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October 17, 2016

Via Electronic Filing

Ms. Barcy McNeal
Public Utilities Commission of Ohio
Administration/Docketing
180 East Broad Street, 11th Floor
Columbus, OH 43215-3793

Re: Hardin Wind Energy LLC, Case No. 09-479-EL-BGN

Dear Ms. McNeal:

The March 22, 2010 Opinion, Order, and Certificate ("Certificate") approving Hardin Wind Energy LLC's ("Hardin Wind Energy") Certificate of Environmental Compatibility and Public Need established a set of conditions as part of the Certificate. On April 29, 2011 in Case No. 11-3446-EL-BGA, the Ohio Power Siting Board ("OPSB") approved an amendment ("Amended Certificate") to Hardin Wind Energy's Certificate, which also established an additional set of conditions.

Within this set of conditions, **Certificate Condition No. 22** requires that:

Hardin shall complete a full geotechnical investigation to confirm that there are no issues to preclude development of the wind farm. The geotechnical investigation shall include borings at each turbine location to provide subsurface soil properties and recommendations needed for the final design and construction of each wind turbine foundation, as well EIS the final location of the transformer substation and interconnection substation. All boreholes must be filled, and borehole abandonment must comply with state and local regulations. The applicant shall provide copies of all geotechnical boring logs to staff and to ODNR Division of Geological Survey.

In compliance with Certificate Condition No. 22, attached is a copy of the Geotechnical Engineering Report dated January 2012, and the October 5, 2016 Supplement Geotechnical Investigation and Analysis. Thus Hardin Energy is in compliance with this condition.

If you have any questions please call at the number listed above.

Sincerely,

Sally W Bloomfield

Sally W. Bloomfield

Attachment

cc: Andrew Conway (w/Attachment)
Derek Collins (w/Attachment)

Geotechnical Engineering Report
Hardin Wind Project
Hardin County, Ohio

Prepared for
Invenergy, LLC
Chicago, Illinois

January 2012

BARR

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Hardin Wind Project
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Geotechnical Engineering Report
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January 2012

Table of Contents

Tables

Figures

Appendices

1.0 Introduction

Barr Engineering Company (BARR), under authorization and contract with Invenergy, LLC (Invenergy), completed a design phase geotechnical investigation of the Hardin Wind Project site in Hardin County, Ohio.

Under subcontract to BARR, Minnesota GeoServices of St. Paul, Minnesota completed cone penetration testing (CPT) for 114 of 133 proposed turbine locations; 19 turbine locations were on hold at the time of the field investigation. Marchetti Dilatometer Testing (DMT) was performed at 13 proposed turbine locations in conjunction with CPT testing to evaluate settlement potential.

Under subcontract to BARR, GEOCON Professional Services of Frankfort, Illinois completed hollow-stem auger (HSA) drilling at fifteen proposed turbine locations and the switchyard, substation, and O&M building locations. Piezometers were installed at each of the proposed turbine locations investigated by HSA drilling.

A Barr representative was present during the explorations. Selected soil samples collected during the borings were tested by GEOCON and Soil Engineering Testing (SET) of Bloomington, Minnesota.

This report describes the geotechnical investigation and testing performed, presents the results of this work, and provides geotechnical analysis and conclusions for foundations to be designed and constructed for the proposed wind project.

1.1 Site Location

The proposed Hardin County Wind Project is located on rural farmland in Hardin County, Ohio, outside of the town of McGuffey (Figure 1). Figure 2 shows the proposed turbine locations and the numbering provided at the start of the investigation. The coordinates for each proposed turbine location and related geotechnical testing locations are included in Table 1.

The proposed turbines sites were located in agricultural fields. The majority of the site lies in the Scioto Marsh area which is a very flat area of former marsh land that formed in the glacial lake basin and resulted in approximately 2 to 10 feet of peat overlying the underlying lacustrine clay (Reference 19). Natural and man-made drainage waterways are located in low-lying areas of the site. It was reported that County Road 195 south of McGuffey has had numerous serviceability issue with respect to settlement in the area of the marsh. During the time of the geotechnical field investigation a portion of the road was closed. Increased topographic variation is present around the edges of the Scioto Marsh area.

1.2 Site Geology

The project site is located within a physiographic region known as the Eastern Lake and Till Plains Sections of the Central Lowland Province of the Interior Plains. This entire region is glaciated, and most areas are dominated by ground moraines that are broken in places by lake plains, outwash plains, flood plains, and many recessional moraines (Reference 8).

More localized to the project area, surficial deposits are mostly ground and end moraines (glacial till), lake plain (lacustrine soils), and a covering of modern peat (organic soil) over deeper glacial deposits and bedrock. Ground moraine and end moraine deposits consist primarily of a clay matrix with interlayered sand, silt, and some gravel. Ground moraine areas are characterized by relatively flat to gently undulating topography, while end moraine areas occur as hummocky ridges. To the north of project site, the topography tends to have less relief. These areas are represented by lacustrine deposits consisting of primarily clay and silt (Reference 14). The soil origin (parent material) is described in Figure 3. Surficial USCS soil classifications are provided in Figure 4.

Water wells in the project area indicate a static water level of 10-feet to about 40-feet below ground surface. These wells produce from both glacial aquifers and bedrock aquifers. The glacial overburden probably represents a confining layer to the underlying bedrock aquifers and contains several confined aquifers within. The uppermost water table is likely to be shallow (5- to 20-feet below ground surface) across the entire project site (Reference 10).

Overburden thickness varies around the project area, from less than 50 feet (to the east) to over 200 feet (to the west) (Figure 5). To the west of the proposed switchyard somewhat in line with the Scioto River, there is a buried bedrock valley. Bedrock found beneath the overburden consists of Silurian-age dolomites and shale. The Tymochtee and Greenfield Dolomites overlay the Lockport Dolomite, which is found in the buried bedrock valley (Reference 9 and Reference 16) See Figure 6 for the bedrock geology map.

The State of Ohio maintains a database of oil and gas drilling and exploration activity (Reference 11) There are a few abandoned and plugged wells in the vicinity of the project, but no current oil and gas drilling activity. Similarly, state mapping of underground mines indicates no known underground mines in Hardin County (Reference 13).

Dolomite and limestone are the principle rock types that contain karst features. Karst features such as dissolution cavities, caves, and sinkholes can cause subsidence of the ground surface. Karst features have not been mapped in Hardin County. However the area is underlain by carbonate rocks and karst areas have been mapped in nearby northern Logan County and are known to occur throughout western Ohio. Karst in Ohio is particularly prevalent in areas with less than 20 ft of glacial drift or alluvium overlying the carbonate bedrock (Reference 12). Regional mapping indicates there is generally greater than 50 ft of overburden at the proposed project site so karst formation is unlikely at this time. However, there is still a risk of karst formation on site, particularly in areas with locally decreased overburden thicknesses (Reference 12).

1.3 Previous Investigation

Barr has completed two previous reports for the proposed Hardin Wind Project site. These include the switchyard geotechnical investigation report and the electrical resistivity report. Barr is not aware of any other investigations for the Hardin Wind Project site.

2.0 Geotechnical Exploration Methods

The geotechnical investigation for the Hardin Wind Project consisted of Seismic/Piezocone Penetration Testing (CPT), Marchetti Dilatometer Soundings (DMT), hollow stem auger (HSA) soil borings, standard penetration tests (SPT), piezometers, field tests, and laboratory tests. Figures 8 through 10 show the plan location of all soundings and borings completed for the project, as well as the layout of the turbines. The site investigation for the turbines was conducted in August, 2011. Laboratory testing for the turbines is ongoing. Coordinates of the proposed turbine locations and a list of testing performed at each turbine location are provided in Table 1. The turbine locations were located using hand-held GPS units during field activities at the direction of Invenergy.

2.1 Field Work

2.1.1 CPT Soundings

CPT soundings were performed at 114 potential turbine sites being considered for development at the time of this report (see Figure 8). CPT testing was performed in accordance with ASTM D5778, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils.” The CPT soundings generally reached a depth of greater than 30 feet below ground surface, with the exception of the following proposed turbine locations 12, 14, 18, 19, 26, 30, 31, 34, 35, 37-39, 117, 118, and 129. Logs of CPT soundings are in Appendix A.

All CPT soundings were conducted by Minnesota GeoServices of St. Paul, Minnesota. CPT soundings were performed with either a 20-ton truck mounted rig or 15-ton track mounted rig with an enclosed work space. All equipment was in accordance with ASTM D-5778. For the CPT test, a cylindrical cone is pushed vertically into the ground at a constant rate of penetration of 20 mm/sec. During penetration, measurements are made of the cone tip resistance (q_c), the side friction of the cylindrical shaft (f_s) just above the tip, and porewater pressure generated by cone penetration (u_2). The cones used in the investigation have a 15 cm^2 base area with a 60 degree apex angle. The sleeve area of the cones is 300 cm^2. The fluid used for saturation of the filter was glycerin. Minnesota GeoServices provided BARR with complete records of tip resistance, sleeve friction, pore pressure, and friction ratio of all CPT soundings. These records included a hard copy showing the graphical variations of all readings with depth.

The following describes the procedures and processes used to interpret the CPT data and the interpreted lithology.

CPT data reduction and interpretation were performed using an in-house program designed by Barr specifically for use on wind turbine projects. The in-house program has been cross-checked with CPTINT version 5.2 for quality assurance and has been found to be compliant. The program uses the soil behavior type classification system from CPT data proposed by Robertson et al. (1986). The classification system is based on the corrected tip resistance (q_t), the friction ratio (R_f), and pore-water pressure parameter (B_q), and includes a total of 12 soil behavior types.

These cone parameters are defined as follows:

$$q_t = q_c + (1 - a) \cdot u_2$$ (Reference 6, page 25)

$$R_f = \frac{f_s}{q_t} \cdot 100\%$$ (Reference 6, page xiv)

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{vo}}$$ (Reference 6, page 51)

where

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

a = the area ratio of the cone

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

f_s = unit sleeve friction resistance, load per area

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

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

The cone was also equipped with a seismometer that measured the arrival time of shear and compression waves generated at the ground surface. The shear waves were generated at the ground surface, by the CPT rig, in the selected locations, and arrival times were measured at depth intervals of approximately 3 m (~10 ft.), to determine the interval shear wave velocity. In a similar manner, the compression waves were generated at the ground surface, by the CPT rig, in selected locations, and arrival times were measured at depth intervals of approximately 3 m (~10 ft.), to determine the interval compression wave velocity. Locations were selected to provide spatial coverage over the entire project site. The results of shear and compression wave testing can be found in Appendix B.

2.1.2 DMT Soundings

A total of 13 DMT soundings were performed at proposed turbine site locations 5, 45, 48, 51, 54, 64, 70, 78, 81, 89, 92, 107, 131 as illustrated in Figure 10. These locations were chosen for DMT testing to represent the settlement characteristics across the site and to evaluate the apparent lower strength materials encountered during CPT testing. The results of the DMT soundings performed during this investigation are included in Appendix C.

The Marchetti Dilatometer consists of a 95-mm stainless steel blade with a thin, flat, and expandable steel membrane (60-mm diameter) on the side. Performing a DMT test consists of pushing the dilatometer blade into the ground vertically to a desired test depth, measuring the thrust necessary to accomplish this penetration, and then using gas pressure to expand the circular steel membrane against the soil. The test operator obtains three readings: the A-pressure required to initiate movement of the membrane against the soil; the B-pressure required to move its center 1 mm into the soil; and the C-pressure during deflation of the membrane, which is related to the in-situ pore-water pressure in sands and penetration pore-water pressure in clays. The operator then pushes the blade to the next depth and repeats the test. A dilatometer sounding log consists of the results from all the measured and correlated parameters with depth.

The DMT parameter generally includes the measured material index I_d, dilatometer modulus E_d, horizontal stress index K_d, constrained modulus of soil compressibility M, and undrained shear strength s_u. The main objective for performing the DMT soundings was the determination of the constrained modulus of soil compressibility in order to evaluate settlement potential. The DMT has the advantage of providing quasi-continuous soil compressibility information as part of the field investigation. Traditionally, the compressibility soil parameters are obtained by performing a soil boring, taking an undisturbed Shelby tube sample, and performing a consolidation test in the laboratory. The use of the DMT results in obtaining required compressibility parameters much more quickly and comprehensively. The DMT test also is an in-situ test method which does not require sampling and transportation of soils to a testing laboratory.

2.1.3 Soil Borings

Hollow stem auger (HSA) borings were performed at proposed turbine locations 2, 14, 19, 31, 35, 39, 49, 51, 62, 72, 82, 88, 99, and 117 during the subsurface exploration. The boring locations were selected to spatially cover the site and evaluate several of the sites with shallow CPT refusal. The borings at turbine locations 2, 14, 19, 31, 35, 62, and 82 reached auger refusal at depths ranging from 18 to 45 feet below ground surface on limestone bedrock. The boring locations are indicated on Figure 9. The soil borings were performed by GEOCON Professional Services of Frankfort, Illinois with a track-mounted drill rig using hollow stem auger (HSA) techniques with split spoon sampling at maximum 5-foot intervals. The soil boring logs can be found in Appendix D.

Three-inch diameter Shelby tube samples were collected for laboratory testing in accordance with ASTM D1587. The split-spoon and Shelby tube samples were sealed and labeled in the field and delivered to GEOCON Testing Services in Frankfort, Illinois for testing physical properties and to Soil Engineering Testing (SET) laboratory in Richfield, Minnesota for laboratory testing of mechanical properties.

2.1.4 Groundwater Piezometers

PVC standpipe piezometers were installed at 13 of the proposed turbine locations investigated by HSA drilling. A record of the piezometer monitoring events to date is contained in Table 2. Groundwater generally ranged from 2.5 to 10 feet below ground surface in the piezometers.

2.2 Soil Testing

The following tests were performed or coordinated by GEOCON or SET:

- Moisture content tests were performed in accordance with ASTM D2216-05, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"
- Soil unit weight tests in accordance with ASTM D7263 "Standard Test Method for Laboratory Determination of Density (Unit Weight) of Soil Specimens"
- Specific Gravity determinations in accordance with ASTM D854-05, "Standard Test Method for Specific Gravity of Soil Solids by Water Pycnometer"
- Unconfined compressive strength in accordance with ASTM D2166, "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil"
- Triaxial (UU) compressive strength in accordance with ASTM D2850-03a, "Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils"
- Grain Size and Hydrometer analysis in accordance with ASTM D422-63(2007), "Standard Test Method for Particle-Size Analysis of Soils"
- Percent Fines (silt and clay) in accordance with ASTM D1140-00, "Standard Test Method for Amount of Material in Soils Finer Than the No. 200 Sieve"
- Atterberg Limit determinations in accordance with ASTM D4318-05, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
- Soil pH tests in accordance with ASTM D4972-01(2007) "Standard Test Method for pH of Soils"
- Soluble chloride and soluble sulfate of soils

All laboratory test results are provided in Appendix E and summarized on Tables 6.

2.3 Soil Electrical Resistivity Testing

Soil resistivity testing was completed in accordance with ASTM method G57-95a "Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method" (equivalent to IEEE Std. 81). Values were determined at turbine locations 2, 14, 35, 39, 49, 62, 66, 72, 82, 88, 99, and 117. At each location, measurements were taken to yield average soil resistivity at a-spacings of 2, 5, 10, 20 and 40 feet. The soil electrical resistivity is included in Appendix F.

2.4 Thermal Resistivity Testing

Bulk soil samples were collected at the project site for thermal resistivity testing. Samples were obtained from a depth of 3 to 5 feet below the surface and placed in sealed 5 gallon buckets. The samples were shipped to Soil Engineering Testing (SET) of Minnesota. Testing in accordance with ASTM D:5334-08 “Guide to Thermal Conductivity of Soil and Soft Rock by Thermal Needle Probe Procedure” was ongoing at the time of this draft report. Laboratory tests included measurement of the soil’s moisture content, unit weight, and thermal dryout characteristics, which is a function of moisture content. The thermal resistivity report is included in Appendix G.

3.0 Results

This Section presents the data from testing and investigation in accordance with the field and laboratory investigation procedures described in Section 2, and provides further analysis of these results.

3.1 Soil Lithology

The results of the soil borings (Appendix D), laboratory testing, and CPT soundings (Appendix A) were compiled to obtain an understanding of the lithology of the study area.

The soil conditions encountered during drilling generally agreed with the soil information obtained from the information reviewed from the soil survey information. The existing conditions consist of topsoil underlain primarily by glacial till soils and bedrock at depths of approximately 15 to greater than 50 ft.

Detailed information for soil strata and groundwater conditions are contained in the following sections:

3.1.1 Surficial Materials

The topsoil consists primarily of organic silt to silty clay and is actively farmed. The thickness varies, depending upon the plowed zone and location. The boring logs indicate a range of topsoil thickness of approximately 12 to 18 inches. These soils appear to be reworked site soils with a moderate amount of organic materials. Deeper layers of highly organic soils and peat were encountered across the site, but these layers appeared to be confined to depths shallower than the proposed foundation embedment depth.

3.1.2 Lacustrine Clay

Very weak clay was encountered in many of the CPT soundings and borings. This clay was present across the site and extended to depths of greater than 50 feet in some locations. Based on the geologic history, these very weak clays were likely deposited in the glacial lake or marshes and depressions upon retreat of the glaciers and were not exposed to significant over consolidation.

Undrained shear strength of these clays from CPT generally ranged from 300 to 500 psf while SPT N-values as low as 0 to 2 bpf were observed in many locations.

Atterberg limit testing on samples of the glacial till indicated Plastic Limit values range from about 11 to 20 percent, Liquid Limit values range from about 17 to 42 percent, and Plasticity Index values range from 3.4 to 19 percent. Natural moisture contents ranged from about 10.5 to 24 percent, with a typical range of 12 to 18 percent. According to the Plasticity Chart (Reference 7, pg 7.1-18), these soils generally plot as CL (lean silty clay) according to the USCS Classification System.

