costs and development viability for the proposed project.

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

Greater quantities of steel (i.e. thicker sections, greater pile lengths) may be required for foundation support in the area. We expect that the results of a full-scale pile load test program could demonstrate more favorable geotechnical parameters and, consequently, a more cost-effective final design for the racking system foundations.

The ultimate axial capacity of the straight sided pile in compression can be determined by the following equation:

$$Q_u = Q_s + Q_p = \sum f_i A_{si} + qA_p$$

Where:
Q_u = ultimate axial capacity in compression (lb)
Q_s = ultimate skin-friction resistance (lb)
Q_p = ultimate end bearing (lb)
f_i = ultimate unit stress transfer in skin friction (lb/ft^2) in depth zone
q = ultimate unit stress transfer in end bearing (lb/ft^2)
A_{si} = side surface area of the pile (ft^2) (pile perimeter x depth interval)
A_p = gross end area of the pile (ft^2)

The following preliminary geotechnical parameters can be used to estimate the capacity of driven W-section pile foundations. These values should also be suitable to prepare a full-scale pile load testing program which is recommended as part of the overall project design. Final design values will vary from the preliminary estimates below.

Layer Depth (ft)	Ultimate unit skin friction (psf)	Ultimate unit toe-bearing resistance (psf)
Zone A (B-1, B-2, B-6, B-8, B-9)		
0 to 1 [1]	---	---
1 to 6	780	10,800
6 to 13	1,750	31,500
13 to 20 [2]	2,000	50,000
Zone B (B-3, B-4, B-5, B-7, B-10)		
0 to 1 [1]	---	---
1 to 3	780	---
3 to 6	1,100	19,800
6 to 20	1,750	31,500
Zone C (B-11, B-12, B-19, B-20)		
0 to 1 [1]	---	---
1 to 3	950	---
3 to 6	1,000	18,000
6 to 13	1,750	31,500
13 to 20 [2]	2,000	50,000
Zone D (B-13, B-14)		
0 to 1 [1]	---	---
1 to 6	750	9,000
6 to 13	1,100	19,800
13 to 16	1,750	31,500
16 to 20 [2]	2,000	50,000
Zone E (B-15, B-16, B-17, B-18)		
0 to 1 [1]	---	---
1 to 3	700	---
3 to 6	780	10,800
6 to 20	1,750	31,500

1. The upper 1 foot should be neglected in pile design due to frost heave.
2. Appropriate for pile toe bearing at depths of at least 5 feet below the ground surface. The ultimate end bearing capacity values are selected based on the type of the soil/rock and our experience with similar geology. We assumed that section W6X9 would be utilized for the pile foundations.

The recommended geotechnical design parameters in this table are based on average conditions encountered in our borings. Additional subsurface exploration and pile load testing should be performed to determine actual design parameters across the site.

The axial tensile (pull-out) capacity can be developed from skin friction while the axial compressive capacity can be developed from skin friction and end bearing. The above indicated allowable skin friction is appropriate for uplift and compressive loading. The skin friction perimeter can be calculated using the perimeter of the pile which equals twice the sum of the flange width and web depth. The end bearing is applicable for piles founded at depths greater than 6 feet below existing ground surface.

For normal load cases, we suggest that a factor of safety of 2.0 be used against the provided ultimate resistance values for determining allowable skin friction values, and a factor of safety of 3.0 be used for determining the allowable end bearing values above. A pile load test program can be performed to refine the side friction parameters and to utilize a relatively lower factor of safety for design. Terracon can assist with review of extreme event loading combinations to provide recommendations on desired factors of safety.

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

Preliminary Lateral Pile Capacity

We understand that the structural engineer may perform p-y based analyses to model the soil structure interaction for driven pile foundations subjected to lateral load. We developed p-y models and parameters for this use based on the results of our subsurface investigation. L-Pile will estimate values of soil modulus (k_h) and strain (ε_{50}) based on strength; default values for both should be used. These values are presented in the table below:

Layer Depth (feet)	(P-y) Curve Type Model	Effective Unit Weight (pcf)	Undrained Cohesion, c (psf)	Uniaxial Compressive Strength (psi)	Strain Factor ε_{50}	RQD (%)	Initial Modulus Rock	P-Multiplier [1]
Zone A (B-1, B-2, B-6, B-8, B-9)								
0 to 2	Stiff Clay without Free Water (Reese)	120	1,200	---	Default	---	---	0.7
2 to 6		120	1,200	---	Default	---	---	1.0
6 to 13		125	3,500	---	Default	---	---	1.0
13 to 20 [2]	Weak Rock (Reese)	135	---	100	0.0005	10	50,000	1.0
Zone B (B-3, B-4, B-5, B-7, B-10)								
0 to 2	Stiff Clay without Free	110	1,200	---	Default	---	---	0.7
2 to 3		110	1,200	---	Default	---	---	1.0
3 to 6		120	2,200	---	Default	---	---	1.0

Layer Depth (feet)	(P-y) Curve Type Model	Effective Unit Weight (pcf)	Undrained Cohesion, c (psf)	Uniaxial Compressive Strength (psi)	Strain Factor ε_{50}	RQD (%)	Initial Modulus Rock	P-Multiplier [1]
6 to 20	Water (Reese)	125	3,500	---	Default	---	---	1.0
Zone C (B-11, B-12, B-19, B-20)								
0 to 2	Stiff Clay without Free Water (Reese)	120	1,900	---	Default	---	---	0.7
2 to 3		120	1,900	---	Default	---	---	1.0
3 to 6		125	2,000	---	Default	---	---	1.0
6 to 13		125	3,500	---	Default	---	---	1.0
13 to 20 [2]	Weak Rock (Reese)	135	---	100	0.0005	10	50,000	1.0
Zone D (B-13, B-14)								
0 to 2	Stiff Clay without Free Water (Reese)	120	1,000	---	Default	---	---	0.7
2 to 6		120	1,000	---	Default	---	---	1.0
6 to 13		120	2,200	---	Default	---	---	1.0
13 to 16		125	3,500	---	Default	---	---	1.0
16 to 20 [2]	Weak Rock (Reese)	135	---	100	0.0005	10	50,000	1.0
Zone E (B-15, B-16, B-17, B-18)								
0 to 2	Stiff Clay without Free Water (Reese)	120	880	---	Default	---	---	0.7
2 to 3		120	880	---	Default	---	---	1.0
3 to 6		120	1,200	---	Default	---	---	1.0
6 to 20		125	3,500	---	Default	---	---	1.0

1. P-multiplier of 0.7 should be applied to the upper 2-feet of near-surface soils to account for potential strength losses due seasonal effects.
2. Our scope of work did not include rock coring. For the limestone stratum below the explored depth, we assumed a preliminary parameter based on our experience with similar projects. For the final design, rock coring should be performed to confirm the strength parameters of the rock.

The above indicated effective unit weight, effective friction angle, and L-Pile parameter contain no factor of safety. These parameters are based on correlations with SPT results, published values, and our experience with similar soil types. Existing p-y models typically under-predict the lateral capacity of shallow driven piles. Therefore, the P-multiplier is most likely higher but would need to be confirmed based on results of site-specific load test results.

Driven Pile Construction Considerations

The contractor should select a driving hammer and cushion combination which can install the selected piling without overstressing the piles. The hammer should have a rated energy in foot-pounds at least equal to 15 percent of the design compressive load capacity in pounds. The contractor should submit the pile driving plan and the pile hammer-cushion combination to the engineer for evaluation of the driving stresses in advance of pile installation. During driving a maximum of 10 blows per inch is recommended to reduce the potential of damage to the piles.

Our exploration encountered auger refusal and refusal to sampler penetration at depths of 12½ to 23 feet. Due to the karst nature of the site, residual soils with rock floaters, bedrock pinnacles, and areas of shallow bedrock should be expected. These conditions would cause difficult driving and refusal to driving conditions that may require predrilling. If additional capacity needs to be developed within the rock strata or refusal occurs at shallowed depths in areas, predrilling through the "floaters" or weathered rock layers would likely be required depending on the drivability of the piles. If practical refusal is experienced above the anticipated bedrock rock surface elevation, the pile may be on a boulder or other obstruction and a replacement pile should be driven. If this occurs, the conditions should be evaluated by Terracon during the pile driving operations.

Difficult driving could also be encountered in the very stiff soils and weathered bedrock strata. Consideration should be given to using protective points and/or flange stiffening if W-piles are used. The contractor should be prepared to cut or splice piles, as necessary. Splicing of piles should be in accordance with specifications provided by the project Structural Engineer.

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

EARTHWORK

The following presents recommendations for site preparation, excavation, and placement of engineered fills on the project. The designers of the PV panel racking system foundations should ultimately stipulate earthwork specifications in areas to support the PV panels. All earthwork on the project should be observed and evaluated by the Geotechnical Engineer.

Site Preparation

Strip and remove existing vegetation, organics, and other deleterious materials from proposed construction area. In the proposed solar array field, stripping of vegetation and rooted material may not be necessary if final grades are the same as the existing grades. Keeping existing topsoil and vegetation at the array field could minimize storm water erosion during construction and maintain overall ground surface stability for the life span of the solar energy development.

All exposed surfaces should be free of mounds and depressions that could prevent uniform compaction. Stripped materials consisting of vegetation and organic materials should be wasted from the site or used to revegetate landscaped areas or exposed slopes after completion of grading operations.

Where possible, the site should be initially graded to create a relatively level surface to receive fill and to provide for a relatively uniform thickness of fill beneath the proposed structures. All exposed areas that will receive fill, once properly cleared, should be scarified to a minimum depth of 8 inches, moisture conditioned to near optimum moisture content, and compacted. It is imperative the prepared materials be protected from moisture loss.

Given past site usage as agricultural farmland, the potential exists to encounter disturbed soil across the site in the form of soft, disturbed plow zone soils. During our preliminary geotechnical exploration this zone appeared to extend to depths of about 1 feet. As such, the development budget should include a contingency to remediate areas of potential soft soils, should the need arise. This may entail either over-excavation and rework and recompaction to replacement with engineered fill dependent on risk tolerances.

Please note, that any soil placed over topsoil will settle with time with the magnitude of the settlement being directly related to the thickness of these types of soils. Therefore, any materials consisting of topsoil, vegetation and organic matter should be stripped and wasted off site or could be re-spread in landscaped areas after completion of grading operations. Stripping depths between our boring locations and across the site could vary considerably. We recommend actual stripping depths be evaluated by a representative of Terracon during construction to aid in preventing removal of excess material.

Removal and/or relocation of any "to be abandoned" utilities should also be performed prior to rough site grading activities. We would anticipate removal and relocation, or re-routing, of any existing utilities that may currently exist within the footprint of the proposed development area would interfere with new construction. Where abandoned underground pipes are located beneath any mat or shallow foundations, they should be fully grouted if left in place. Excavations created due to utility relocations should be backfilled with engineered fill material, placed and compacted in accordance with the recommendations provided in the following paragraphs, or with lean concrete or flowable fill. If lean concrete, the contractor should refer to all of the new build Mechanical-Electrical-Plumbing (MEP) and foundation drawings to confirm that concrete backfill materials will not conflict with any new item installations or construction.

Wet or dry material should either be removed, or moisture conditioned and recompacted to the project specified densities and moisture contents. Moderate to highly plastic clays, encountered at the subgrade for lightly-loaded grade-supported elements should be undercut and replaced with at least 2-feet of low volume change (LVC) material meeting the requirements of the **Fill Material Types** section of this report. We recommend the actual stripping depth and undercutting of unsuitable soils be observed and documented by a representative of Terracon during construction.

The subgrade should be proof-rolled 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 undercutting and replacement with engineered fill or other stabilization recommended by the Geotechnical Engineer at the time of construction.