Moist unit weights ranged from 120.5 to 140.5 pounds per cubic foot (pcf) with corresponding dry densities ranging from 76 to 120 pcf. Unconfined compressive strengths laboratory testing indicated unconfined compressive strengths ranging from 0.17 to 1.5 tsf. Grain Size test results are discussed in Section 3.3.3.

Laboratory consolidation test values indicated compression indexes ranging from 0.24 to 0.27, initial void ratios ranging from 1.02 to 1.16, and preconsolidation pressures (ie maximum past pressures) between 0.42 and 0.59 tsf. With these values for maximum past pressure, the samples appear underconsolidated. After extrusion from the Shelby tube samplers, the soils wanted to consolidate under their own weight prior to testing and were very difficult to keep undisturbed for testing purposes. This confirms the very soft and highly compressible nature indicated by both the CPT and DMT testing.

The gray silty clay soils tended to be softer and more compressible in the large area noted as "Marsh" on Figure 3. Slightly stiffer gray silty clay soils were noted on the margins of the "Marsh" area. Their appearance was similar except for the marginally higher strengths observed.

3.1.3 Sand Layers

Sand layers were encountered in borings at turbine locations 19, 35, 39, 62, 66, 72, and 117 at varying depths. These layers were generally silty to clayey fine to medium grained sand soils, with fines contents typically ranging from 40 to 50 percent as indicated by grain size test results.

SPT N-values ranged from 8 to54 bpf were observed in the sand soils indicating loose to dense conditions. The seams encountered appeared to be wet to saturated.

The majority of the CPT soundings indicated relatively thin layers of sand also at varying depths and of varying thickness.

3.1.4 Glacial Till

The glacial till soils were encountered below the topsoil extending to CPT and boring refusal on bedrock at depths as shallow as 15 to 20 feet. These soils consist primarily of lean clay with varying amounts of silt and trace sand and gravel though thicker deposits of clayey silt were also encountered.

SPT N-values were also highly variable and ranged from 4 to 66 blows per foot (bpf). Atterberg limit testing on samples of the glacial till indicated Plastic Limit values range from were all about 17 percent, Liquid Limit values range from about 22 to 32 percent, and Plasticity Index values range from 5 to 15 percent. According to the Plasticity Chart (Reference 7, pg 7.1-18), these soils generally plot as CL (lean silty clay) according to the USCS Classification System.

Moist unit weights ranged from about 129 to 141 pounds per cubic foot (pcf) and dry densities ranged from about 106.5 to 124.5 pcf. Natural moisture contents ranged from about 5 to 29 percent, with a typical range of 9 to 19 percent.

3.1.5 Weathered Bedrock

In borings 2, 14, 19, 31, 35, 62, and 82 intensely fractured limestone/dolostone bedrock was encountered at depths ranging from 18 to 45 feet below ground surface. The bedrock was hard, but could be drilled for a couple feet prior to auger refusal being encountered. Coring was used to extend borings 14 and 35 into the bedrock. RQD in the weathered bedrock were observed to be zero.

Based on encountering bedrock in these borings, the shallow CPT refusals at turbine locations 1-8, 16-19, 30-38, 39-41, 60-64, and 82 also likely indicate depth to weathered bedrock.

3.2 Groundwater Conditions

Due to the lack of significant granular deposits, the results of the CPT are not definitive enough with respect to pore water pressure to discern the depth of the groundwater table.

The groundwater level was measured in the PVC standpipe piezometers installed at the turbine locations during the HSA boring portion of the investigation. There were a total of 14 piezometers installed at the turbine locations. After a third piezometer monitoring event, the water levels measured from the piezometers averaged 1.5 feet below ground surface (BGS) with a minimum reading of 0.7 feet BGS (Table 2). These readings indicate that groundwater is present at or above proposed foundation bearing elevations for at least a portion of the year.

Many factors contribute to water level fluctuations, such as heavy rainfall events, dry periods, sand seams, and etc. Based on the observed water level readings, water likely will collect in and around the proposed turbine foundations and the foundation design will need to take buoyancy effects into consideration at all of the sites. Based on the readings, a groundwater depth of 0.5 ft below ground surface should be used for foundation design.

3.3 General Laboratory Testing

3.3.1 Moisture Content

As is summarized in Table 6, 86 moisture content tests were run on soil samples collected from the turbine soil borings and bulk samples. The soils tested included loess, silty clay to clayey silt glacial till soils, and weathered shale. The native soil had moisture contents ranging from about 5.2 percent (sandy soil) to 76.4 percent (organic clay), indicating that the soils are in a moist to very moist condition. In general, the moisture contents of the organic clay and lacustrine clay soils were the highest across the site. The glacial till soils exhibited moderate moisture contents and the sandy soils exhibited the lowest moisture contents across the site. The moisture content test results for the individual soil strata are discussed in Section 3.1. Moisture content test results are summarized in Table 6, included on the boring logs in Appendix D and provided in Appendix E.

3.3.2 Atterberg Limits

Atterberg limits were determined and used to identify soil behavior characteristics and classify the material encountered in the soil borings, as designated under the Unified Soil Classification

System (USCS) using the plasticity chart from NAVFAC Design Manual 7.1 (Reference 7). A total of 13 Atterberg limits tests were conducted on selected soil samples from the turbine borings. Results of the Atterberg limits tests indicate the soils on site have Liquid Limits generally ranging from 17 percent to 42 percent, Plastic Limits generally ranging from 11 to 22 percent, and Plasticity Indices varying between 5 and 22.7 percent. Atterberg limits test results for the individual soil strata are discussed in Section 3.1. Lab test results are summarized in Tables 6 and provided in Appendix E.

3.3.3 Grain Size Analysis

Grain size analyses were performed on 7 soil samples collected at various depths in the soil borings for the turbine sites and substation. Tests were performed on a silty sand soil, clayey glacial till, and lacustrine clay. The sand soils were considered silty to clayey sand soils (SM) with a fines contents of 26 percent. The glacial till soil test results indicated gravel content ranging from 6 to 27 percent, sand content ranging from 5 to 27 percent, and silt content ranging from 26 to 38 percent and clay content ranging from 20 to 32 percent. Test results on the lacustrine clay soils indicated 1 percent sand, 35 to 38 percent silt and 60 to 64 percent clay. Gradation test results are included on the boring logs provided in Appendix D and laboratory test results are included in Appendix E.

3.3.4 Dry Density and In-Situ Unit Weight

Dry density tests were performed on 28 soil samples from Shelby tube samplers. The dry density test results on the samples selected for testing ranged from 72.2 (organic clay) to 124.3 (glacial till) pounds per cubic foot. In-situ unit weight calculations were performed using the moisture content test results on samples which dry density testing was performed. The in-situ unit weight calculations on the samples with dry unit weight results ranged from 100.8 to 141.1 pounds per cubic foot. The lower density values were associated with the organic and lacustrine soils and the higher density values were associated with the glacial till soils. Density test results are summarized in Table 6, and lab test results are provided in Appendix E.

A recommended design value for in-situ density is 120 pounds per cubic foot (the approximate average of the calculated in-situ density values based on lab testing).

3.3.5 Unconfined Compressive strength and UU Testing

Unconfined compressive strength and unconsolidated undrained (UU) triaxial strength tests were performed on 16 soil samples. Tests were performed on Shelby tube samples on the lacustrine soils and glacial till. The test values range from about 0.14 to 1.71 tsf. The strength of lacustrine soils generally were much lower than the strength of the glacial till soils. In general, the results of the lab testing correlated fairly well with the strengths derived from CPT testing.

3.4 Shear Strength

3.4.1 Design Value Determination

For estimation of the allowable bearing capacity in glacial till the limiting design strength is that based on the undrained shear strength of the primarily clayey soil.

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

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \qquad \text{(Reference 6, page 64)}$$

where:

s_u = undrained shear strength

N_{kt} = empirical cone factor (16 was used for this project based upon previous experience at similar sites). It is pointed out that common values for in situ soft intact clays are generally taken to be between 10 and 20. Further refinement of this value may be performed once laboratory testing is complete.

σ_{vo} = total in-situ vertical stress at the depth of interest, varying and directly available from recorded CPT data

q_t = corrected cone tip resistance at the depth of interest, varying and directly available from recorded CPT data

Figures 13 through 26 show the resulting undrained shear strengths calculated from CPT data for each proposed wind turbine site investigated to date. The values generally show two material types, with the lacustrine clay represented by undrained shear strengths of less than 500 psf and the glacial till by undrained shear strengths of typically greater than 1000 psf.

In many cases, however, the glacial till did exhibit undrained shear strengths of less than 1000 psf for significant thicknesses. As such, selection of the design undrained shear strength by typical methods (selection of a lower bound value) is not suitable for this site. To arrive at the minimum required shear strength for support of the proposed foundations we back-calculated the approximate undrained shear strength required to provide a minimum allowable bearing capacity of 3,000 psf under the extreme load. Based on our experience with similar soils and foundations a minimum undrained shear strength of 1,100 psf was therefore selected as the design value for preparation of this draft report.

3.4.2 Low Strength Zones and Alternate Turbine Location Testing

Ultimately, the shear strength of the soil is the primary factor in determining the allowable bearing capacity (Section 4.4.2) as well as for determining the need for soil remediation. A review of the undrained shear strengths determined from the CPT investigation was conducted to identify zones of weak soils. Of particular concern are weak soils near the turbine foundation depth (i.e. in the upper 15 feet of soils), as well as thicker zones of weak soils (i.e. several feet thick) at depths between 15 and 50 feet.

The results of the CPT investigation indicated significant deposits of very weak clay at 32 of the 114 proposed turbine locations investigated (8, 25, 26, 27, 28, 29, 30, 42, 45, 46, 47, 49, 50, 51, 52, 53, 54, 55, 56, 59, 65, 66, 67, 70, 71, 72, 76, 77, 122, 128, 131, and 133). The CPT results also indicate significant zones of lower strength clay at varying depths throughout the soil profile at 34 of the 114 proposed turbine locations investigated (7, 9, 10, 11, 13, 15, 18, 22, 23, 26, 27, 31, 39, 40, 41, 48, 60, 70, 78, 79, 80, 82, 83, 84, 85, 86, 87, 96, 110, 125, 127, 129, 130, and 132). These sites are based on a foundation embedment depth of 9 feet below existing grade. If embedment is to be less, additional turbine sites may require mitigation.

Of these sites, turbine locations 8, 9, 25, 28, 29, 45, 46, 47, 48, 49, 50, 51, 53, 66, 67, 71, 72, 76, 77, and 131 exhibited weak soils to depths beyond that typically considered suitable for mitigation (on the order of 30 feet below existing grade). These sites may require specialized deep mitigation or a deep foundation system if they are to be considered for development.

Table 3 summarizes the required mitigation depths at each proposed turbine location. Figure 12 also shows the locations of the sites requiring remediation. As illustrated on the figure, the sites requiring soil remediation are primarily located within the areas indicated as having peat or "marsh" soils according the the soil survey data. Further discussion on potential methods of soil remediation is contained in Section 4.4.3 of this report.

3.5 Shear and Compression Wave Velocities

The results of determination of the shear and compression wave velocities are contained in Appendix B. Shear wave velocity (interval average) results measured by a seismic cone penetrometer are summarized in Table 4. The interval shear wave velocities were measured from 6.6 ft (2 meters) to the depth of the sounding. The interval shear wave velocities (V_s) were used to compute the weighted average shear velocity from the assumed base of the foundation to the end of the sounding.

The weighted average shear wave velocity (V_s) of the underlying soil at 16 selected turbine locations varied from 635 ft/s to 1,377 ft/s (Table 4). The minimum average shear wave velocity of 635 ft/s corresponding to turbine location 41 is recommended as a design basis value for performing soil stiffness calculations as part of the structural foundation design. This value will be used in the remainder of the calculations in this report for consistency.

The compression wave velocity was also measured during this investigation at the same turbine locations. Table 4 summarizes the compression wave velocity (interval average) results measured by the seismic cone penetrometer at selected locations. The average compression wave velocity (V_p) measured at selected sites varied from 3,273 ft/s to 10,143 ft/s.

The compression and shear wave velocity information was used to compute the Poisson ratio (υ). The following equation relates shear and compression waves with Poisson ratio:

$$\nu = \left(\frac{Vp^2}{2Vs^2} - 1 \right) \Big/ \left(\frac{Vp^2}{Vs^2} - 1 \right) \qquad \text{(Reference 1, page 1108)}$$

Table 4 summarizes the computed Poisson ratio values at the 16 locations where shear and compression wave velocities were both measured. The Poisson ratio ranges between 0.41 and 0.50. The minimum value of 0.41 is recommended as the design value.

3.6 Compressibility

3.6.1 Compressibility Characteristics from DMT

Flat-blade dilatometer testing (DMT) was performed at proposed turbine locations 5, 45, 48, 51, 54, 64, 70, 78, 81, 89, 92, 107, and 131. The DMT data was used to obtain the one-dimensional constraint modulus M as follows:

$$M = R_M * E_D$$ (Reference 18, page 20)

Where,

M = constrained modulus from DMT testing

R_M = Marchetti ratio factor relating M to E_D (calculated from horiz. stress index, K_D)

E_D = Dilatometer Modulus (calculated from DMT readings)

The one-dimensional constrained modulus M is related to the one-dimensional coefficient of volume compressibility m_v by the following equation:

$$M = 1/m_v$$

The constrained modulus values are used to compute the settlement of the proposed wind turbine foundations in a later section of this report.

3.6.1 Compressibility Characteristics from laboratory testing

Compressibility characteristics of soil at the site were evaluated using laboratory consolidation testing from Shelby tube samples. Consolidation testing was performed on three samples of lacustrine clay. The tests were performed according to ASTM D-2435 using the incremental loading test procedure. The void ratio vs. effective stress relationship and the deformation vs. time test results are in Appendix E.

Table 6 summarizes the consolidation test results in terms of the compression index C_c, recompression index C_r, initial void ratio e_o, and preconsolidation pressure.

The samples of lacustrine clay soil tested had preconsolidation pressures ranging from 0.42 to 0.59 tons per square foot (tsf). Initial void ratios varied from 1.02 to 1.16. The calculated compression index ranged from 0.24 to 0.27, while the recompression index ranged from 0.04 to 0.05 for all samples tested.

Based on the preconsolidation pressures from the lab testing, sample depths, and calculated overburden pressures, the lacustrine clay is normally consolidated to underconsolidated and all compression will be in the virgin compression range. Underconsolidated soils may also experience creep (settlement independent of new loads).

3.7 Shrink/Swell Potential

The shrink/swell potential of a soil is related to its liquid limit and plasticity index. Soils with Liquid Limit values less than 50 and Plasticity Index values less than 25 are considered to have low shrink-swell potential. Soils with Liquid Limit values of 50 to 60 and Plasticity Index values of 25 to 35 are considered to have moderate shrink-swell potential. Soils with Liquid Limit values greater than 60 and Plasticity Index values greater than 35 are considered to have high shrink-swell potential (Reference 2, page 63).

The Atterberg limits testing on the soils at this site indicated liquid limits ranging from 17 to 42 percent, plastic limits ranging from 11 to 22 percent, and plasticity indices ranging from 5 to 22.7 percent. Based on this testing, the site soils are considered to have low shrink-swell potential.

Soil survey mapping of the near surface soils also indicate that low plasticity clay dominates the site (Figure 4). Low plasticity clay also is considered to have low shrink-swell potential.

3.8 Compaction and CBR Testing

A total of 12 laboratory compaction tests were conducted on bulk soil samples collected across the site. Standard Proctor density testing was performed primarily on the lean and fat clay soils encountered below the topsoil across the site. One sample (T-66) appeared to be an organic clay, and widely different test results were obtained on the sample from that location. Test results on the lean to fat clay soils indicated the maximum dry densities ranged from 95.5 to 109.2 pcf, with an average of 103.1 pcf. The corresponding optimum moisture content varied from 16.8 to 25.8 percent.

Standard Proctor density testing on the sample of organic clay indicated a maximum dry density of 67.5 pcf and a corresponding optimum moisture content of 47.5 percent.

Using an in-situ moisture content of 15 percent to evaluate long term moist densities of compacted lean to fat clay soils, long term moist density values are anticipated to range from 104 to 119 pcf, with and average of 112.5 pcf. Based on these results, a minimum moist density of 110 pcf or 95% of maximum dry density as determined by Standard should be used for design and verified in the field during construction.

It should be noted that the density of the surficial organic soils and peat have a much lower dry density and use of these materials as covering backfill over the foundations is not recommended.

The results of the compaction testing can be found in Table 7.

Design for roads working areas is based in part on the strength of the subgrade that can be reasonably achieved. California Bearing Ratio (CBR) tests were completed on soil samples from the site to determine the field strength of the subgrade.

A total of 12 samples were collected from road borings across the site (Figure 8). The bulk samples were collected from soil immediately below the existing roadbeds at a depth of 1 to 4 feet below the surface. The soil samples were prepared to approximately 95 percent of the maximum standard Proctor density at the optimum moisture content. The results of the CBR testing can be found in Table 7.

Results from the samples collected below the topsoil indicate that corrected CBR values at 0.1 inch range from 0.8 to 3.2 percent, when compacted to 95 percent of the standard Proctor density at optimum moisture.

It should be noted that achieving 95 percent of a standard proctor value may be difficult with the lacustrine and organic soils and relatively high shallow moisture contents and groundwater levels across the site. It is anticipated that soil stabilization will be required, especially in the central marsh area of the site, to provide an adequate roadway subgrade.

A CBR value of 1.1 percent is recommended for road design in Section 4.1, reflecting that the very soft subgrades will be improved and should not be used as the basis for design of the entire site.