Fill Material Types

Fill required to achieve design grades should meet the following material property requirements:

Soil Type [1]	USCS Classification	Acceptable Location for Placement
Low-plasticity Cohesive	CL, CL-ML	All locations and elevations
High-plasticity Cohesive	CH	Should not be used within 2 feet of shallow foundations, or floor slabs.
Low Volume Change Material (LVC)	CL or GM-GW, GM [2,3] (LL<40% & PI<15)	All locations and elevations
Imported Granular Material [2]	GW, GM, GC, SW, SM, SC	All locations and elevations

1. Engineered fill should consist of approved materials free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to use.
2. Maximum particle size of 3 inches and less than 10% Passing #200 sieve.

Fill Compaction Requirements

Engineered fill should meet the following compaction requirements.

Item	Description
Maximum fill lift thickness	■ 8 inches or less in loose thickness when heavy, tamping foot or vibratory drum compaction equipment is used ■ 4 inches or less in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used
Minimum compaction requirements	95% of the material's standard Proctor maximum dry density (ASTM D 698)
Moisture content	Low plasticity cohesive soils: Within the range of -1% to + 3% of the optimum moisture content High plasticity cohesive soils: Within the range of 0 to + 4% of the optimum moisture content Well graded granular material containing little or no silt: Workable moisture contents, sufficient to achieve compaction without the material pumping when proof-rolled

Utility Trench Backfill

Trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular materials, they should be capped with at least 6 inches of cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill.

Utility trenches are a common source of water infiltration and migration. All utility trenches that penetrate beneath the foundation should be effectively sealed to restrict water intrusion and flow through the trenches that could migrate below the foundation with a clay plug. The plug material should consist of clay compacted at a water content at or above the soil's optimum water content. The clay fill should be placed to completely surround the utility line and be compacted in accordance with recommendations in this report.

Grading and Drainage

Final surrounding grades for should be sloped away from structures on all sides to prevent ponding of water. All grades must provide effective drainage away from the structures during and after construction. Water permitted to pond next to the structures can result in greater soil movements than those discussed in this report. Estimated movements described in this report are based on effective drainage for the life of the structure and cannot be relied upon if effective drainage is not maintained.

Exposed ground should be sloped at a minimum of 5 percent grade for at least 5 feet beyond the perimeter of the structures. Backfill against the structures, if necessary, should be well compacted and free of all construction debris to reduce the possibility of moisture infiltration. After construction and prior to project completion, we recommend that verification of final grading be performed to document that positive drainage, as described above, has been achieved.

Earthwork Construction Considerations

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

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

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

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

SHALLOW FOUNDATIONS

We understand within the substation that some equipment may be supported on mat/slab foundations, while other building(s) may be supported on shallow footing foundations. The proposed structure types and loading information were not available at the time of this report. Once loading for these ancillary structures is better known, detailed settlement analyses can be performed to confirm shallow foundation recommendations.

If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow spread foundations and mat slab foundations for proposed lightly loaded structures and related structural elements.

Spread Footings

Item	Description
Maximum Net Allowable Bearing pressure [1, 2]	1,500 psf
Required Bearing Stratum [3]	Engineered fill, LVC, or approved native soil as discussed in **Earthwork**
Minimum Foundation Dimensions	Columns: 24 inches Continuous: 18 inches
Ultimate Passive Resistance [4] **(equivalent fluid pressures)**	250 pcf
Ultimate Coefficient of Sliding Friction [5]	0.4
Minimum Embedment below Finished Grade [6]	24 inches
Estimated Total Movement from Structural Loads [2, 7]	About 1 inch
Estimated Differential Movement [2, 8]	About ½ of total movement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. An appropriate factor of safety has been applied. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.
2. Values provided are for maximum loads noted in **Project Description**.
3. Unsuitable or soft soils should be over-excavated and replaced per the recommendations presented below. A 2-foot LVC material layer is recommended to remove the fat clays from beneath the bottom of footing as discussed in **Earthwork**.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Should be neglected for foundations subject to net uplift conditions.
6. Embedment necessary to minimize the effects of frost and/or seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7. The foundation settlement will depend upon the variations within the subsurface soil profile, the structural loading conditions, the embedment depth of the footings, the thickness of compacted fill, and the quality of the earthwork operations. Footings should be proportioned to relatively constant dead-load pressure in

Item	Description
	order to reduce differential movement between adjacent footings. Assumes column loads are less than 150 kips
8.	Differential movements are as measured over a span of 50 feet.

Mat Foundations

Mat foundations could be considered for supporting heavy equipment loads or structures that cannot tolerate movements. Subgrades for mat foundations should be prepared following the recommendations presented in **Earthwork**. We recommend that mat foundations should be supported a minimum 12-inch thick free draining granular base, such as relatively clean, crushed limestone.

The mat foundation can be designed using a theoretical allowable soil bearing capacity of 1,500 psf. A modulus of subgrade reaction design value should be based on anticipated soil contact pressure and theoretical vertical displacements. In this case, based on an assumed 1,500 psf contact stress and anticipated settlement up to 1 inch, a value of 10 pci is computed as the average subgrade modulus for mat foundation widths in the range of about 5 to 20 feet.

If lateral load resistance is required, an ultimate coefficient of friction between the bottom of the concrete mat and the underlying structural granular fill can be assumed to be 0.35. Typically, we recommend that a safety factor of about 1.5 be used against sliding. It is recommended that passive pressure resistance along the sides of the foundation be neglected.

Equipment Slabs

Relatively lightly-loaded equipment pads that can tolerate some movements can be supported on slabs-on-grade. Slab-on-grade construction should be suitable for the proposed equipment slab provided the subgrade is prepared in accordance with the recommendations provided in **Earthwork**.

For design under "point loading" conditions, a subgrade modulus of 100 pci is recommended. For a turned-down concrete slab, we recommend that “footings” at the edges of the slab bear upon or within at least stiff native soils or new structural fill at a depth of at least 2 feet below exterior grade.

The slab should be supported directly on new base course material consisting of free draining granular material. We recommend a minimum 6-inch thick free draining granular base, such as relatively clean, well-graded crushed limestone with less than 5% fines. This material will serve as a leveling course, a capillary moisture break, help provide load distribution, and expedite construction. Care will be necessary to avoid contaminating this layer with soil prior to slab placement.

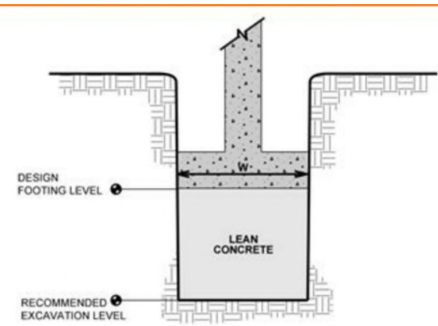
During earthwork procedures, care should be taken to maintain the subgrade moisture content prior to construction of the slab. If the subgrade should become desiccated, the affected material should be removed, or these materials should be scarified, moistened, and re-compacted prior to floor slab placement.

Where appropriate, 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. The use of a vapor retarder should be considered beneath concrete slabs-on-grade that will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Shallow Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the direction of the Geotechnical Engineer. We recommend that fat clay if encountered be undercut a minimum of 2-foot below design foundation bearing elevation and replaced with LVC material such as lean clay engineered fill or well-graded aggregate material, or lean concrete extending to at least stiff clay. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

If unsuitable bearing soils are encountered at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. This is illustrated on the sketch below.

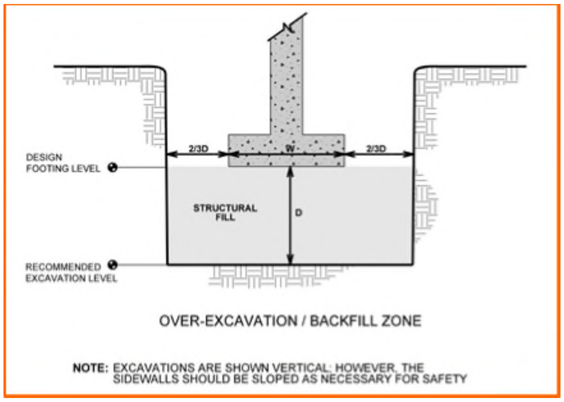


LEAN CONCRETE BACKFILL

NOTE: EXCAVATIONS ARE SHOWN VERTICAL; HOWEVER, THE SIDEWALLS SHOULD BE SLOPED AS NECESSARY FOR SAFETY

Over-excavation for engineered fill placement below footings should be conducted as shown below. Over-excavation for compacted engineered fill placement below footings should extend laterally beyond all edges of the footings at least 8 inches per foot of over-excavation depth below footing base elevation. The over-excavation should then be backfilled up to the footing base elevation with granular engineered fill material placed in lifts of 8 inches or less in loose thickness

(4 inches or less if using hand-guided compaction equipment) and compacted according to the recommendations provided in **Earthwork**.



PRELIMINARY DRILLED SHAFT FOUNDATION PARAMETERS

Substation structures will likely be supported on drilled shaft foundations. As an alternative to driven piles, other structures for the solar array areas (i.e. inverters and embedded poles) can be supported on drilled shaft foundation systems. The other structures within the array field can be supported on drilled shaft foundations with a minimum depth of 4B (where B is the shaft diameter).

Preliminary parameters for design of drilled shaft foundations in the planned substation area are provided below based on exploration results of Boring B-21 and B-22. If the location of the new substation and equipment pad areas change we should be consulted prior to the design and construction of foundations. The recommended allowable design parameters for the drilled shaft design include a factor of safety of 3.0 for end bearing and 2.0 for side resistance.

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. 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 30 inches is recommended for the foundations.

Drilled Shaft Foundation Design Parameters

Layer Depth (feet)	(P-y) Curve Type Model	Unit Weight, γ (pcf)	Ultimate Skin Friction [2, 4] (ksf)	Ultimate End Bearing Pressure [3,4]	Undrained Cohesion, c (psf)	Uniaxial Compressive Strength (psi)	Strain Factor ε_{50}	RQD (%)	Rock Mass (psi)
0 to 2 [1]	Stiff Clay without Free Water (Reese)	120	---	---	1,500	---	Default	---	---
2 to 6			1.5	---	1,500	---	Default	---	---
6 to 23			2.0	54.0 [2]	2,000	---	Default	---	---
23 to 23	Weak Rock (Reese) [3]	135	6.0	150.0	---	100	0.0005	10	50,000

1. The side resistance of the uppermost 2 feet of the soil should be ignored due to the potential for disturbance caused during the drilled shaft construction.
2. Drilled shafts should be founded at a depth of at least 10 feet below the ground surface.
3. For the weathered limestone stratum and anticipated limestone bedrock below refusal, we assumed preliminary parameters based on our experience with similar projects. For the final design, rock coring should be performed to confirm the parameters.
4. For normal load cases, we suggest that a factor of safety of 2.0 be used against the provided ultimate resistance values for determining allowable skin friction values, and a factor of safety of 3.0 be used for determining the allowable end bearing values above.

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 Shaft Construction Considerations

Due to likely presence of shallow bedrock, rock coring or augers fitted with rock teeth may be required to advance the drilled shaft excavations to the proposed depth. 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.

The actual bearing elevation at each drilled shaft location should be determined in the field during construction through inspection by an authorized representative of the geotechnical engineer. 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.

To facilitate pier construction, concrete should be on-site and ready for placement as pier excavations are completed. It is recommended that no completed drilled shaft holes be left open overnight without being filled with concrete.

SEISMIC CONSIDERATIONS

The seismic design requirements for structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil properties encountered at the site and as described on the exploration logs and results, it is our professional opinion that the ***Seismic Site Classification is C***. Subsurface explorations at this site were extended to a maximum depth of 23 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring depth.

FIELD ELECTRICAL RESISTIVITY TESTSING

Field measurements of soil electrical resistivity were performed using the Wenner Four Electrode Method with “a” spacings of 2½, 5, 10, 20, 50, 70, 100, and 150 feet at 10 locations within the solar array area, and at “a” spacings of ½, 1, 1½, 2, 3, 5, 7, 10, 15, 20, 30, 45, 70, 100, 150, 250, 350, and 450 feet at one location within the proposed substation. The “a” spacing is generally considered to be the depth of influence of the test. The testing was performed in both a north-south and an east-west orientation at each location. Results of the soil resistivity measurements are presented in **Exploration Results**. The resistivity ranged from as low as 4300 ohm-cm to as high as 75200 ohm-cm.