3.9 Karst Potential

Karst risk evaluation is a difficult task. While certain features may be visible at the ground surface, most of the features are hidden underground. Though not all soluble bedrock contains karst and karst like features can form in non-soluble bedrock, the presence of soluble bedrock is generally considered the key factor in determining that some risk may be present, and thereafter it is very difficult to completely eliminate that risk. The project area is underlain by bedrock that has been identified as having the potential for the development of dissolution features present elsewhere western Ohio.

The Ohio geological survey has completed an extensive mapping project to define the areas in which karst is present (Figure 7). While no karst has been mapped to date in the immediate vicinity of the project site it is present in the same geologic deposit in adjacent counties and has been identified as regularly occurring in areas with less than 20 ft of glacial till overburden. Shallow refusal was encountered within many of the CPT soundings performed on site, with refusal at 20 feet or less at proposed turbine locations 19, 35, 37, 38, and 117.

The major indicators for karst conditions are the presence of existing cave openings, sinkholes, or paleosinks. Though no sinkholes have been observed on site to date, the beginnings of a potential sinkhole consisting of raveling soils may be identified in a decrease in SPT or CPT tip resistance with depth, particularly in the immediate vicinity of tip refusal on bedrock. Though not definitive, decreasing and potentially decreasing equivalent SPT N-values were observed at proposed turbine locations 2, 3, 5, 7, 8, 9, 10, 11, 16, 17, 19, 20, 25, 29, 30, 32, 33, 35, 37, 38, 41, 46, 52, 56, 59, 65, 66, 67, 70, 77, 78, 82, 83, 86, 89, 90, 109, 113, 116, 124, 125, 129, 130, and 133.

Considering the depth of refusal, a decrease in equivalent SPT N-value, and presence of weak soils zones above refusal, there is a concern of potential karst terrain underlying select turbine locations. Based on the review of the CPT and SPT data, proposed turbine locations 7, 8, 9, 25, 30, 77, 125, and 130 are of the highest concern for karst underlying the proposed foundation location. Proposed turbine locations 10, 17, 19, 20, 32, 33, 35, 37, 38, 56, 59, 65, 67, 72, 78, 82, 98, and 113 are potentially of concern as well. Options for additional evaluation and/or preventative measures that may be taken to reduce the risk of karst formation are discussed further in Section 4.4.4.

4.0 Analysis and Recommendations

Results of the field and laboratory investigation have been presented in Section 3. Based on these results, Section 4 provides analysis, conclusions and recommendations for the design and construction of wind turbine foundations and roads, as well as general construction considerations.

For roads, the primary factors addressed include gravel thickness and subgrade preparation. For foundations, the design factors addressed include bearing capacity, footing stiffness, foundation settlement, and sliding friction.

4.1 Roadway Design

Roadway design covers preparation of surface, preparation of subgrade, and materials necessary for roadway construction.

4.1.1 Surface Preparation for Roadways

Site preparation for roadways should be initiated by removing all surface vegetation, root zones, the upper layer of organic topsoil, and loose, soft or otherwise unsuitable materials. The organic-rich topsoil thickness generally ranges from 12 to 18 inches. Actual stripping depths will likely vary and should be evaluated by a geotechnical engineer at the time of construction.

Although significant thicknesses of peat were not encountered in the borings performed for the project, peat thicknesses of up to 10 feet are reported in the Scioto Marsh. Removal of the full depth of peat will not be feasible for roadway construction. Consideration to moving the roads outside of the deep peat deposits or construction using multiple layers of geogrid or geotextile in conjunction with a thick gravel section should be considered.

Topsoil removed during site stripping should be graded into existing site topography or used as fill materials in non-critical areas. Incorporation of topsoil in compacted fill which will support turbines, roadways, pavement, equipment pads, or other site structures is not recommended. The surficial soils shall be graded to prevent accumulation of surface water and to allow for proper drainage in the vicinity of the proposed roadways.

4.1.2 Subgrade Preparation

After stripping or excavating to rough grade is complete, the exposed subsurface along the entire roadway should be proof-rolled. Proof-rolling should be performed with a fully loaded tandem axle dump truck having a minimum gross weight of 25 tons. Proof-rolling will aid in identifying areas of unstable subgrade. Proof-rolling should be performed in the presence of a geotechnical engineer. Typical standards for proof-rolling should include no rutting greater than 1 to 1-1/2 inch, and no "pumping" of the soil behind the proof-roll. Proof-rolling is not an indication that the subgrade strength is adequate or that it meets design requirements, but simply highlights potentially unsuitable subgrade conditions. If the compacted subgrade soil conditions do not meet the required compaction test results, per the construction specifications, the deficient materials shall be removed and replaced with the required thickness of additional road base material. Areas which fail proof-rolling tests should be sub-cut and replaced with suitable fill.

The clay to clayey silt glacial till and lacustrine clay soils likely will be easily disturbed by construction traffic or become unstable during proof-rolling and/or subsequent construction operations and some means of subgrade stabilization may be required to facilitate construction. Use of a vibratory roller is not recommended for cohesive soil subgrades.

In addition, with the low soil strengths, high moisture content, and high groundwater table, achieving 95 percent compaction of a Standard Proctor value is not anticipated in the marsh areas. An alternative method of subgrade stabilization likely will be needed to provide an adequate soil subbase for the project roadways. Alternatives for roadway subgrade stabilization include the following:

- **Removal and Replacement** – The inadequate materials can be removed and replaced with granular structural fill consisting of well-graded sand and gravel materials (similar to typical roadway base course materials). Compaction of this material is required to achieve a minimum of 95 percent of the laboratory maximum dry density measured according to Standard Proctor. The granular structural fill can be used in conjunction with a geotextile fabric or geogrid to potentially reduce depth of over excavation or to reduce the amount of granular materials required. However, the wet soft soil subgrades extend very deep on this site and compaction of engineered fill on the soils at the base of the overexcavation likely will be difficult to achieve. Removal and replacement in areas with engineered fill is not recommended in these areas. In areas where a thin extent of soft soils is encountered, removal and replacement could be considered as a option for stabilization.

- **Scarification and Re-compaction** – It may be feasible to scarify, dry, and re-compact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Even with adequate time and weather, however, stable subgrades may not be achievable if the thickness of the soft soil is greater than 1 to 1-1/2 feet. The soil explorations indicated areas of deep, soft soils and potentially high groundwater levels. Therefore scarification and recompaction also is not recommended in these areas.

- **Soil Stabilization** – The use of cement, lime, or fly-ash as a soil stabilizing agent can be considered in lieu of removal and replacement or scarification and recompaction. The type and quantity of materials used to stabilize the soils will be dependent upon soil type. Typically lime stabilization is used for higher moisture content clay to clayey silt soils similar to those encountered at the site. Although, cement stabilization may be better suited to areas with higher organic content, such as sites were only limited topsoil removal is desired. Design of a soil stabilization program should be performed by a geotechnical engineer in conjunction with laboratory testing to provide the proper stabilizing agent, application rate, and depth of soils stabilized. Due to the wet, soft soils encountered on the site, soil stabilization appears to be the most favorable method to improve the soil subgrades.

Placed fill for subgrade stabilization shall be compacted with a sheepsfoot or pad-foot compactor at sites on cohesive material and a smooth drum roller for granular and gravel fill material. Native clay to clayey silt materials present across the site indicate the use of the sheepsfoot or pad-foot compactors. Vibratory versions of these compactors are acceptable, although not required for cohesive soils. Vibratory rollers likely will disturb the cohesive soils, especially those with higher moisture contents as encountered on this site. The number of passes required will vary depending upon the equipment used, fill material type, and moisture condition of the fill.

Imported fill material may consist of sand, silty sand, clayey sand, sandy lean clay, lean clay, or more granular materials, although the liquid limit of these materials should not exceed 45 and the plastic index should not exceed 20. Note that fine-grained fill soils may be particularly difficult to compact if wet or allowed to become wet, or if spread and compacted over wet or marginally stable subgrades. The majority of the on-site glacial till soils likely will be suitable as fill materials, however, additional testing may be necessary to determine the suitability of materials for use as fill.

After completion of proof-rolling, but prior to placement and compaction of granular fill, any soils loosened during the excavation activities should be re-compacted to a minimum of 95 percent of the standard Proctor maximum dry density at or near the optimum moisture content.

The roadway surfaces should be crowned or sloped to prevent water ponding on or around the roadway surfaces. The roadway crowns and slopes should have a 2 percent slope to promote drainage. Culverts should be used where needed to allow drainage underneath the roadways and to prevent ponding either over or on the side of the roadways. If rain occurs during roadway construction, the subgrade should be allowed to dry prior to continuing work.

4.1.3 Road Base Design Considerations

The design thickness of placed granular fill is determined using CBR values. Based on the results discussed in Section 3.8. a CBR value of 1.1 percent will be used for the non-modified roadway subgrade compacted to 95 percent of the standard Proctor maximum dry density. The required aggregate thickness was determined by SpectraPave3 using the Giroud-Han iterative equation:

$$h = \frac{0.868 + \left(0.661 - 1.006 J^2\right)\left(\frac{r}{h}\right)^{1.5} \log N}{\left[1 + 0.204\left(\frac{3.48 CBR_{bc}^{0.3}}{CBR_{sg}} - 1\right)\right]} \left(\sqrt{\frac{\frac{P}{\pi r^2}}{\frac{s}{f_s}\left[1 - 0.9 e^{-\left\{\frac{r}{h}\right\}^2}\right] N_c f_c CBR_{sg}}} - 1 \right) r$$

where;

h = required thickness (meters)

J = aperture stability modulus (m-N/degree)

P = wheel load = axle load/2

r = radius of tire print

N = number of axle passes

CBR_{sg} = Subgrade CBR = 1.1 %

CBR_{bc} = Aggregate CBR = (~5 x CBR_{sg})

f_s = rut depth factor = 75 mm (~ 3 inches)

s = maximum rut depth = 1.5 inches and 3 inches

N_c = bearing capacity factor (5.14 for geotextile reinforced pavements)

f_c = factor relating CBR of subgrade to equivalent c_u value = 30

Two traffic conditions were evaluated and analyzed for use of the road: (1) conditions during construction of the project and (2) maintenance traffic consisting of light duty trucks.

The construction condition assumes: a subgrade CBR value of 1.1 percent; a subbase CBR value of 8 percent; a maximum axle load of 25 kips; a tire pressure of 80 psi; 800 axle passes; and maximum allowable rut depths of 1.5 and 3.0 inches. The required aggregate thickness for the construction condition varies from 6 to 32 inches, depending on the level of subgrade compaction, aggregate reinforcement and maximum allowable rut depth. Recommended thickness of aggregates for roadways is provided in Table 8.

The maintenance condition assumes: a subgrade CBR value of 1.1 percent; a subbase CBR value of 8 percent; a maximum axle load of 3.5 kips; a tire pressure of 65 psi; 2000 axle passes; and maximum allowable rut depths of 1.5 and 3.0 inches. The required aggregate thickness for the maintenance condition varies from 6 to 12 inches, depending on the level of subgrade compaction, aggregate reinforcement and maximum allowable rut depth. Recommended thickness of aggregates for roadways is provided in Table 8.

It is recommended that a minimum of 6 inches of aggregate base be placed to compensate for topsoil stripping and for aggregate stability.

Please note that axle loads and/or axle passes in excess of the design values noted above may decrease the overall life of the road because of premature road deterioration. In the event of heavy traffic leading to excessive rutting or surface deterioration, it is recommended that 2 inches of gravel be added and re-graded to reestablish the road surface.

It is recommended that granular roadway material be placed on the roadways. The granular roadway surface should consist of crushed limestone gravel. To facilitate local purchase, this

aggregate should meet the requirements of Ohio Department of Transportation standards for typical roadway base course materials. Alternative road surface materials may be used depending upon availability and suitability. A smooth drum vibratory compactor should be used to compact the gravel roadway. The gravel roadway should be compacted to 95 percent of maximum standard Proctor dry density, as determined by ASTM D-698.

4.2 Excavation and Fill

The following sub-sections present general recommendations for site clearing, grading, and compaction for construction roads, wind turbine foundations, and laydown areas.

4.2.1 Clearing and Grubbing

The project site is predominantly farmland, and clearing and grubbing will generally be restricted to the removal of planted agricultural crops or remains of crops, grass, and topsoil. Based on the borings, the thickness of this organic material or topsoil typically varies from 12 to 18 inches at the boring locations.

The topsoil and organic material is usually mixed during the excavation process, and thus, should not be used for structural fill. This material should be placed separately away from the rest of the excavated material to avoid contamination. Topsoil removed during site stripping should be graded into existing site topography or used as fill in non-critical areas. This material could be used in grading non-structural fill such as fields, or service areas in which compressibility of the material does not have an impact on overlying structures or roadways.

4.2.3 General Cut

Due to the presence of very weak zones and shallow groundwater the glacial till soils should be considered Type C soils from OSHA soil classifications (29 CFR 1926 Subpart P-Excavations). It is the responsibility of the competent field personnel at the time of construction to verify the in-situ soil classification at each excavation and insure that the benching or slopes are adequate during construction (29 CFR 1926 Subpart P-Excavations).

Bedrock was encountered across the site but a depths greater than the anticipated turbine foundation embedment depth. Rock removal for the turbine foundations is not anticipated. Excavations extending greater than 15 feet below existing grade may require rock removal.

4.2.4 General Fill

In all common fill areas outside of foundation limits, lifts should be placed as close to horizontal as possible, stepped into the existing slope, with lift thickness not to exceed 12 inches in a loose condition.

Any soil placed as common fill should be an approved material classified by the unified soil classification system (USCS) as ML, CL, SM, SC or more granular, free of organic matter or debris, rocks greater than 3 inches in diameter, and have a liquid limit and plasticity index less than 45 and 20, respectively. High plasticity silt or clay (MH, CH) or organic soils should not be used as structural fill. Crushed rock must be graded such that the placement and compaction of the material results in no voids within the fill.

Fill should be placed in lifts not exceeding 12 inches in loose thickness or one third the diameter of the roller, whichever is less, and as appropriate for the equipment being used. We recommend that each lift be compacted to a minimum of 95 percent of the maximum dry density obtained in accordance with ASTM Specification D-698, Standard Proctor Method. In areas of fill greater than 5 ft in height the entire height of the fill should be compacted to a minimum 98 percent of the maximum dry density (ASTM D698). Similarly, any soil placed and compacted to support a proposed foundation should be compacted to a minimum 98 percent of the maximum dry density (ASTM D698).

Periodic sampling and material testing (including grain size analysis, moisture content and density testing) should be performed during the work at a minimum rate of 1 suite of tests per 2,500 cubic yards of backfill. Compaction tests should be performed at the rate of 1 per 2,500 square ft of each fill lift.

4.2.5 Excavation, Backfill and Compaction for Foundations

Compaction of native soils intended for support of the foundation base is not required, unless visual inspections indicate suitable soils become disturbed during construction activities. At many of the proposed turbine locations, the native subgrade soils should be firm, stable, and capable of providing sufficient support for the structure.

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

1. Disturbed soils should be leveled, recompacted, and proof rolled. Compaction of this material is required to achieve at least 98 percent of the laboratory maximum dry density measured according to the standard Proctor method (ASTM D698). Testing should be performed to ensure proper support can be provided by the recompacted materials.

2. The inadequate materials are to be removed and replaced with granular structural fill consisting of well-graded sand and gravel materials with less than 10 percent fines (similar to typical roadway base course materials). The removal and replacement of materials shall be performed extending out from all sides of the excavation a minimum distance equal to the over excavation depth to create a 45 degree slope of replacement materials below the base of the foundation. Compaction of this material is required to achieve the greater of a minimum of 75 percent relative density or a minimum of

98 percent of the laboratory maximum dry density measured according to the standard Proctor method (ASTM D698).

Moisture content of the materials should be maintained within reasonable levels to allow for proper compaction. Compaction should only be carried out during favorable weather conditions, fill materials should not be allowed to freeze, and placed fill material subgrades should be firm and stable after placement and compaction.

In all cases of foundation excavation, the soil base shall be protected against damage by use of a flowable concrete mud mat after final excavation (or fill placement). If proper compaction of backfill or structural fill soils cannot be achieved, lean concrete (flowable fill) can be used as backfill in lieu of granular structural fill.

Based on results of Section 3.8, the native material utilized as backfill over the foundation should be compacted to achieve a minimum dry density corresponding to 95 percent of the maximum dry density as determined by a the standard Proctor method. The moist unit weight of the soils also shall meet or exceed the design value specified in the final foundation design plan and report.

Each lift should have a uniform thickness according to the equipment being used but not to exceed 12 inches in loose thickness. During construction, the top surface of the fill should be kept with sufficient slope (¼" per foot) to allow runoff of water during a rainfall without inducing erosion. Periodic inspection and testing in accordance with Section 4.2.4 should be performed during the work to verify that the recommended compaction has been achieved.

4.3 Groundwater and Dewatering

Based on the discussion presented in Section 3.2, significant groundwater seepage was generally not observed during drilling and not observed in the piezometer wells until after a significant stabilization time.

A system of sloped excavations with sump(s) and small pump(s) should be adequate to control water at the proposed turbine locations for short periods of time. Excavations allowed to remain open for longer durations of time may require more comprehensive dewatering methods, such as well points or a cut-off trench, depending on the soil conditions, to allow for construction to proceed in relatively dry conditions. In the event of heavy rainfall, the impermeable nature of the clay soils could limit water outflow from the excavation, which would also require the use of a sump and small pump for dewatering purposes. The excavations should be kept relatively free of accumulated water during construction to minimize softening of the subgrade soils. Other drainage elements such as sub-drains or fill instrumentation do not appear to be required. Water should not be allowed to pond in the base of the excavations during construction.

Long-term groundwater readings from piezometers indicated water levels as shallow as 0.7 feet below ground surface, with most groundwater levels at 1.5 to 2 feet below ground surfaec. Due to water levels at or above the proposed bearing elevation of the proposed turbines, a foundation design accounting for buoyant forces is recommended for the site. A design groundwater level of 0.5 feet below existing ground surface is recommended. If multiple foundation design are desired in an attempt to take advantage of deeper groundwater levels at some turbine locations additional piezometer installation at every potential turbine location and subsequent measurement of groundwater levels would be required.

4.4 Wind Turbine Tower Foundation

4.4.1 Foundation Type

Investigation and testing of the proposed wind turbine locations generally found clayey glacial till at the proposed turbine locations. The recommended minimum design frost depth for the wind turbine locations is 35 inches (Reference 7). Based on these conditions and the analysis presented below, a conventional spread footing bearing on soil a minimum of 9 feet below grade is a feasible and cost effective foundation system to utilize at many of the proposed turbine sites included in this investigation. Due to water levels at or above the proposed bearing elevation of the turbine foundations, a buoyant design is recommended for all turbine foundations.

4.4.2 Bearing Capacity

Allowable soil bearing pressure for a spread footing is based on the shear strength obtained from testing and investigation. A brief discussion of shear strength was provided in Section 3.4. The following is a more detailed description of the procedure used to determine design shear strength and allowable bearing capacity.

Though both granular and cohesive soils are present on site, the limiting case with respect to bearing capacity is for a foundation overlying clayey glacial till or lacustrine clay subject to failure under the undrained strength condition. Based on our experience with similar sites and turbine loads a minimum allowable bearing capacities of approximately 3,000 psf under the extreme load and 2,300 psf under the normal load are required for support of the proposed foundations without mitigation of weak soils.

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

$$q_{ult} = \frac{1}{2}\gamma' B_{eff} N_{\gamma} F_{\gamma s} F_{\gamma i} + qN_q F_{qs} F_{qi} + s_u N_c F_{cs} F_{ci}$$ (Reference 17)

where:

q_{ult} = ultimate bearing pressure

γ = unit weight of the soil

B = average footing width over the length in bearing

N_{γ} = bearing capacity factor

q = surcharge at foundation level

N_q = bearing capacity factor

s_u = design undrained shear strength of the soil
N_c = bearing capacity factor

F = shape (subscript "s") and inclination (subscript "i") factors

The first term of the above equation is associated with granular soils which typically exhibit drained modes of failure (except under earthquake loading) and where excess pore pressures cannot build up in the soil when sheared. This term represents the ultimate drained bearing capacity.

The third term of the equation is associated with fine-grained/clayey soils which typically exhibit an undrained mode of failure and where excess pore pressures can build up in the soil when sheared. Since the soils encountered at the project site are pre-dominantly fine grained (clays), the critical mode of failure is associated with an undrained condition, the first term is dropped from the equation, and the second term reduces to the overburden pressure, representing the ultimate undrained bearing capacity shown as follows:

$$q_{ult} = q + s_u N_c F_{cs} F_{ci} \qquad \text{(Reference 17)}$$

The following are formulas for the dimensionless factor, N_c, and shape (F_{cs}) and inclination (F_{ci}) factors above (Reference 17):

$$N_q = e^{\pi \tan\phi} \frac{1+\sin\phi}{1-\sin\phi} = 1 \text{ (for } \phi = 0)$$

$$N_c = (N_q - 1)\cot\phi = 5.14 \text{ (for } \phi = 0)$$

$$F_{cs} = 1 + 0.2\frac{B_{eff}}{L_{eff}}$$

$$F_{ci} = \frac{1}{2} + \frac{1}{2}\sqrt{1 - \frac{H_d}{A_{eff} s_u}}$$

where:

B_{eff} = average effective footing width

L_{eff} = average footing length

H_d = design horizontal load

A_{eff} = effective area as a result of a wind load causing a moment on the foundation

The allowable soil bearing pressure is then obtained by dividing the ultimate bearing capacity by an appropriate factor of safety. The following factors of safety are taken from Reference 1:

$$\text{Factor of safety} = \begin{cases} 2.26 & \textit{for extreme wind loading} \\ 3.0 & \textit{for normal operation loads} \end{cases}$$

Based on back calculation from the desired allowable bearing capacities of 3,000 and 2,300 psf under the extreme and normal loading conditions, respectively, a minimum undrained shear strength of 1,100 psf is required of the cohesive soils supporting the proposed foundations. Those proposed turbine locations with significant soil deposits of undrained shear strength less than 1,100 psf must be mitigated (Sections 3.4.2 and 4.4.3).

4.4.3 Turbine Locations Requiring Soil Remediation

All of the proposed turbine locations were analyzed for lower undrained shear strength soils near the turbine foundation depth (i.e. in the upper 15 feet of soils) or thicker zones of low strength soils (i.e. roughly 5 to 10 feet or more in thickness) at depths between 15 and 50 feet.

The results of the CPT investigation indicated significant deposits of weak clay at 20 of the 114 proposed turbine locations investigated (8, 9, 25, 28, 29, 45, 46, 47, 48, 49, 50, 51, 53, 66, 67, 71, 72, 76, 77, and 131) to depths beyond that typically considered suitable for mitigation (on the order of 30 feet below existing grade). These sites would require specialized deep mitigation or a deep foundation system if they are to be considered for development.

The CPT results also indicate significant deposits of weak clay that may be mitigated by standard means at 42 of the 114 proposed turbine locations investigated (7, 10, 11, 13, 15, 18, 22, 23, 26, 27, 30, 31, 39, 40, 41, 42, 54, 55, 56, 59, 60, 65, 70, 78, 79, 80, 82, 83, 84, 85, 86, 87, 96, 110, 122, 125, 127, 128, 129, 130, 132, and 133). These sites are based on a foundation embedment depth of 9 feet below existing grade. If embedment is to be less additional turbine sites will require mitigation. Table 3 summarizes the required mitigation depths and anticipated type of improvement at each proposed turbine location.

Options for soil remediation include: (1) engineered fill (2) stone columns/geopiers, (3) deep foundations, and (4) alternate turbine locations. Details regarding each option for soil remediation are discussed below.

4.4.3.1 Engineered Fill

At sites where relatively shallow soils exhibit low shear strength, one possible option for remediation is excavation of the soft material and backfill with adequate material.

It should be noted that these depths of overexcavation are based on shear strength values obtained from CPT testing, and may vary depending on the conditions encountered during construction. Other turbine sites also may require minor overexcavations of low strength soils, or excavations may need to be extended deeper to remove low strength soils. The base of every footing should be inspected by the geotechnical engineer or a representative of the geotechnical engineer to evaluate the soil conditions at the bearing elevation or bottom of overexcavation elevation.

An engineered fill approach is generally undertaken when improvement depths are no greater than 15 feet below existing grade. At depths beyond 15 feet, the excavations may become too large and expensive as a result of the need for over sizing (as discussed below) and some equipment limitations. Engineered fill used to replace the shallow soft soils should be similar to that described by Ohio Department of Transportation standards for typical roadway base course materials and contain less than 10 percent by weight passing the number 200 sieve. Loose lifts should not to exceed 12 inches. The backfill should be compacted to the greater of a minimum of 98 percent of a maximum dry density as determined by standard Proctor method, or 75 percent relative density. The excavations and subsequent engineered fill should be oversized one (1) foot on all sides for each foot of excavation below the foundation embedment depth of 9 feet. For example, a one (1) foot excavation below the foundation depth will require a bottom of excavation width (and length) two (2) feet greater than a standard excavation width (one foot on each side of the footing).

It should also be noted that water was observed at depths as shallow as 2.5 feet below existing grade at piezometer locations after significant stabilization times. As a result, the presence of water could adversely affect the foundation over-excavation. Due to the presence of primarily fine grained soils (clays) encountered during the drilling operations, it is recommended that sumps be installed to prevent water from collecting in the foundation bottom. Depending on the amount of water seeping from the excavation sidewalls from silt or sand seams, it may also be necessary to install dewatering wells in the vicinity of the excavation.

Dewatering of the excavations will be crucial to achieving the specified level of compaction on the structural fill materials placed beneath the proposed turbine foundations. If the specified level of compaction cannot be achieved, lean concrete (flowable fill) can be used instead of granular structural fill materials.

4.4.3.2 Stone Columns/Geopiers

One possible ground improvement method for deeper or thicker low strength soil layers at the turbine locations is construction of stone columns or geopiers. Stone columns or geopiers are constructed by drilling to open a hole, then placing aggregate and using deep vibratory methods to densify the aggregate and surrounding native soils to increase the strength and stiffness of the aggregate and surrounding soil. Stone columns or geopiers are generally used when the depth of improvement ranges from 15 to 30 feet (although the upper limit is dependent on the relative proximity of an aggregate source to the site and the contractor's experience) and the lower limit is determined by improvement of the soils which can be attained. This method may be advantageous for the turbine sites where over-excavations are not feasible due to the depth of soil requiring remediation or the presence of shallow groundwater.

If selected for use on the site, stone columns or geopiers should be installed to improve the bearing capacity and settlement characteristics of the soils. Modulus tests should be conducted on the stone columns or geopiers to verify the bearing capacity of the improved soil stratum.

Stone columns and geopiers are generally provided on a design build basis.

4.4.3.3 Deep Pile Foundation System

Deep foundation systems are commonly comprised of steel pipe piling and installed either by a diesel hammer or auger-cast method. Typically, deep foundations are used at sites where the extent of low strength soil extend deep below the proposed bearing elevation of the foundations. At this depth, stone columns may become more expensive than a deep foundation system. Based on the results of the CPT soundings and soil borings, proposed turbine locations 8, 9, 25, 45, 46, 47, 49, 50, 51, 53, 66, 67, 71, 72, 76, 77, and 131 would likely require a deep foundation system for support.

4.4.3.4 Use of Alternate Location

A fourth option is to move the turbine locations requiring soil remediation for turbine foundation support and utilize alternate turbine locations. Review of the locations and depths of the turbine sites requiring remediation or deep foundations (see Figure 12) indicate that the soils surrounding the marsh will likely require less remediation for turbine support. It may be advantageous to select alternate sites outside of the marsh area to lessen the need for remediation. Taking into account site soil variability, if new (unexplored) turbine sites are selected which are greater than 50 feet from the current turbine locations, each new site should be evaluated for adequate foundation support by geotechnical explorations and engineering analysis of the test results. Evaluation at new turbine sites should generally consist of CPT soundings.

4.4.4 Karst Risk Mitigation

If a site has been identified as having a high or moderate risk for karst terrain, follow-up site visits are typically recommended to document features that could lead to subsurface failures. In the case of this site, however, the agricultural nature of the site results in disturbance of the surficial soils and obscuring any topography that would indicate karst is present. As such, other options for additional evaluation of karst potential include:

- Surveying the landowners regarding historic sinkholes, etc.
- Detailed geophysical investigation
- Borings extended into the underlying bedrock

In our opinion, a two-tiered should be considered, beginning with a survey of local persons (land owners, county highway departments, university professors, geological society members, cavers, etc…) followed by either a geophysical investigation or additional borings at the sites of highest concern. Due to the presence of high groundwater and variable overburden, however, geophysical testing may not be ideal for this site. While not a comprehensive evaluation of risk, this approach provides a cost effective method to minimize the risk posed by karst. As a general rule, the more one tries to reduce the risk, the higher the costs. The site developer carries the risk and must ultimately decide what the balance point is for risk reduction and cost.

4.4.5 Foundation Stiffness

Elastic theory relates shear wave velocity with the shear modulus at small strain using the following equation:

$$G_o = \rho V_s^2 \qquad \text{(Reference 6, page 74)}$$

where

G_o = shear modulus at small strain

V_s = shear wave velocity using CPT seismic data

ρ = mass density of the soil. The mass density is the ratio of the unit weight (γ) and the acceleration of gravity, g (32.2 ft/s^2 or 9.81 m/s^2).

In order to calculate the design shear modulus, the minimum average shear wave velocity of 635 ft/s was selected as the design value (Section 3.5). All test locations either met or exceeded this value. Based on an assumed design moist unit weight of 120 lbs/ft^3 the shear modulus at small strain is computed to be 1,500 kips per square foot (ksf). This value corresponds to the small strain shear modulus. For foundation design, the structural engineer should reduce the shear modulus based upon the estimated level of soil stress caused by the foundation in order to account for large strain conditions. Once the shear modulus (G) is determined, other moduli such as Young's modulus (E) and bulk modulus (K) can be derived based on the relations $E = 2(1 + \upsilon)G$ and $K = E/(3(1 - 2\upsilon))$ where υ is the Poisson' ratio.

4.4.6 Sliding Friction

The friction coefficient between the clayey soil of the site and concrete should be taken as 0.68 in accordance with recommendations provided by Potyandy (Reference 15), assuming a smooth concrete surface.

4.4.7 Foundation Settlement

Immediate, long-term and differential settlements of the foundation were computed based on results of the geotechnical investigation and testing described here. GE limits the tilt of the foundation to 0.17 degrees (3mm/m) for soil settlement (Reference 5); however, a total settlement limit is not stipulated. Based on the anticipated turbine foundation, an allowable differential settlement of approximately 1 inch corresponds to 3mm/m under the normal operating load for a 60 ft diameter foundation.

4.4.7.1 Immediate Settlement

The immediate settlement can be computed based on the application of the mean wind load with buoyant conditions, using the following equation based on elastic theory:

$$S = \frac{B_{eff} q_o}{E_s} \left(1 - \upsilon^2\right) I$$ (Reference 4, page 7.16)

where:

S = immediate settlement

q_o = buoyant contact pressure (1,750 psf – estimated)

B_{eff} = effective foundation width (45 feet – estimated)

υ = Poisson's ratio = 0.41 (from Section 3.5)

E_s = elastic soil modulus = $2G_o\ (1 + \upsilon)b$ = 1,269 ksf (G_o = 1,500 ksf from Section 4.4.5 and b = 0.3, typical reduction factor from small strain to one percent strain)

I = shape factor = 1.12 (Reference 4, page 7.17)

This formula allows for a quick estimate of the immediate settlements induced by the footing. From the parameters and values considered, an immediate settlement based on the native soils is anticipated to be less than 1 inch for the current buoyant foundation design. This settlement should be reevaluated during preparation of the final foundation design to account for any changes to the above assumed values.

4.4.7.2 Long-term Settlement from DMT

The long-term settlement was estimated using the data collected from the DMT test data. The procedure proposed by Schmertmann, which uses the one-dimensional constraint modulus *M,* was used to calculate the settlement. In this procedure, the soil strata under the proposed foundation are subdivided into several layers. Then the stress increment induced by the foundation load is calculated at the mid-point of each layer. The compression of each layer can be computed using the following equation:

$$s_t = \sum_i \Delta\sigma' \frac{\Delta H_i}{M_i}$$

where:

$\Delta\sigma'$ is the effective stress at the mid-point of the soil layer,

ΔH_i is the thickness of layer i, and

M_i is the average one-dimensional constraint modulus of the layer i.

M values derived from the DMT are discussed in Section 3.6.1. Using this equation, the settlement from the DMT results can be calculated based on the application of the mean operating load at the midpoint of each layer. To calculate the consolidation settlement, the soil is be split into several layers, with the total settlement calculated as the sum of the individual layer settlements.

Considering the time required for long-term settlement from cohesive soils, differential settlement will be realized as a function of the applied foundation load during normal operating conditions at the center of the affective bearing area, and at the edge of the turbine foundation. Based on the results of the settlement estimated from the consolidation testing and DMT testing, the differential settlement across the turbine foundation can be calculated using the following equation.

$$\Delta S = \frac{S - S_{edge}}{B_{mean}/2}$$

where:

ΔS = differential settlement

S = settlement due to mean operating load (calculated settlement at center of affective bearing area)

S_{edge} = settlement due to mean operating load (calculated settlement at edge of the foundation)

B_{mean} = Elliptical bearing area width (calculated from 2*(R-e_{mean}))

R = Foundation Radius

e_{mean} = Load eccentricity under mean operating conditions

The soil parameters over the footing width should remain constant, so only the applied load from the turbine differs from the center of the affective area to the edge of the turbine foundation.

The applied load from the wind turbine foundations at the center of the affective bearing area and at the edge of the foundation were calculated using the following formula:

$$\Delta\sigma = I * q$$

where,

$\Delta\sigma$ = applied foundation load at midpoint of soil layer(variable with depth and location)

I = Influence factor varying with depth and location beneath foundation

q = foundation bearing pressure for mean operating load condition

The influence factor at the center of the effective bearing area was calculated using the following equation:

$$I = 1 - \frac{1}{[(0.5B \,/\, (Dmid - Df))^2 + 1]^{3/2}}$$ (Reference 3, page 132)

Where,

B = foundation width

$Dmid$ = depth to midpoint of soil layer

Df = bearing depth of foundation

The influence factor for the edge of the foundation was calculated using the design figure, Influence Value for Vertical Stress Under Uniformly Loaded Circular Area (Reference 7 page 7.1-169), with an x/r value of 1.0, which is at the edge of the loaded area under mean loading conditions.

Influence values at the center of the area and at the edge of the foundation were calculated for at the midpoint of each layer where DMT settlement was analyzed and settlement was calculated using the appropriate formulas noted herein, for the type of test data used. The total and differential settlement at the test locations is provided in Table 5. These values are preliminary and should be revised during final foundation design to account for changes to the effective bearing area and load distribution from those values assumed herein.

4.4.7.3 Long-term Settlement from Laboratory Consolidation Testing

The long-term settlement of the foundation can be computed using the results of the consolidation test results and the following equation:

$$S = \frac{C_r}{1+e_o} * L * \log\left(\frac{\sigma'_p}{\sigma'_{vo}}\right) + \frac{C_c}{1+e_o} * L * \log\left(\frac{\sigma'_f}{\sigma'_p}\right)$$ (Reference 2, page 272)

where:

C_r = recompression index

C_c = compression index

e_o = initial void ratio

L = height of soil layer

σ'_p = minimum past effective stress where soil transitions from overconsolidated to normally consolidated

σ'_{vo} = original effective stress at the midpoint of the clay layer below foundation (mean operating load conditions)

σ'_f = final effective stress equal to $\sigma'_{vo} + \Delta\sigma$, where $\Delta\sigma$ = average pressure increase to the clay layer caused by the applied foundation load

Using this formula, the long-term consolidation settlements induced by the footing can be calculated, based on the application of the mean operating load. To calculate the consolidation settlement, the soil should be split into several layers, with the effective stress recalculated at the midpoint of each layer. Samples were obtained from the wet, gray lacustrine clay soils. The results of the consolidation testing indicate that the clay is normally consolidated to underconsolidated and applied foundation loads will cause virgin (normally consolidated) compression of any of the soil layers.