It should be noted that the electrical resistivity values measured in the field could vary by material type, moisture content, surface temperature, ground-water depth, and other climatic conditions. Cultural features such as fences, utilities, and railroads can also influence the resistivity readings.

CORROSIVITY TESTING

Samples for corrosion testing were obtained from 11 locations. The samples were obtained from depths of approximately 2 to 3 feet below existing ground surface. The samples were tested for Water-soluble sulfate ion content (ASTM C1580), water-soluble chloride ion content (ASTM D512), pH (ASTM D4972), Sulfides (ASTM D4658), Oxidation Reduction Potential (ASTM G200), Loss on Ignition (LOI) of Solid Combustion Residues (ASTM D7348), and electrical resistivity using the “soil box” method (ASTM G187). The results of the Corrosion Series Testing are presented on **Exploration Results**.

Boring No.	Sample Depth (feet)	Water-Soluble Sulfate (mg/kg)	Sulfides (mg/kg)	Chlorides (mg/kg)	pH	Electrical Resistivity (ohm/cm)	RedOx, (mV)	Total Salts (mg/kg)
B-1	2 to 3	116	nil	18	6.1	6,092	+373	278
B-6	2 to 3	115	nil	13	5.8	8,776	+387	211
B-10	2 to 3	80	nil	18	5.7	9,912	+420	156
B-12	2 to 3	123	nil	16	4.3	9,912	+425	157
B-13	2 to 3	87	nil	10	4.1	9,809	+428	177
B-14	2 to 3	97	nil	11	5.1	5,524	+428	261
B-16	2 to 3	150	nil	8	5.3	5,730	+434	231
B-17	2 to 3	75	nil	8	5.3	8,673	+415	206
B-20	2 to 3	54	nil	11	4.8	12,390	+437	125
B-21	2 to 3	47	nil	14	4.7	10,377	+440	140
B-22	2 to 3	66	nil	23	4.7	10,325	+448	162

The degradation of concrete or cement grout can be caused by chemical agents in the soil that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. These test results are provided to assist in determining the type and degree of corrosion protection that may be required. We recommend that a certified corrosion engineer be employed to determine the need for corrosion protection and to design appropriate protective measures, if required.

THERMAL RESISTIVITY TESTING

Laboratory thermal resistivity testing was performed on 7 soil samples obtained during our field exploration from depths of approximately 1 to 4 feet below the existing ground surface. The thermal resistivity testing was performed in general accordance with the IEEE 442 standards. Interpretation of the thermal resistivity test results should be performed by the design team in determination of underground cable sizes and/or rating.

The dry-out curves were developed from soil specimens compacted to 90% of standard proctor maximum dry density (ASTM D698). This degree of compaction should be specified for backfill materials in order to utilize the test values. The results of the thermal resistivity testing are summarized below and the relationship of values with decreasing moisture content (dry out curves) for the remolded and undisturbed soils are presented in the **Exploration Results**.

The measured thermal resistivity values ranged from about 141 to 269 °C-cm/watt at “dry” conditions for the samples tested. The test values were about 46 to 59 °C-cm/watt at the “wet” conditions for the samples tested.

Sample Location and Soil Type	Sample Type	Depth (feet bgs)	Sample Optimal Moisture Content (%)	Approx. Dry Unit Weight (pcf)	Approx. % of Standard Proctor Dry Unit Weight	Thermal Resistivity (°C-cm/watt)	
						Wet [1]	Dry [2]
B-1 (CH)	Bulk	1 to 4	17.6	101.5	90	59	177
B-6 (CL)	Bulk	1 to 4	14.0	110.4		46	158
B-12 (CL)	Bulk	1 to 4	18.4	103.1		59	182
B-14 (CL)	Bulk	1 to 4	16.6	106.1		55	269
B-17 (CH)	Bulk	1 to 4	15.6	107.0		50	141
B-21 (CH)	Bulk	1 to 4	17.7	103.1		57	164
B-22 (CL)	Bulk	1 to 4	17.1	106.5		49	156

1. "Wet" thermal resistivity reported at initial moisture content.
2. "Dry" thermal resistivity reported at 0% moisture content.

ACCESS ROADWAYS

We understand that the proposed gravel access road will be primarily used by light duty maintenance vehicles. We would expect gravel access roads with a minimum 9-inch thick crushed stone base course over the final prepared subgrade to be sufficient for support of post-construction traffic. Base course materials should conform to the Kentucky Standard Specifications. We recommend an estimated CBR value of 2 be used in preliminary design, which would be representative of lean clay materials encountered near the surface in some of the borings. CBR tests should be performed as part of the final design level study.

The performance of all roadways can be enhanced by minimizing excess moisture which can reach the subgrade soils. We recommend constructing the subgrade and base course surface with a minimum ¼ inch per foot (2%) slope to promote proper drainage and site grading at a minimum 2% grade away from the road. For unpaved roads, maintaining the proper slope over the life of the roadway with periodic re-grading and resurfacing, as needed, will enhance long term performance. Placement of a geotextile such as Mirafi® HP370, Tensar TX160 geogrid, or similar beneath the roadway base course may reduce the overall thickness and the amount of maintenance required to maintain the unpaved road.

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.