Using the laboratory consolidation test values for turbine sites 49, 51, and 66 (three of which had lacustrine soils for the majority to all of the soil profile) indicates potential differential settlement on the order of about 6 to 7 inches. This will greatly exceed the 3mm/m threshold for the turbine foundations.

Considering that some of the clay soils appear underconsolidated, it may be appropriate to consider negative skin friction (downdrag) on the sides of piles, should they be used in foundation design.

4.4.7.4 Turbine Locations with Excessive Differential Settlement

Based on the calculations detailed in Section 4.4.6.2 of this report, of the thirteen proposed turbine locations tested by DMT, seven showed unacceptable levels of differential settlement (Table 5). These turbine locations correspond directly to the sites with lacustrine, marsh, or depression deposits extending significant depths beneath the turbine sites. These sites also were identified in Sections 3.4.2 and 4.4.3 as requiring mitigation for support of the proposed foundations due to low bearing capacity/strength concerns. The required mitigation for bearing capacity will also address the unacceptable levels of estimated differential settlement. As such, any site identified as either suitable with respect to bearing capacity without mitigation or suitable for bearing capacity following mitigation is suitable with respect to differential settlement as well, once the required mitigation has been completed.

4.4.8 Backfill Density

Standard Proctor testing performed as a part of this geotechnical evaluation was ongoing at the time of this draft report. In the interim an in-situ unit weight of 120 pcf is recommended for use in the ongoing foundation design pending the laboratory test results.

4.4.9 Soil Chemical Content and Cement Type

The results of the chemical testing from 12 samples (Appendix H) indicate that the soils have pH ranging from 6.0 to 7.7. The analytical laboratory testing results indicate that the soils contain 37 to 66 ppm of Chlorides (detection limit) and 55 to 3,500 ppm soluble sulfates. The laboratory test results are included in Appendix G.

The results from turbine sites 66 and 72 exceed 2,000 ppm which is considered severe sulfate exposure. The remaining test sites indicated negligible sulfate exposure (Reference 20, page 79.) As a result, Type V cement or a specialized mix design of Type II cement in conjunction with fly ash can be considered for use on this site.

5.0 Limitations of Analysis

The analysis and conclusions provided are based on the results of fieldwork which focused on investigation of 112 potential wind turbine locations for the Hardin Wind Project. Using generally accepted engineering methods and practices, both the investigation performed and the data gathered has made every reasonable effort to characterize the project site. However, the likelihood that conditions may vary from any specific location tested is still possible, and careful attention to soil conditions should be undertaken during the time of construction by qualified personnel.

During construction, if turbines are subsequently relocated to different locations, additional soil testing may be recommended to ensure that soil conditions at the relocated turbines sites are of similar characteristics to that assumed when designing this project. Based on the conditions encountered at the project site, turbine relocations greater than 50 feet from the existing soil sounding or boring location will require re-testing.

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Tables

Table 1
Proposed Turbine Coordinates and Testing Summary

Turbine Number	UTM Zone 17 NAD 83		WGS 84		CPT Testing	Seismic Testing		Soil Borings and Lab	DMT Testing	Electrical Resistivity Testing	Thermal Resistivity Testing
	East	North	WGS 84 Longitude	WGS 84 Latitude		Vs	Vp				
1	260491	4509449	-83.834914	40.701155							
2	260246	4509134	-83.837690	40.698250	X			X		X	X
3	260249	4508733	-83.837501	40.694643	X	X	X				
4	260574	4508357	-83.833516	40.691354							
5	261049	4509473	-83.828326	40.701533	X				X		
6	261128	4509183	-83.827282	40.698947	X						
7	261208	4508861	-83.826213	40.696073	X						
8	261360	4508570	-83.824306	40.693498	X						
9	261579	4508328	-83.821625	40.691384	X						
10	261702	4507886	-83.820003	40.687443	X						
11	261778	4507534	-83.818971	40.684298	X						
12	262525	4509245	-83.810789	40.699908	X	X	X				
13	262687	4508976	-83.808772	40.697535	X						
14	262825	4508706	-83.807039	40.695145	X			X		X	
15	260856	4507957	-83.830030	40.687837	X						
16	260862	4507603	-83.829824	40.684653	X						
17	260869	4507304	-83.829628	40.681965	X						
18	260956	4507020	-83.828491	40.679435	X						
19	261043	4506725	-83.827351	40.676806	X			X			X
20	260133	4507959	-83.838577	40.687645	X						
21	260266	4507700	-83.836906	40.685353	X						
22	261922	4506980	-83.817059	40.679354	X						
23	262059	4506685	-83.815328	40.676739	X	X	X				
24	262128	4506277	-83.814358	40.673088							
25	262144	4505900	-83.814026	40.669701	X						
26	262362	4505663	-83.811361	40.667631	X						
27	262919	4506205	-83.804983	40.672668	X						
28	262944	4505799	-83.804534	40.669022	X						
29	263086	4505398	-83.802705	40.665455	X						
30	261606	4505686	-83.820303	40.667620	X						
31	261529	4505067	-83.820978	40.662028	X			X			
32	260659	4505242	-83.831324	40.663351	X						
33	259497	4506173	-83.845410	40.671390	X	X	X				
34	259858	4506121	-83.841125	40.671027	X						
35	260246	4506091	-83.836528	40.670870	X			X		X	X
37	259494	4504345	-83.844746	40.654941	X						
38	260116	4504490	-83.837453	40.656427	X	X	X				
39	263844	4507759	-83.794636	40.686916	X			X		X	
40	263835	4507466	-83.794632	40.684277	X						

Turbine Number	UTM Zone 17 NAD 83		WGS 84		CPT Testing	Seismic Testing		Soil Borings and Lab	DMT Testing	Electrical Resistivity Testing	Thermal Resistivity Testing
	East	North	WGS 84 Longitude	WGS 84 Latitude		Vs	Vp				
41	263827	4507173	-83.794616	40.681639	X	X	X				
42	263818	4506880	-83.794613	40.679000	X						
43	263804	4506575	-83.794663	40.676251							
44	263838	4506218	-83.794127	40.673049							
45	263797	4505759	-83.794439	40.668907	X				X		
46	264092	4505590	-83.790890	40.667470	X						
47	264341	4505335	-83.787852	40.665247	X						
48	264366	4504981	-83.787423	40.662069	X	X	X		X		
49	263661	4504602	-83.795611	40.658457	X			X		X	X
50	263594	4504326	-83.796299	40.655954	X						
51	263477	4504005	-83.797561	40.653033	X			X	X		X
52	264012	4503894	-83.791198	40.652187	X						
53	263345	4503622	-83.798976	40.649549	X						
54	263375	4503270	-83.798489	40.646390	X				X		
55	263532	4502957	-83.796517	40.643618	X						
56	263605	4502615	-83.795526	40.640562	X						
57	263720	4502188	-83.794007	40.636753	X						
58	263946	4501959	-83.791252	40.634757	X						
59	264484	4503856	-83.785608	40.651980	X						
60	264525	4503558	-83.785012	40.649310	X						
61	264554	4503290	-83.784569	40.646907							
62	264596	4502930	-83.783938	40.643679	X			X		X	X
63	264834	4502684	-83.781035	40.641534	X	X	X				
64	264954	4502405	-83.779513	40.639057	X				X		
65	262701	4501657	-83.805843	40.631683	X						
66	262764	4501392	-83.804999	40.629316	X			X		X	
67	262837	4501104	-83.804029	40.626746	X						
68	262984	4500839	-83.802193	40.624404	X	X	X				
69	263063	4500513	-83.801137	40.621493							
70	262959	4500145	-83.802227	40.618152	X				X		
71	262264	4500477	-83.810559	40.620939	X						
72	262202	4500169	-83.811175	40.618150	X			X		X	
73	263571	4501308	-83.795437	40.628792	X						
74	263697	4500723	-83.793729	40.623564							
75	263826	4500509	-83.792126	40.621675							
76	265028	4507206	-83.780433	40.682278	X						
77	265224	4507002	-83.778040	40.680499	X						
78	265800	4507611	-83.771459	40.686142	X				X		
79	265946	4507349	-83.769636	40.683826	X						
80	266056	4507067	-83.768231	40.681320	X						
81	266085	4498110	-83.764558	40.600731	X				X		
82	266102	4506509	-83.767479	40.676312	X	X	X	X		X	

Turbine Number	UTM Zone 17 NAD 83		WGS 84		CPT Testing	Seismic Testing		Soil Borings and Lab	DMT Testing	Electrical Resistivity Testing	Thermal Resistivity Testing
	East	North	WGS 84 Longitude	WGS 84 Latitude		Vs	Vp				
83	265284	4508628	-83.777939	40.695147	X						
84	265631	4508545	-83.773806	40.694499	X						
85	266029	4508389	-83.769043	40.693208	X						
86	266285	4508195	-83.765944	40.691535	X						
87	266892	4508135	-83.758746	40.691167	X						
88	266792	4508520	-83.760071	40.694603	X	X	X	X		X	
89	265582	4496351	-83.769843	40.584760	X				X		
90	267549	4509260	-83.751396	40.701475	X						
91	267559	4508931	-83.751156	40.698518	X						
92	267705	4508527	-83.749280	40.694924	X				X		
93	268146	4509313	-83.744357	40.702120	X						
94	268349	4509090	-83.741874	40.700171	X						
95	265927	4496101	-83.765678	40.582608	X	X	X				
96	268296	4508559	-83.742305	40.695378	X						
97	269077	4508956	-83.733217	40.699169	X	X	X				
98	269069	4508649	-83.733199	40.696405	X						
99	265425	4496681	-83.771818	40.587685	X			X		X	
100	268345	4507995	-83.741517	40.690316							
101	267330	4507683	-83.753400	40.687223							
102	265955	4498613	-83.766280	40.605220	X						
103	267897	4507472	-83.746620	40.685484							
104	268175	4507355	-83.743291	40.684510							
105	268456	4507242	-83.739927	40.683572							
106	268735	4507123	-83.736586	40.682579							
107	269776	4506585	-83.724083	40.678029	X				X		
108	266963	4507110	-83.757526	40.681964	X						
109	267267	4506884	-83.753849	40.680016	X						
110	267526	4506629	-83.750693	40.677794	X						
111	270613	4509989	-83.715435	40.708894	X						
112	271441	4510527	-83.705839	40.713965	X						
113	271473	4509681	-83.705153	40.706361	X						
114	271543	4509456	-83.704243	40.704356							
115	272158	4509120	-83.696849	40.701502							
116	272141	4508780	-83.696926	40.698438	X						
117	270579	4508384	-83.715250	40.694441	X			X		X	
118	270668	4508105	-83.714095	40.691956	X	X	X				
119	265676	4499139	-83.769769	40.609874	X						
120	265900	4498897	-83.767034	40.607760	X	X	X				
121	267621	4507596	-83.749928	40.686522							
122	266036	4506777	-83.768359	40.678705	X						
123	267884	4508208	-83.747046	40.692103	X						
124	268306	4508818	-83.742282	40.697711	X						

Turbine Number	UTM Zone 17 NAD 83		WGS 84		CPT Testing	Seismic Testing		Soil Borings and Lab	DMT Testing	Electrical Resistivity Testing	Thermal Resistivity Testing
	East	North	WGS 84 Longitude	WGS 84 Latitude		Vs	Vp				
125	266199	4506137	-83.766194	40.672992	X						
126	266285	4505856	-83.765073	40.670488	X						
127	261775	4507236	-83.818894	40.681615	X						
128	265605	4504959	-83.772775	40.662224	X						
129	264311	4501585	-83.786801	40.631496	X	X	X				
130	267685	4506413	-83.748734	40.675895	X						
131	264430	4507160	-83.787484	40.681694	X				X		
132	262970	4506951	-83.804662	40.679395	X						
133	263033	4506622	-83.803793	40.676453	X						
SS-1	263766	4501718						X			
SS-2	263787	4501741						X		X	X
O&M-1	264012	4501767						X			
O&M-2	264079	4501775						X			

Table 2
Groundwater Levels from Standpipe Piezometers

Turbine ID	Soil Boring	Initial Water Condition (ft)	Second Reading		Third Reading	
			Date	Depth BGS (ft)	Date	Depth BGS (ft)
2	8/2/2011	Dry	8/4/2011	3.9	11/21/2011	1.4
14	8/10/2011	Dry	8/10/2011	4.3	11/21/2011	3
19	8/8/2011	Dry	8/10/2011	--	11/21/2011	1.8
31	8/9/2011	Dry	8/10/2011	5.1	11/21/2011	0.7
35	8/8/2011	12.6	8/10/2011	4.8	11/21/2011	1.9
39	8/15/2011	Dry	8/10/2011	9.1	11/21/2011	2.4
49	8/4/2011	Dry	8/10/2011	9.7	11/21/2011	damaged
51	8/15/2011	Dry	--	--	--	--
62	8/14/2011	Dry	--	--	--	--
66	8/3/2011	Dry	8/4/2011	7	11/21/2011	1.1
72	8/3/2011	Dry	8/4/2011	2.7	11/21/2011	1.9
82	8/9/2011	Dry	8/10/2011	7.6	11/21/2011	3.9
88	8/9/2011	13.53	8/10/2011	5.5	11/21/2011	1.6
99	8/4/2011	--	--	--	--	--
117	8/9/2011	Dry	8/10/2011	5.1	11/21/2011	2.6

Table 3
Mitigation Depth Summary

Turbine	Refusal (ft)	HSA Depth to Rock/ Refusal(ft)	Decreasing SPT (from CPT)	Weak Lacustrine Clay Depth (ft)	Significant Weak Zones < 1100 psf Below 9ft				Required Mitigation Depth (ft)	Anticipated Remediation Type	Karst Concern
1	--		--	--	--	--	--	--	--	--	--
2	33.45	45.3	Possible	--	--	--	--	--	--	--	--
3	34.09		Possible	--	29-31.5	--	--	--	--	None*	--
4	--		--	--	--	--	--	--	--	--	--
5	37.86		Possible	--	--	--	--	--	--	--	--
6	34.58		No	--	--	--	--	--	--	--	--
7	30.91		Yes	--	14.5-15	29.5-30	--	--	15	Geopier	Yes
8	32.91		Yes	27-33	15.5-17.5	27-33	--	--	33	Geopier/Piles	Yes
9	36.7		Possible-Yes	--	14.5-16.5	19.5-20.5	25-27.5	33-34.5	35	Geopier/Piles	Yes
10	50.03		Yes	--	15.5-16.5	19-22	--	--	22	Geopier	--
11	39.73		Possible- No	--	9-10.5	12-12.5	--	--	12.5	Geopier	--
12	26.18		No	--	--	--	--	--	--	--	--
13	30.4		No	--	11.5-12	--	--	--	12	Overexcavation	--
14	27.46	28	No	--	--	--	--	--	--	--	--
15	50.03		No	--	14.5-16.5	--	--	--	16.5	Geopier	--
16	39.73		Possible- No	--	33-34.5	--	--	--	--	None*	Maybe
17	36.29		Possible- No	--	24-25	33-34.5	--	--	--	None*	Maybe
18	25.51		No	--	11.5-12	--	--	--	12	Overexcavation	--
19	20	32	Possible- No	--	16-17.5	--	--	--	--	None*	Maybe
20	34.45		Yes	--	--	--	--	--	--	--	Maybe
21	44.03		No	--	26-27.5	37-38	--	--	--	None*	--
22	50.03		No	16	27-28	30-30.5	--	--	16	Geopier	--
23	34.61		No	11.5	23-25 very weak	--	--	--	25	Geopier	--
24	--		--	--	--	--	--	--	--	--	--
25	46.8		Possible	17.5	37-45 partially very weak		--	--	45	Geopier/Piles	Yes
26	28.46		No	16.5	--	--	--	--	16.5	Geopier	--
27	50.03		No	17	--	--	--	--	17	Geopier	--
28	50.05		No	20	27.5-29 very weak	--	--	--	29	Geopier/Piles	--
29	50.03		Possible- No	20.5	34.5-36	--	--	--	20.5	Geopier/Piles	--
30	25.72		Yes	9.5	11-12.5	23.5-25	--	--	12.5	Overexcavation	Yes
31	22.26	23.5	No	--	9-10.5	13.5-14.5	--	--	14.5	Overexc/Geopier	--
32	31.61		Yes	--	--	--	--	--	--	--	Maybe
33	30.43		Yes	--	--	--	--	--	--	--	Maybe
34	27.97		No	--	--	--	--	--	--	--	--
35	14.26	18	Yes	--	--	--	--	--	--	--	Maybe
36	--		--	--	--	--	--	--	--	--	--
37	20.8		Yes	--	--	--	--	--	--	--	Maybe
38	20.19		Yes	--	--	--	--	--	--	--	Maybe
39	29.56	33	No	--	14-23	--	--	--	23	Geopier	--
40	35.37		No	--	14-25	--	--	--	25	Geopier	--
41	38.78		Possible	--	15.5-20	--	--	--	20	Geopier	--
42	49.72		No	24	--	--	--	--	24	Geopier	--
43	--		--	--	--	--	--	--	--	--	--
44	--		--	--	--	--	--	--	--	--	--
45	50.03		No	44	--	--	--	--	44	Piles	--
46	50.03		Possible	35	--	--	--	--	35	Piles	--
47	50.05		No	31	--	--	--	--	31	Geopier/Piles	--
48	50.03		No	29	--	--	--	--	29	Geopier/Piles	--
49	50.03	>50	--	50+	--	--	--	--	50+	Piles	--
50	50.03		No	43	--	--	--	--	43	Piles	--
51	50.03	>50	--	50+	--	--	--	--	50+	Piles	--
52	50.05		Possible	28.5	--	--	--	--	28.5	Geopier	--
53	50.05		--	50+	--	--	--	--	50+	Piles	--
54	50.03		No	23.5	--	--	--	--	23.5	Geopier	--
55	35.25		Possible	22.5	--	--	--	--	22.5	Geopier	--
56	50.03		Possible	14.5	23.5-25.5	40-42	--	--	14.5	Overexc/Geopier	Maybe
57	38.12		No	--	--	--	--	--	--	--	--
58	50.03		No	--	--	--	--	--	--	--	--
59	50.03		Possible	15	44-45	--	--	--	15	Overexc/Geopier	Maybe
60	44.82			--	10.5-11.5	12.5-16	--	--	16	Geopier	--
61	--		--	--	--	--	--	--	--	--	--
62	41.73	44	No	--	12-12.5	--	--	--	--	None*	--
63	--		--	--	--	--	--	--	--	--	--
64	46.31		No	--	--	--	--	--	--	--	--
65	50.03		Yes	20	35.5-39.5	--	--	--	20	Geopier	Maybe
66	50.03	>50	Possible- No	35.5	42-44	--	--	--	44	Geopier/Piles	--
67	50.03		Possible- No	37	41.5-45 mixed	--	--	--	37	Geopier/Piles	Maybe
68	43.5		No	--	--	--	--	--	--	--	--
69	--		--	--	--	--	--	--	--	--	--
70	50.03		Possible	22	--	--	--	--	22	Geopier	--
71	50.02			50+	--	--	--	--	50+	Piles	--
72	50.05	>50	No	to 7	23.5-25	31-32.5	34-42.5	--	43	Piles	Maybe
73	50.03		No	--	27-28	--	--	--	--	None*	--
74	--		--	--	--	--	--	--	--	--	--
75	--		--	--	--	--	--	--	--	--	--
76	50.03		No	13.5	26.5-27	30-43.5			44	Geopier/Piles	--
77	50.03		Possible	21-34	11-13	15.5-18.5	21-33.5	44.5-46	34	Geopier/Piles	Yes
78	50.03		Possible- No	--	9.5-21.5	41.5-46.5 mixed	--	--	21.5	Geopier	Maybe
79	50.03		No	--	13.5-27 poss. peat	--		--	27	Geopier	--
80	50.03		No	--	7-15	--		--	15	Overexc/Geopier	--
81	50.03		No	--	--	--		--	--		--
82	30.51	34.2	Yes	--	6.5-11.5	14.5-15.5	--	--	15.5	Overexc/Geopier	Maybe

Turbine	Refusal (ft)	HSA Depth to Rock/ Refusal(ft)	Decreasing SPT (from CPT)	Weak Lacustrine Clay Depth (ft)	Significant Weak Zones < 1100 psf Below 9ft				Required Mitigation Depth (ft)	Anticipated Remediation Type	Karst Concern
83	50.02		Possible	--	4.5-12.5	--	--	--	12.5	Overexcavation	--
84	50.02		No	--	4.5-19.5	--	--	--	19.5	Geopier	--
85	41.72		No	--	7-11.5	--	--	--	11.5	Overexcavation	--
86	50.02		Possible	--	7-9.5	--	--	--	9.5	Overexcavation	--
87	50.02		No	--	14-18	--	--	--	18	Geopiers	--
88	35.38	>50	No	--	--	--	--	--	--	--	--
89	50.03		Possible	--	--	--	--	--	--	--	--
90	47.31		Possible	--	--	--	--	--	--	--	--
91	50.02		No	--	--	--	--	--	--	--	--
92	50.02		No	--	20.5-21.5	--	--	--	--	None*	--
93	48.46		No	--	--	--	--	--	--	--	--
94	50.02		No	--	--	--	--	--	--	--	--
95	50.03		No	--	--	--	--	--	--	--	--
96	43.47		No	--	10.5-11.5	--	--	--	11.5	Overexcavation	--
97	--		--	--	--	--	--	--	--	--	--
98	50.02		No	--	41.5-43	--	--	--	--	None*	Maybe
99	50.03	>50	No	--	--	--	--	--	--	--	--
100	--		--	--	--	--	--	--	--	--	--
101	--		--	--	--	--	--	--	--	--	--
102	50.03		No	--	--	--	--	--	--	--	--
103	--		--	--	--	--	--	--	--	--	--
104	--		--	--	--	--	--	--	--	--	--
105	--		--	--	--	--	--	--	--	--	--
106	--		--	--	--	--	--	--	--	--	--
107	48.13		No	--	13.5-14	--	--	--	--	None*	--
108	35.42		No	--	--	--	--	--	--	--	--
109	34.58		Possible- No	--	24.5-25.5	--	--	--	--	None*	--
110	33.28		No	--	10.5-11	--	--	--	11	Overexcavation	--
111	50.02		No	--	--	--	--	--	--	--	--
112	50.02		No	--	--	--	--	--	--	--	--
113	50.02		Possible- No	--	37-43.5	--	--	--	--	--	Maybe
114	--		--	--	--	--	--	--	--	--	--
115	--		--	--	--	--	--	--	--	--	--
116	50.02		Possible- No	--	29-29.5	--	--	--	--	--	--
117	19.57	>50	No	--	--	--	--	--	--	--	--
118	25.1		No	--	15-15.5	--	--	--	--	None*	--
119	50.03		No	--	--	--	--	--	--	--	--
120	50.03		No	--	--	--	--	--	--	--	--
121	--		--	--	--	--	--	--	--	--	--
122	50.03		No	11.5	17.5-23.5	--	--	--	23.5	Geopier	--
123	46.42		No	--	--	--	--	--	--	--	--
124	50.02		Possible- No	--	--	--	--	--	--	--	--
125	31.45		Possible-Yes	--	25-27.5 very weak	--	--	--	27.5	Geopier	Yes
126	34.1		No	--	--	--	--	--	--	--	--
127	48.92		No	--	8.5-11.5	21	--	--	11.5	Overexcavation	--
128	50.02		No	15.5	--	--	--	--	15.5	Geopier	--
129	29.17		Possible- No	--	9.5-10	--	--	--	10	Overexcavation	--
130	30.69		Yes	--	11.5-12.25	15.5-19	22.5-23.5	--	20	Geopier	Yes
131	50.03		No	38	--	--	--	--	38	Geopier/Piles	--
132	50.03		No	--	9-15	--	--	--	15	Overexc/Geopier	--
133	50.03		Possible	23	32.5-33.5	--	--	--	23	Geopier	--

*Suitable from multi-layer bearing capacity analysis - remediation not anticipated

Table 4
Summary of Average Compression and Shear Wave Velocity and Poisson's Ratio

Turbine	Vs (ft/s)	Vp (ft/s)	v
3	1189	5008	0.47
12	975	5881	0.49
23	1093	5423	0.48
33	1006	6568	0.49
38	1252	7007	0.48
41	635	10143	0.50
48	680	6071	0.49
63	1259	5948	0.48
68	1377	3598	0.41
82	872	6999	0.49
88	672	7069	0.50
95	1119	5257	0.48
97	1239	6569	0.48
118	1183	5674	0.48
120	1233	3273	0.42
129	1290	5095	0.47
Minimum	**635**	**3273**	**0.41**
Maximum	**1377**	**10143**	**0.50**
Average	**1067**	**5974**	**0.47**

Table 5
Estimated Long-term Settlement under Mean Operating Load Conditions

Turbine No.	5	45	48	51	54	64	70	78	81	89	92	107	131
Soil Survey Parent Materials	Lake/ Marsh	Marsh	Marsh	Marsh	Marsh	Moraine	Lake/ Marsh	Depression	Moraine	Moraine	Depression	Till/ Moraine	Lake/ Marsh
Tot. Settlement [in]	0.9	69.8	23.3	74.5	19.7	1.0	49.4	4.5	0.4	0.8	1.4	0.7	9.0
Diff Settlement [in]	0.4	36.6	12.4	37.4	10.3	0.5	25.3	2.0	0.2	0.3	0.6	0.3	4.7
Diff Settlement [in/ft]	0.0	1.2	0.4	1.2	0.3	0.0	0.8	0.1	0.0	0.0	0.0	0.0	0.2
Diff Settlement [mm/m]	1.1	101.8	34.5	104.0	28.5	1.4	70.3	5.5	0.5	0.9	1.8	1.0	13.1

Table 6
Summary of General Laboratory Test Results

Sample Location		Approx			Calc.	Atterberg Limits			Percent	Unconfined/UU	Consolidation Test Data				pH	Soluble	Chlorides
Boring No.	Depth	Soil Type (1)	Moisture Content (%)	Dry Density (pcf)	Bulk Dens. (pcf)	Liquid Limit (% moisture content)	Plastic Limit (% moisture content)	Plast. Index (% moisture content)	fines (%)	Compressive Strength (tsf)	Cc	Cr	e_o	Pc (tsf)		Sulfates (ppm)	(ppm)
T-02	1	marsh	24.0														
	3	marsh	19.1	109.3	130.2										7.7	9.4	37
	12	till	14.9	120.4	138.3					1.19							
	15	till	13.6			25	17	8									
	25	till	14.1														
	30	till	18.5						46								
	35	till	8.6														
	45	till	13.4														
T-14	1	marsh	21.8														
	5	marsh	25.3														
	17	till	14.2														
	20	till	10.3			22	17	5									
	25	till	8.4														
T-19	1	marsh	30.2														
	3	marsh	24.2	96.6	120.0										7.4	330	55
	10	till	12.7														
	20	till	5.2														
	25	till	14.4			32	17	15									
	30	till	18.0														
T-31	1	lacustrine	27.9														
	5	lacustrine	24.9														
T-35	1	till	25.8														
	3	till	20.8	106.7	128.9										7.7	<50	37
	5	till	15.3														
	12	till	13.5	124.3	141.1					1.3							
	15	till	14.7						58								
T-39	1	marsh	19.7														
	5	marsh	21.4														
	12	marsh	22.5	102.3	125.3					0.79							
	20	marsh	26.2			38	22	16									
	30	till	27.7														
T-49	3	lacustrine	36.5	76.5	104.4										7.6	100	66
	5	lacustrine	32.1														
	12	lacustrine	39.4	78.5	109.4					0.23	0.27	0.05	1.092	0.56			
	22	lacustrine	41.6	80.1	113.4	41.5	19.7	21.8		0.17							
	47	lacustrine	38.5	83	115.0					0.28							
	50	lacustrine	6.0														
T-50	1	lacustrine	27.7														
T-51	3	lacustrine	39.4	81.6	113.8					0.14					7.7	220	48
	20	lacustrine	40.9	77.6	109.3	42	19.3	22.7		0.15	0.26	0.05	1.161	0.42			
	30	lacustrine	39.0														
	37	lacustrine	44.1	72.2	104.0					0.17							
	50	lacustrine	45.3														
T-62	3	marsh	36.9	73.6	100.8										7.6	690	<10
	15	till	12.1														
	20	till	11.5			22	17	5									
	22	till	17.5	113.7	133.6					0.73							
	30	till	19.4														
	35	till	15.2						88								

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Table 6
Summary of General Laboratory Test Results

Sample Location Boring No.	Depth	Approx Soil Type (1)	Moisture Content (%)	Dry Density (pcf)	Calc. Bulk Dens. (pcf)	Atterberg Limits (% moisture content) Liquid Limit	Plastic Limit	Plast. Index	Percent fines (%)	Unconfined/UU Compressive Strength (tsf)	Consolidation Test Data Cc	Cr	e_o	Pc (tsf)	pH	Soluble Sulfates (ppm)	Chlorides (ppm)
	40	till	10.8														
	44	till	13.3														
T-66	1	lacustrine	76.4														
	3	lacustrine	41.3	78.7	111.2										7.6	3500	52
	12	lacustrine	36.7	82.2	112.4	38.4	20.1	18.3		0.29	0.24	0.04	1.020	0.59			
	25	lacustrine	51.9														
	35	lacustrine	43.4														
	40	lacustrine	26.8						99								
	43	lacustrine	23.9	101.6	125.9					0.82							
T-72	1	lacustrine	30.6														
	3	lacustrine	32.4	89.2	118.1										7.5	2000	50
	12	lacustrine	23.1	103.1	126.9					1.6							
T-82	1	lacustrine	23.1														
	3	lacustrine	20.8	105.7	127.7										7.4	250	57
	5	lacustrine	24.4														
	20	lacustrine	16.9														
	25	lacustrine	11.7			17	11	6									
	26	lacustrine	13.5	124	140.7	36	18	18		1.25							
T-88	1	lacustrine	24.8														
	3	lacustrine	31.4	87.2	114.6										7.2	55	53
	10	lacustrine	21.4														
	15	lacustrine	22.0						99								
	17	lacustrine	25.0	100.7	125.9					1.52							
	20	lacustrine	24.1			36	19	17									
	27	till	18.8	112.2	133.3	18.9	13.5	5.4		1.71							
	40	till	13.5														
	50	till	19.6														
T-99	3	till	16.6	112.2	130.8										7.2	59	49
T-117	1	laustrine	23.7														
	3	laustrine	23.2	99.4	122.5										6	<50	53
	10	till	23.4														
	15	till	13.0			28	18	10									
	25	till	14.4						70								
	30	till	15.8														
	40	till	29.7														
	50	till	10.7														
SS-2	3	depression	18.6	113.6	134.7												
Number of Tests			86	28	28	13	13	13	6	16	3	3	3	3	12	12	12
Minimum Values			5.2	72.2	100.8	17.0	11.0	5.0	46.0	0.14	0.24	0.04	1.02	0.42	6.0	9.4	37.0
Maximum Values			76.4	124.3	141.1	42.0	22.0	22.7	99.0	1.71	0.27	0.05	1.16	0.59	7.7	3500.0	66.0
Average Values			23.5	96.7	121.9	30.5	17.6	13.0	76.7	0.77	0.26	0.05	1.09	0.52	7.4	721.3	50.6
Standard Deviations			11.7	16.3	11.5	8.8	2.8	6.6	22.2	0.6	0.0	0.0	0.1	0.1	0.5	1143.1	8.3

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

Table 7
Summary of Standard Proctor and California Bearing Ratio Test Results

Turbine ID / Thermal Test Location	Depth [ft]	Standard Proctor Data		95% Compaction Moist Unit Wt. at Opt. Moisture (pcf)	In-situ Moisture (%)	95% Compaction Moist Unit Wt. at in-situ Moisture (pcf)	CBR Results
		Maximum Dry Density (pcf)	Optimum Moisture (%)				95% Compaction
T-2	1-4	102.2	22.5	118.9	24.0	120.4	1.6
T-14	1-4	109.2	16.8	121.2	21.8	126.4	2.7
T-19	1-4	95.5	25.8	114.1	30.2	118.1	1.1
T-31	1-4	96.9	25.1	115.2	27.9	117.7	0.8
T-35	1-4	102	21.3	117.5	25.8	121.9	1.8
T-39	1-4	108.9	17.9	122.0	19.7	123.8	3.2
T-50	1-4	103	21.4	118.8	27.7	125.0	2.1
T-66**	1-4	67.5	47.5	94.6	76.4	113.1	1.1
T-72	1-4	98.7	21	113.5	30.6	122.5	2.0
T-82	1-4	108.8	18.2	122.2	23.1	127.2	1.7
T-88	1-4	105.1	20.1	119.9	24.8	124.6	1.6
T-117	1-4	103.3	20.2	118.0	23.7	121.4	2.1
Mean		100.1	23.2	116.3		121.8	1.8
St. Dev.		11.2	8.1	7.4		4.0	0.7
Min.		67.5	16.8	94.6		113.1	0.8
Max		109.2	47.5	122.2		127.2	3.2

** Organic Clay soil

Table 8
Subgrade Compaction and Aggregate Thickness

1.5 Max Rut Depth		Compaction (% of Standard Proctor Max Dry Density) 95 or Greater	
Traffic Condition		Maint. Traffic	Const. Traffic
Unreinforced		12	32
Reinforced	Geotextile	8	22
	Type I**	7	18
	Type II***	6	13

3.0 Max Rut Depth		Compaction (% of Standard Proctor Max Dry Density) 95 or Greater	
Traffic Condition		Maint. Traffic	Const. Traffic
Unreinforced		8	21
Reinforced	Geotextile	6	12
	Type I**	6	9
	Type II***	6	8

*Note that a minimum of 6 to 12 inches of aggregate base is recommended for road design to compensate for topsoil stripping.
** Type I: Triaxial geogrid TX 140
*** Type II: Triaxial geogrid TX 160