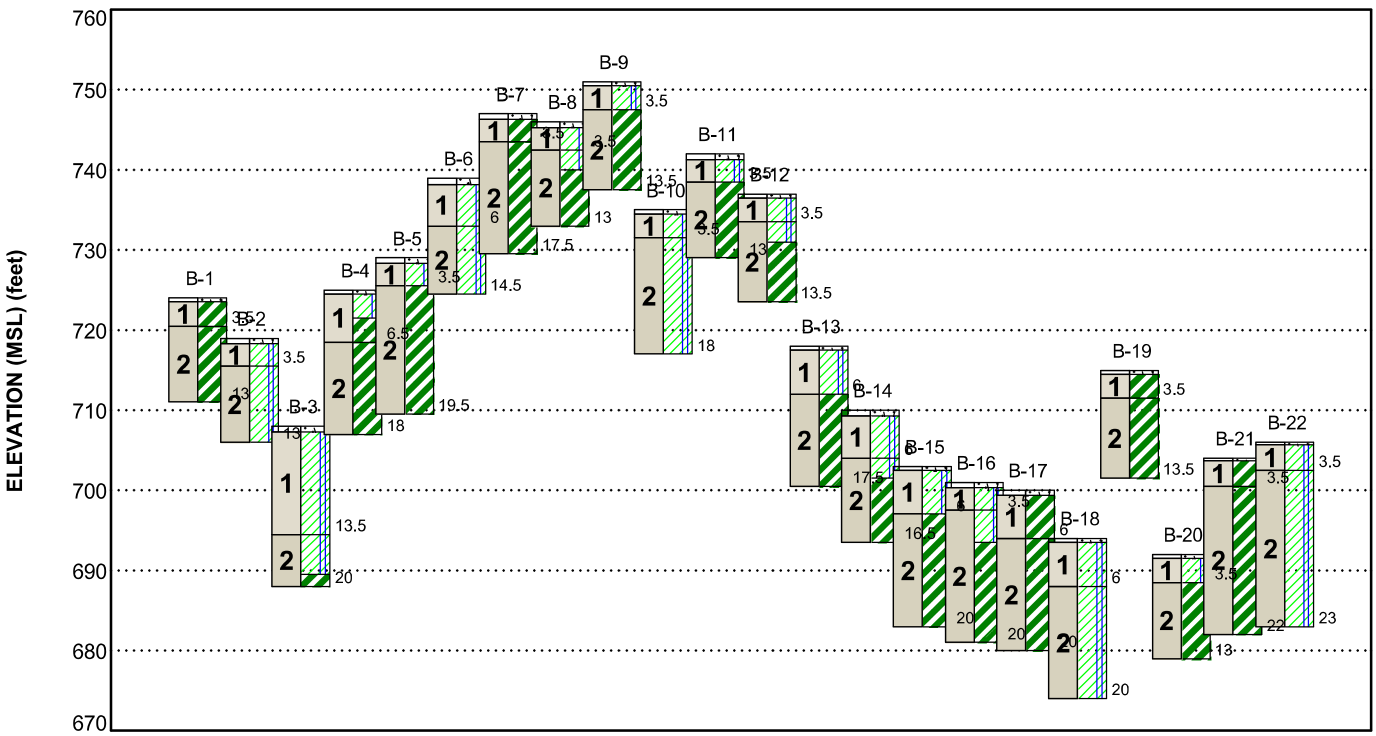
FIGURES

Contents:

GEOMODEL

Hoffman Solar Project ■ Franklin, KY
Terracon Project No. 57205066

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	**Cohesive Soil**	Lean Clay (CL) to Fat Clay (CH), brown to reddish brown, medium stiff to stiff
2	**Cohesive Soil**	Lean Clay (CL) to Fat Clay (CH), with rock fragments, brown to reddish brown, very stiff to hard

LEGEND

Topsoil

Fat Clay

Lean Clay with Silt

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 Description

Number of Explorations	Type of Exploration	Depth or Description	Explored Location
36	Soil Borings	4 to 23 ft	Array and Substation Area
10	Field Electrical Resistivity	2½, 5, 10, 20, 50, 70, 100, and 150 feet	Array Area
1		½, 1, 1½, 2, 3, 5, 7, 10, 15, 20, 30, 45, 70, 100, 150, 250, 350, and 450 feet	Substation Area

Boring Layout and Elevations: Terracon personnel provided the boringlayout. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±20 feet) and approximate elevations were obtained by interpolation from Google Earth Pro™. If more precise elevations and boring layout are desired, we recommend borings be surveyed following completion of fieldwork.

Subsurface Exploration Procedures: We advanced soil borings with a track-mounted drill rig using continuous hollow stem auger. Four to five samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. Soil sampling was performed using a standard 2-inch outer diameter split barrel sampling spoon that was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the middle 12 inches of a 24-inch sampling interval or the last 12 inches of an 18-inch sampling interval was recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. The samples were placed in appropriate containers, taken to our soil laboratory for testing, and classified by a geotechnical engineer. In addition, we observed and recorded groundwater levels during sampling.

Our exploration team prepared field boring logs as part of standard drilling operations including sampling depths, penetration distances, and other relevant sampling information. Field logs included 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.

LABORATORY TESTING

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

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D2166/D2166M Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
- ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort
- ASTM D1883 Standard Test Methods for California Bearing Ratio (CBR) of Laboratory-Compacted Soils

Our laboratory testing program also included examination of soil samples by an engineer. Based on observation and test data, the engineer classified the soil samples in accordance with the Unified Soil Classification System.

SITE LOCATION AND EXPLORATION PLAN

Contents:

SITE LOCATION PLAN

Hoffman Solar Project ■ Franklin, Simpson County, Kentucky

March 2, 2021 ■ Terracon Project No. 57205066

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

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN

Hoffman Solar Project ■ Franklin, Simpson County, Kentucky

March 2, 2021 ■ Terracon Project No. 57205066

Terracon GeoReport

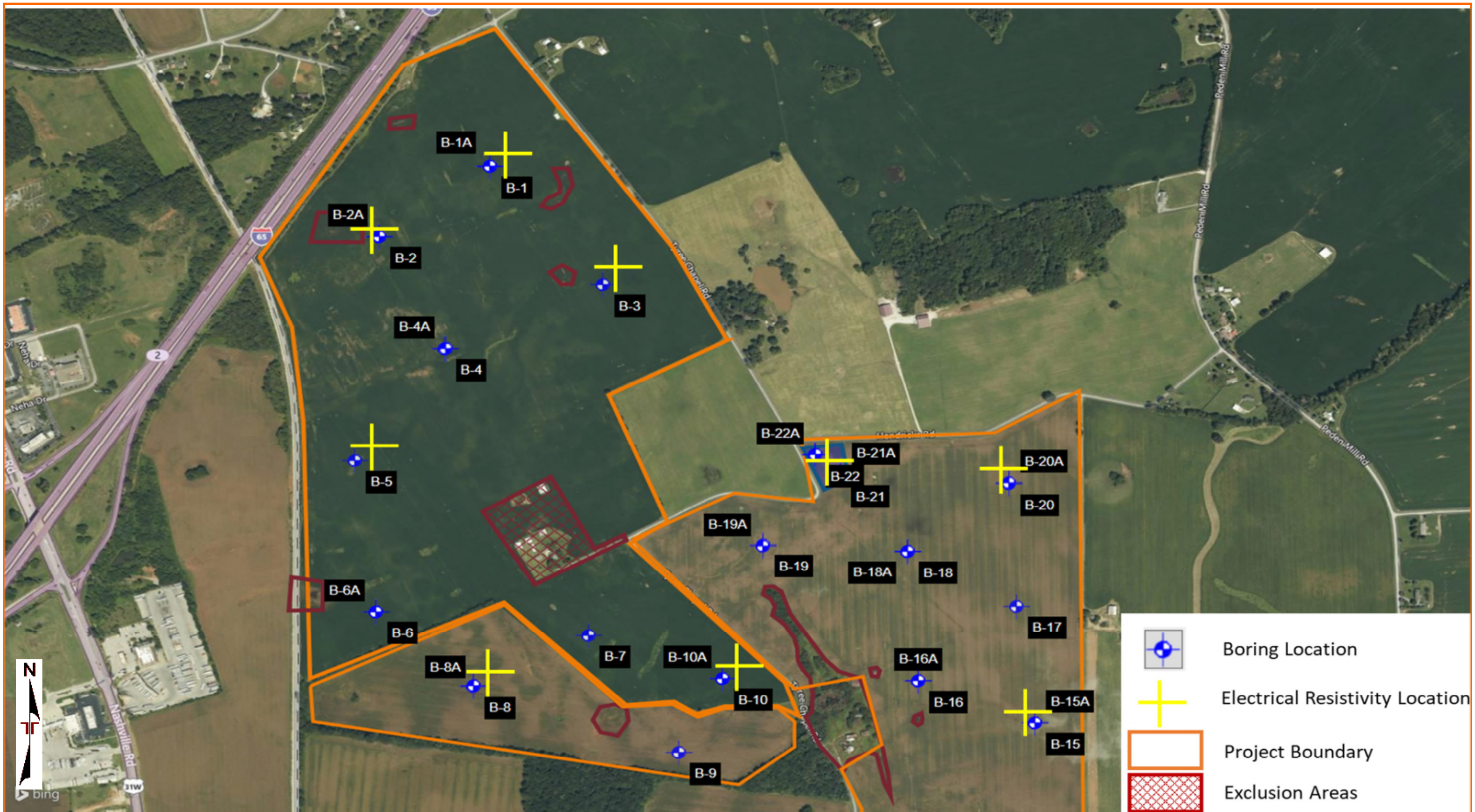


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN

Hoffman Solar Project ■ Franklin, Simpson County, Kentucky

March 2, 2021 ■ Terracon Project No. 57205066

Terracon GeoReport

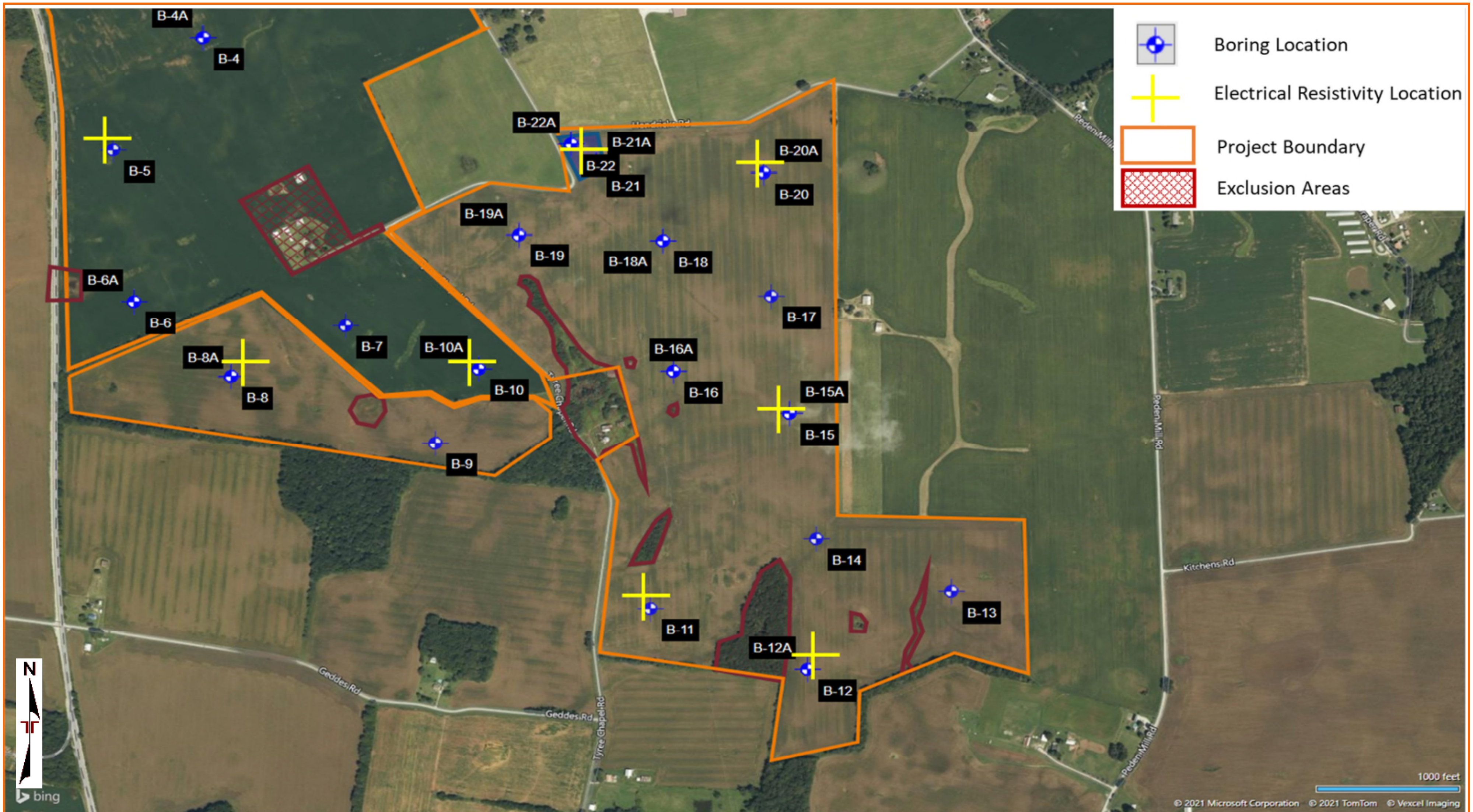


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

BORING LOG NO. B-1

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6731° Longitude: -86.5454°

Approximate Surface Elev.: 724 (Ft.) +/-

MODEL LAYER	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	0.5	**TOPSOIL**	723.5+/-												
1	3.5	**FAT CLAY (CH)**, with rock fragments, reddish brown, medium stiff	720.5+/-			X	14	3-3-4 N=7	1.75 (HP)				25.8		
2		**FAT CLAY (CH)**, with rock fragments, reddish brown, very stiff to hard				X	16	13-14-13 N=27	3.50 (HP)				28.1		
				5		X	13	15-15-15 N=30	4.50+ (HP)				21.9		
						X	15	20-23-21 N=44	3.50 (HP)				28.0		
	13.0		711+/-	10											
		Auger Refusal at 13 Feet													

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-1A

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp
Jupiter, FL

SITE: Tyree Chapel Road
Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6731° Longitude: -86.5454°

Approximate Surface Elev.: 724 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH / ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
1		**FAT CLAY (CH)**, reddish brown												
		3.5 — 720.5+/-												
2		**FAT CLAY (CH)**, reddish brown	5											
			10											
		12.5 — 711.5+/-												
		Auger Refusal at 12.5 Feet												

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-2

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6716° Longitude: -86.5480°

Approximate Surface Elev.: 719 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
		0.7	**TOPSOIL**	718.5+/-												
1		3.5	**LEAN CLAY (CL)**, brown, stiff	715.5+/-			X	15	4-4-5 N=9	4.50+ (HP)				21.9		
2			**LEAN CLAY (CL)**, brown, very stiff to hard				X	16	12-12-14 N=26	4.50+ (HP)				13.4		
					5		X	11	13-15-15 N=30	2.50 (HP)				22.8		
							X	15	20-21-23 N=44	3.75 (HP)				26.7		
		13.0		706+/-	10											
			Auger Refusal at 13 Feet													

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-2A

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6716° Longitude: -86.5480°

Approximate Surface Elev.: 719 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH / DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
1		**LEAN CLAY (CL)**, brown													
		2.0	717+/-												
		LEAN CLAY (CL), brown, stiff					24		4.50+ (HP)	UC	1.20	3.5	23.5	99	40-19-21