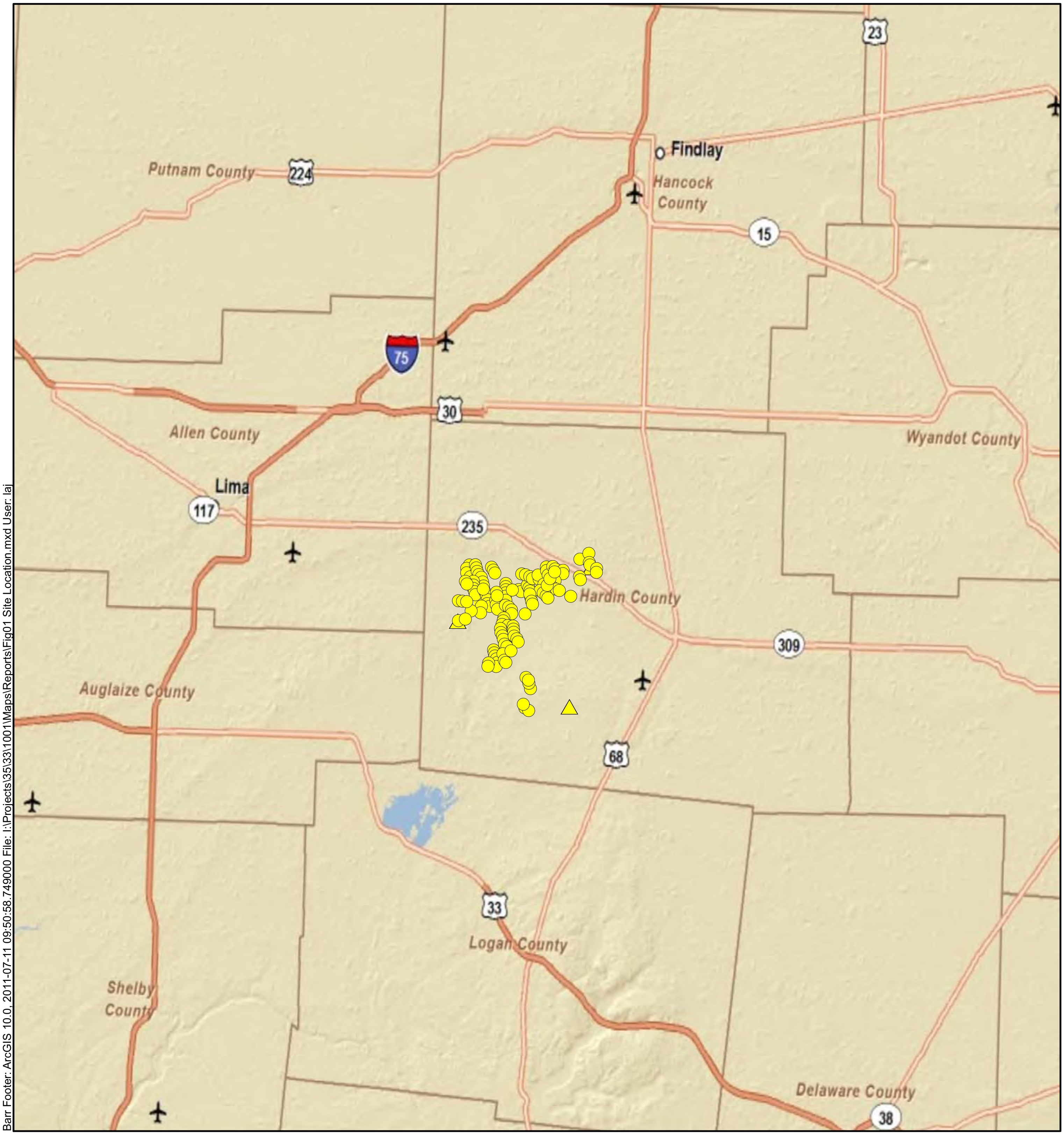
Assumed Traffic Loading Conditions

Construction		Maintenance	
Axle Load [kips]	25	Axle Load [kips]	3.5
Tire Pressure [psi]	80	Tire Pressure [psi]	65
Axle Passes [each]	800	Axle Passes [each]	2000
Max Rut Depth [in]	1.5, 3	Max Rut Depth [in]	1.5, 3

Table 9
Summary of Geotechnical Parameters for Foundation Design

Parameter	Value	Units
Undrained Soil Shear Strength (cohesive soil)	1,100	lb/ft^2
Min. Allowable Bearing Capacity, Normal Operating Load	2,300	lb/ft^2
Min. Allowable Bearing Capacity, Extreme Load	3,000	lb/ft^2
Min. Average Shear Wave Velocity	635	ft/s
Min. Design Small Strain Shear Modulus	1,500	$kips/ft^2$
Poisson Ratio	0.41	unitless
Min. Foundation/Soil Friction Factor	0.68	unitless
Backfill Density over Foundation (dry density = 95 pcf @ moisture content = 15 %)	100	lb/ft^3
Frost Depth	35	Inches

Figures



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

N

Miles

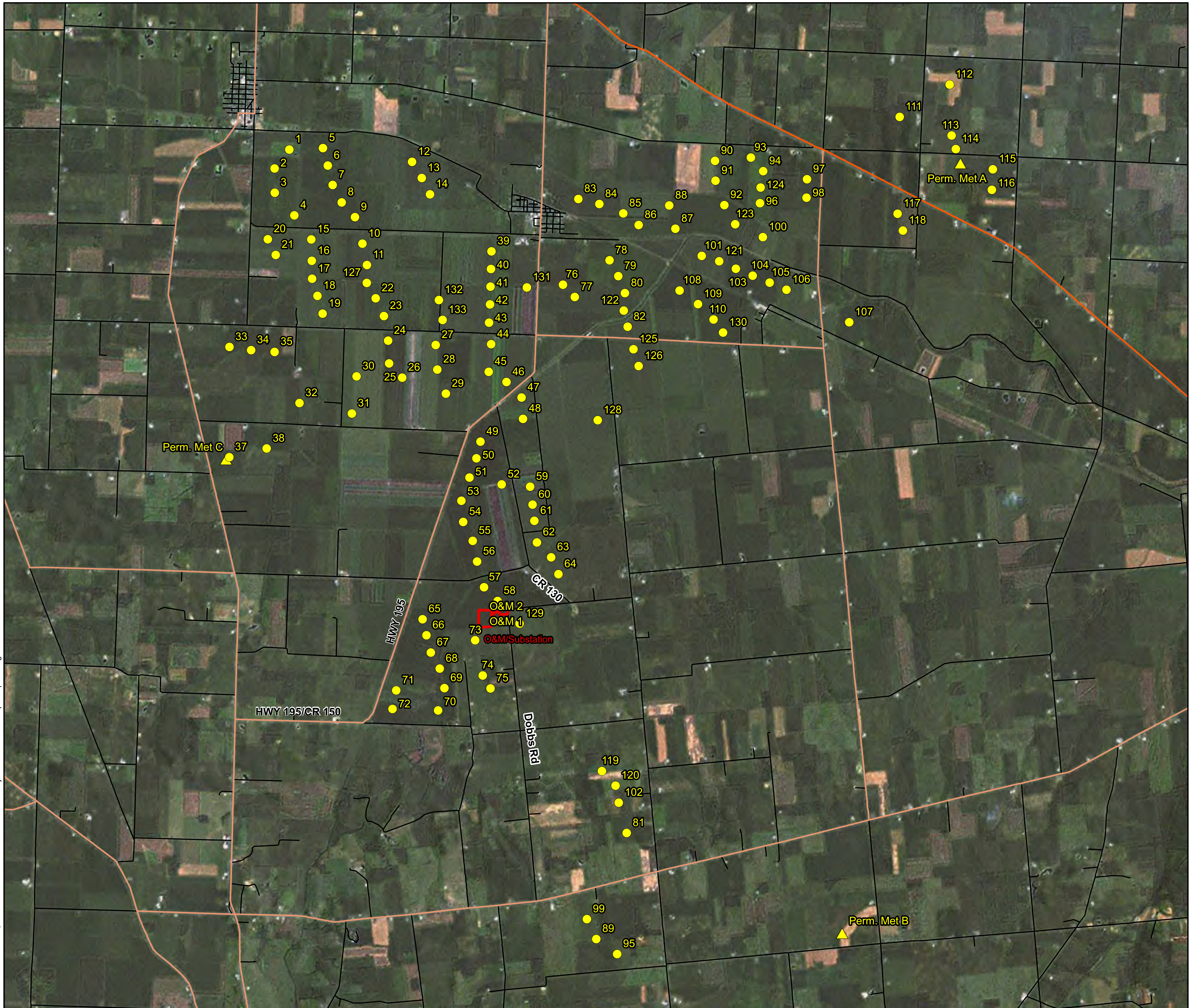
8 0 8

Kilometers

10 0 10

Figure 1

SITE LOCATION
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

Switchyard Boundary (Approximate)

Kilometers

1 0 1 2

Miles

1 0 1

Figure 2

SITE LAYOUT
Hardin County Wind Project
Inenergy
Hardin County, Ohio

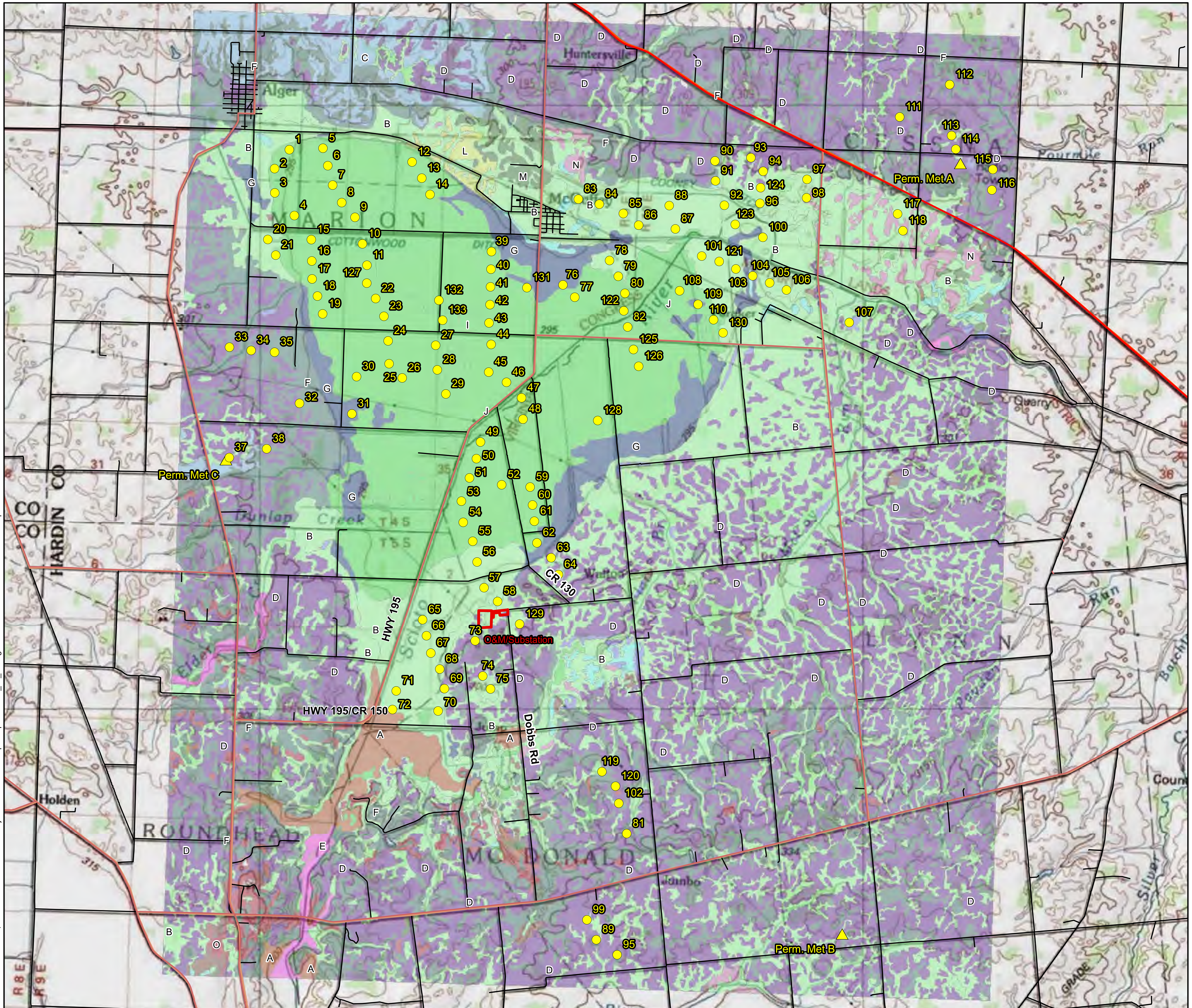


Figure 3

PARENT MATERIAL OF SURFACE SOIL
Hardin County Wind Project
Inenvergy
Hardin County, Ohio

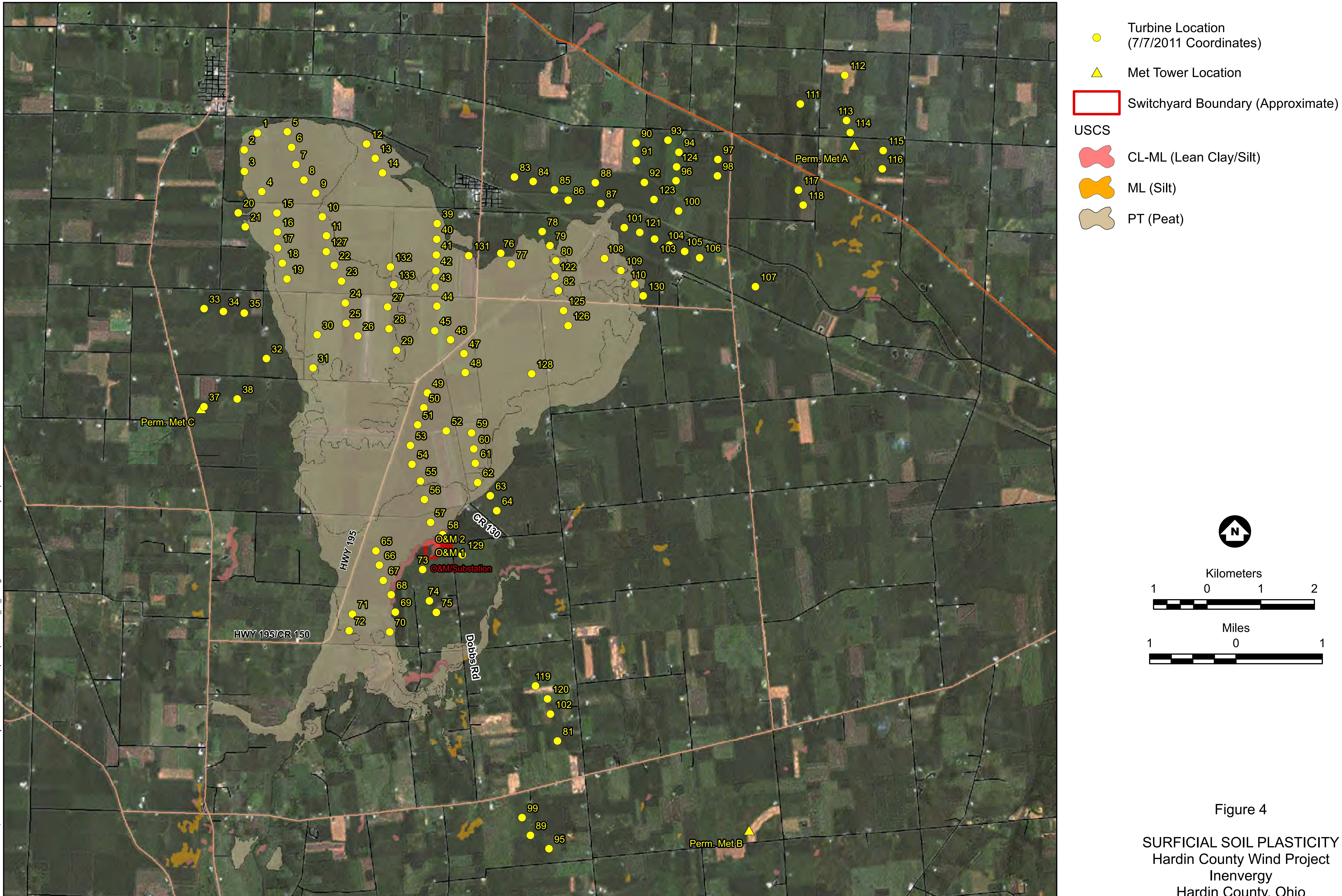
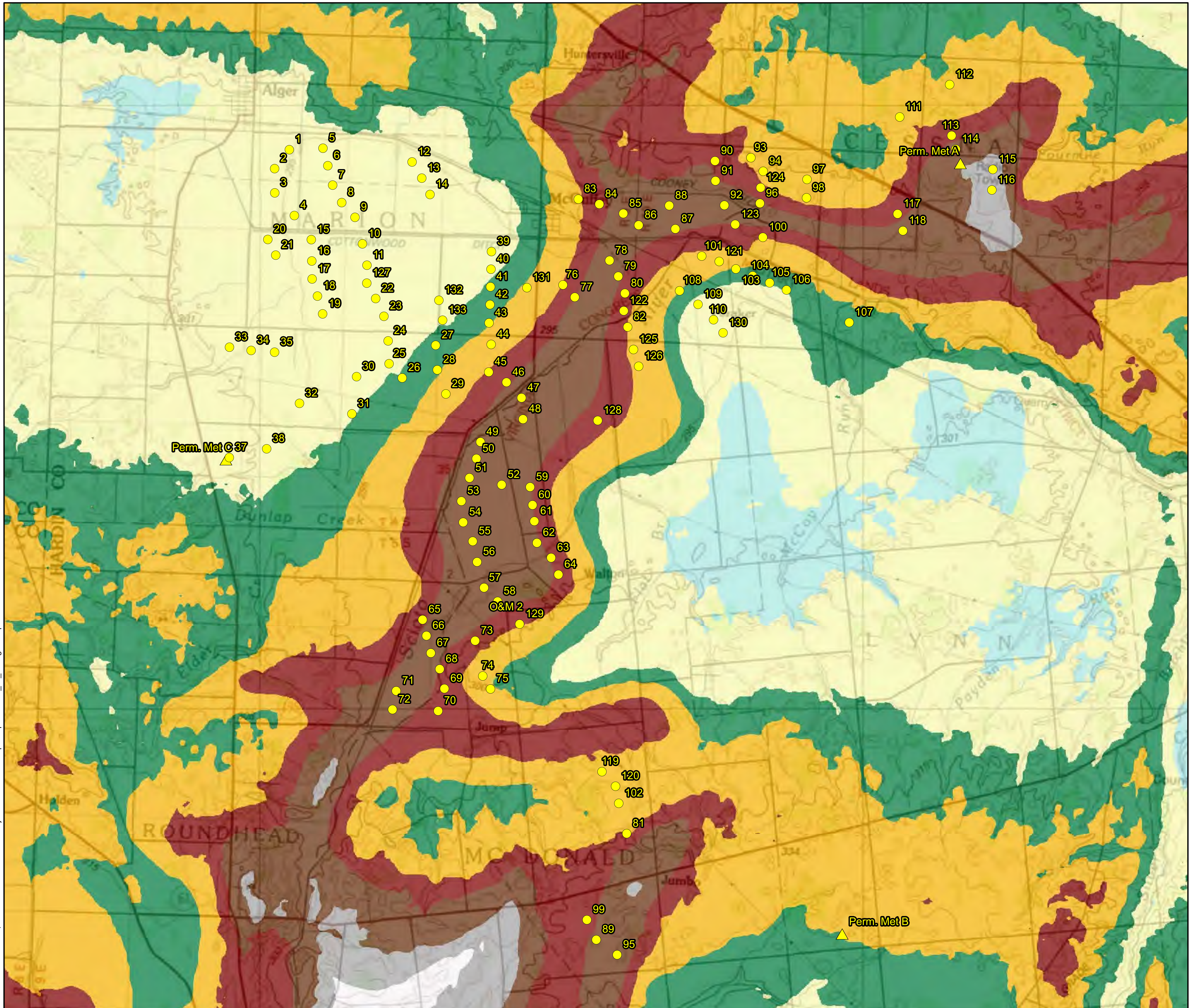