		4.0	715+/-												
		Boring Terminated at 4 Feet													

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-3

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6706° Longitude: -86.5427°

Approximate Surface Elev.: 708 (Ft.) +/-

MODEL LAYER	DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
	0.7	**TOPSOIL**	707.5+/-												
1		**LEAN CLAY (CL)**, brown, medium stiff to stiff				X	14	3-3-3 N=6	1.25 (HP)				24.5		
				5		X	16	5-5-5 N=10	1.25 (HP)				26.7		
						X	16	8-8-7 N=15	1.25 (HP)				27.9		
				10		X	17	7-7-7 N=14	0.50 (HP)				25.9		
	13.5		694.5+/-												
2		**LEAN CLAY (CL)**, brown, very stiff		15		X	16	12-11-14 N=25	1.50 (HP)				22.5		
	18.5		689.5+/-												
		FAT CLAY (CH), with rock fragments, reddish brown, very stiff				X	15	12-14-15 N=29	1.75 (HP)				22.0		
	20.0		688+/-	20											
		Boring Terminated at 20 Feet													

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-4

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6692° Longitude: -86.5465°

Approximate Surface Elev.: 725 (Ft.) +/-

MODEL LAYER	DEPTH	DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
1	0.5	**TOPSOIL**	724.5+/-												
1	3.5	**LEAN CLAY (CL)**, brown, medium stiff	721.5+/-			X	9	4-4-4 N=8	1.00 (HP)				23.1		
1	6.5	**FAT CLAY (CH)**, with rock fragments, reddish brown, stiff	718.5+/-	5		X	14	7-7-8 N=15	1.75 (HP)				21.6		
2	18.0	**FAT CLAY (CH)**, with rock fragments, reddish brown, very stiff to hard	707+/-			X	15	16-18-21 N=39	1.25 (HP)				25.0		
				10		X	16	22-25-25 N=50	2.75 (HP)				21.8		
				15		X	16	21-22-22 N=44	4.00 (HP)				23.1		

Auger Refusal at 18 Feet

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	

BORING LOG NO. B-4A

PROJECT: Hoffman Solar Project

CLIENT: OPDE Energy C/O, Horus Renewables Corp Jupiter, FL

SITE: Tyree Chapel Road Franklin, KY

LOCATION See Exploration Plan

Latitude: 36.6692° Longitude: -86.5465°

Approximate Surface Elev.: 725 (Ft.) +/-

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	LABORATORY HP (tsf)	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI
1		**LEAN CLAY (CL)**, brown													
		3.0	722+/-												
		LEAN CLAY (CL), brown, soft					24		2.00 (HP)	UC	0.24	2.1	23.1	88	49-20-29
		5.0	720+/-	5											
		Boring Terminated at 5 Feet													

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

Hammer Type: Automatic

Advancement Method:
2 1/4" Hollow Stem Auger

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

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

See Supporting Information for explanation of symbols and abbreviations.

Elevations were interpolated from Google Earth pro.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
13050 Eastgate Park Way Ste 101
Louisville, KY

Boring Started: 12-23-2020	Boring Completed: 12-23-2020
Drill Rig: 7822 DT	Driller: M. Reynolds
Project No.: 57205066	