Figure 4

SURFICIAL SOIL PLASTICITY
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

Depth to Bedrock

- 0 - 20
- 21 - 50
- 51 - 80
- 81 - 120
- 121 - 160
- 161 - 210
- 211 - 260
- 261 - 330

N

Kilometers

1 0 1 2

Miles

1 0 1

Figure 5

DEPTH TO BEDROCK
Hardin County Wind Project
Inenvergy
Hardin County, Ohio

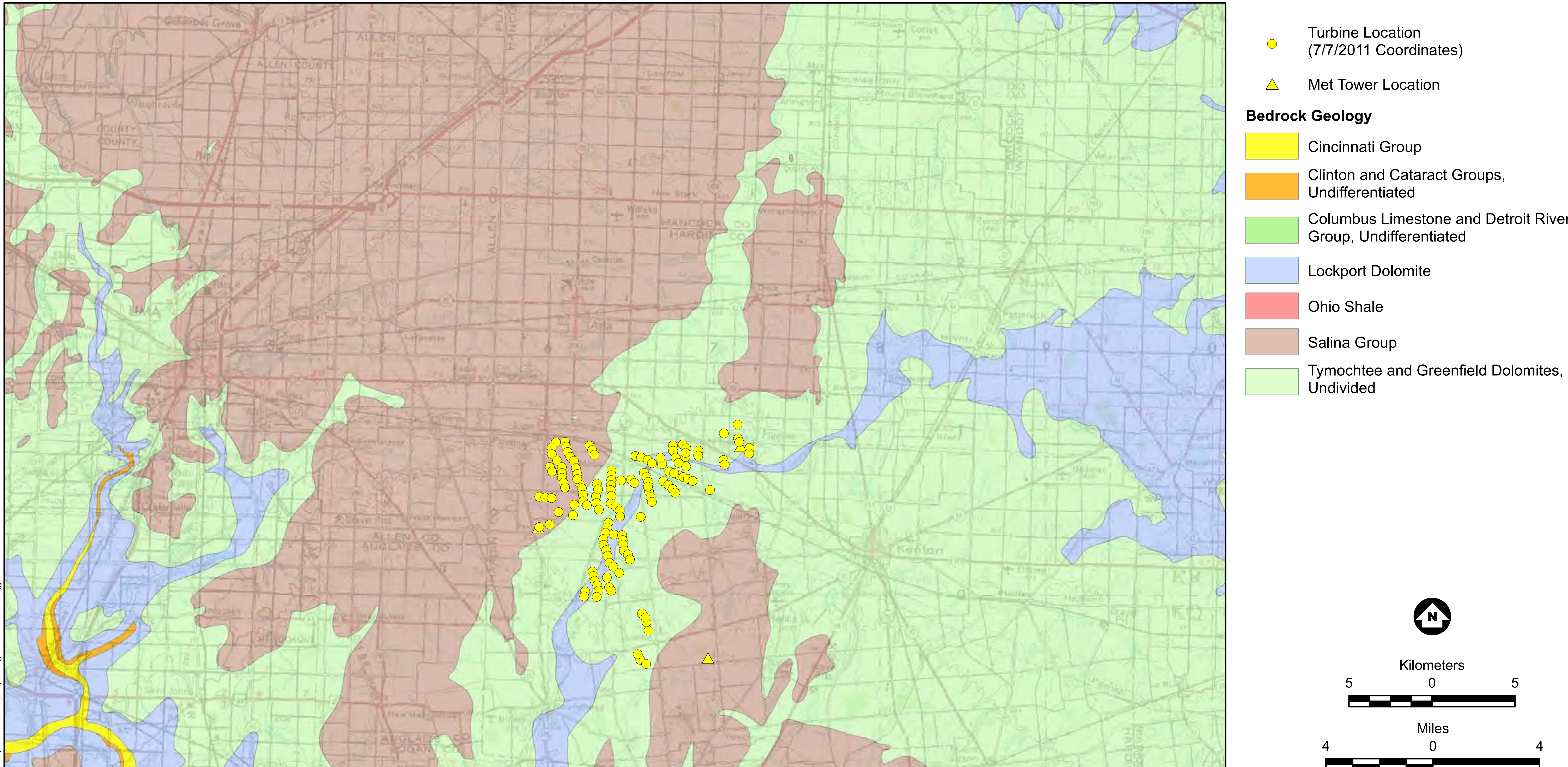
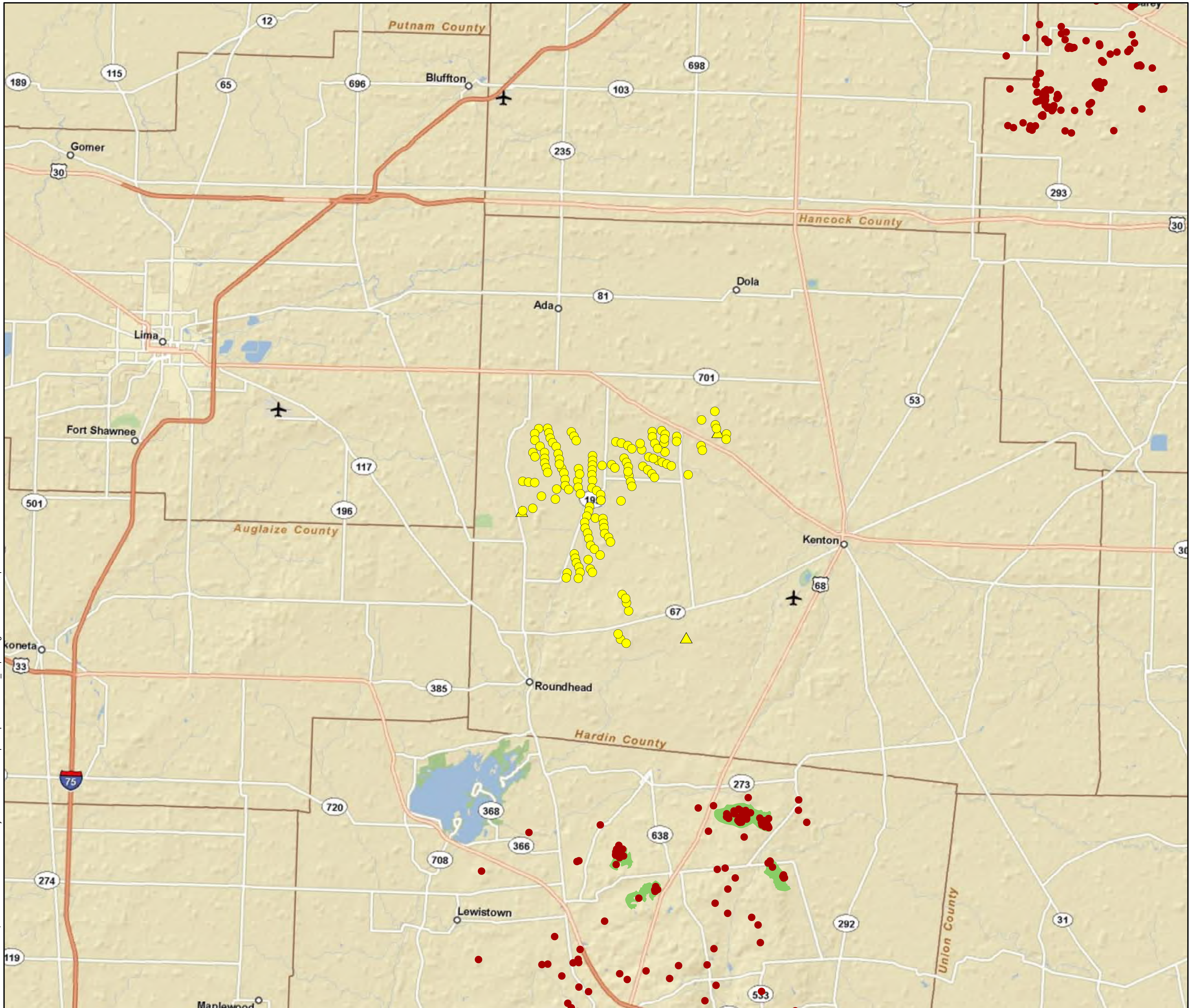


Figure 6

BEDROCK GEOLOGY
Hardin County Wind Project
Inenergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

Known Karsts Location

Probable Karst Areas

N

Kilometers

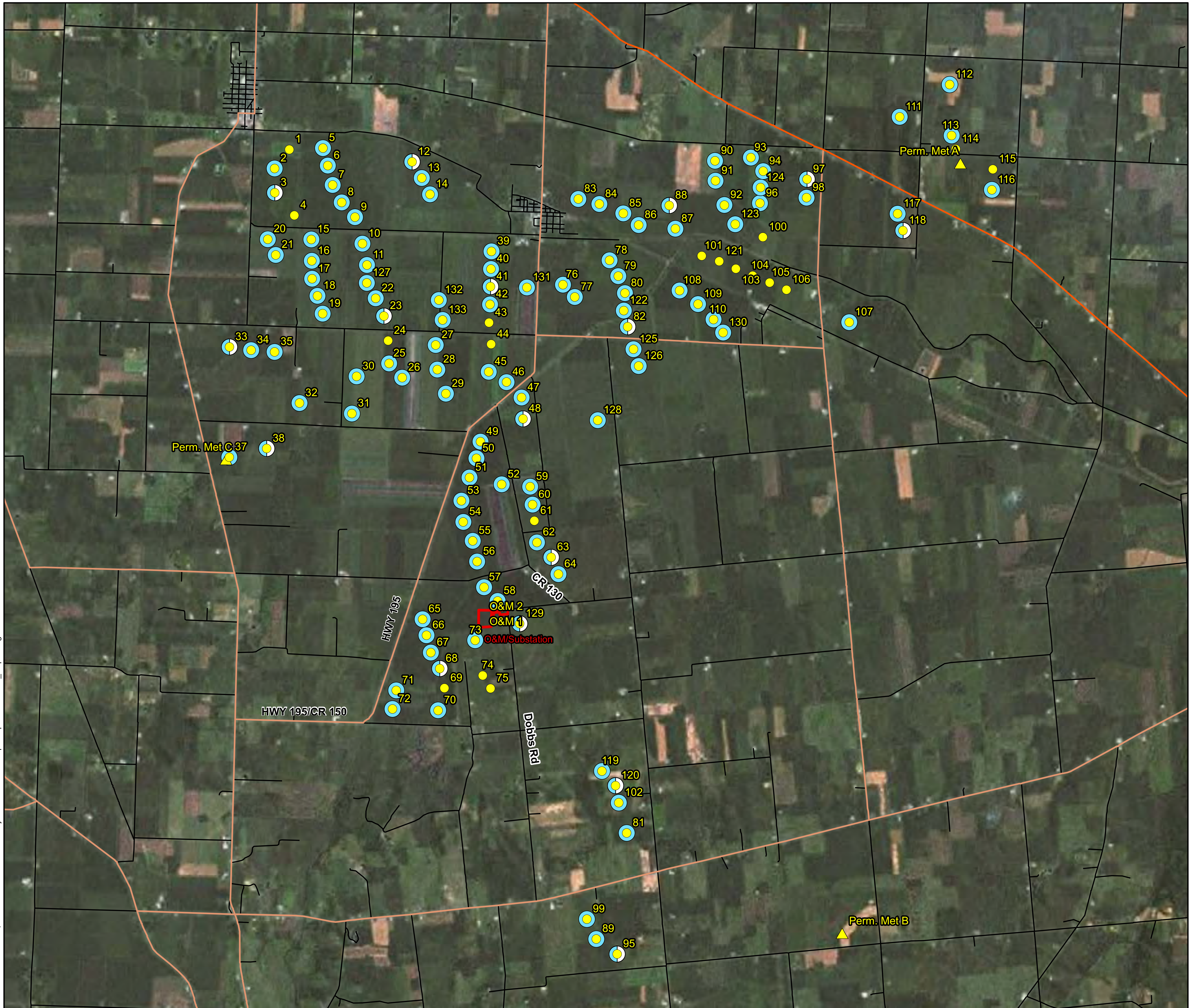
5 0 5

Miles

4 0 4

Figure 7

OHIO KARSTS
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

CPT Location

CPT & Seismic Location

Switchyard Boundary (Approximate)

Kilometers

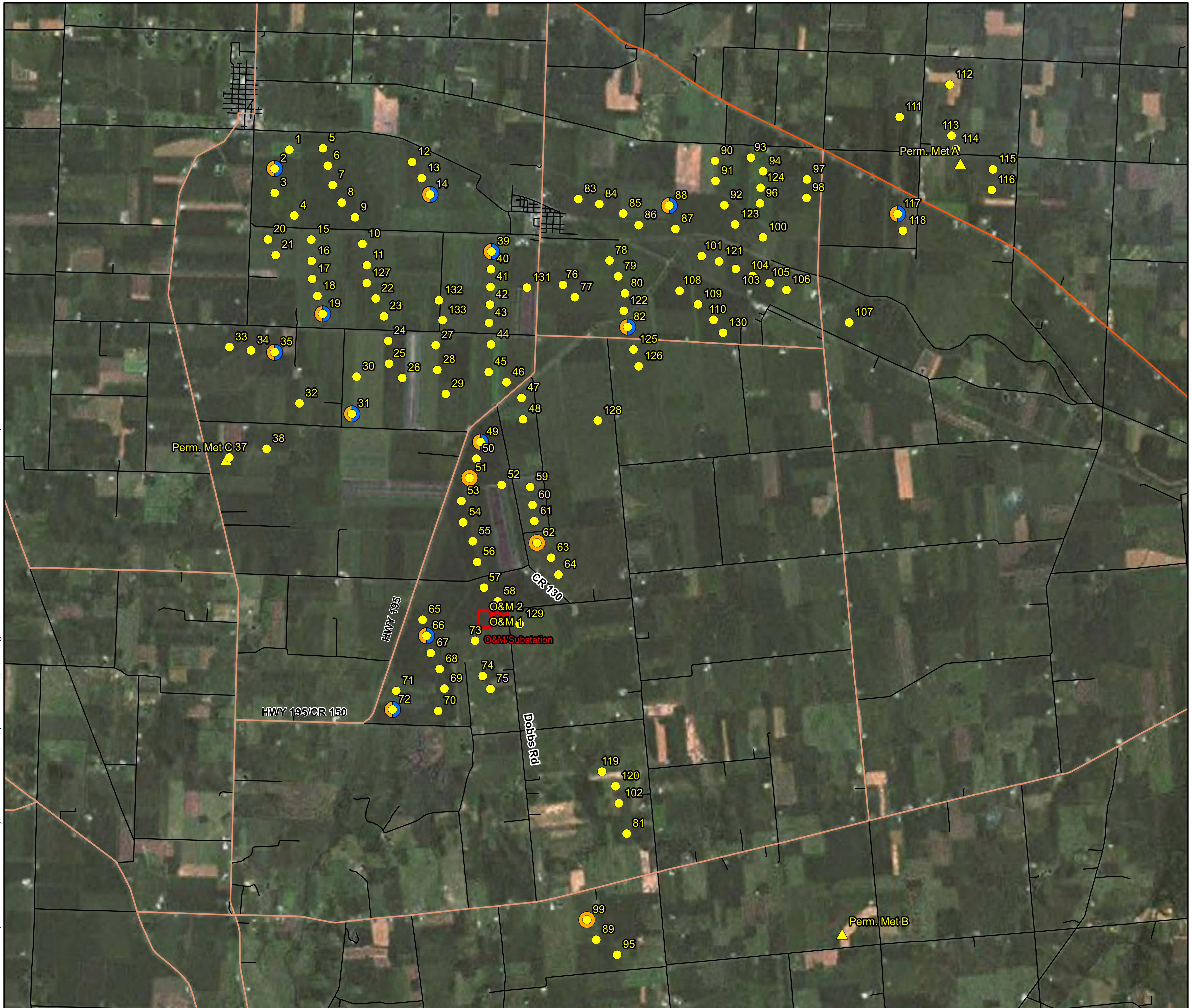
1 0 1 2

Miles

1 0 1

Figure 8

CPT AND SEISMIC TEST LOCATIONS
Hardin County Wind Project
Inenergy
Hardin County, Ohio



Turbine Location
(7/7/2011 Coordinates)

Met Tower Location

Borehole Test Location

Borehole Test & Piezometer Location

Switchyard Boundary (Approximate)

N

Kilometers

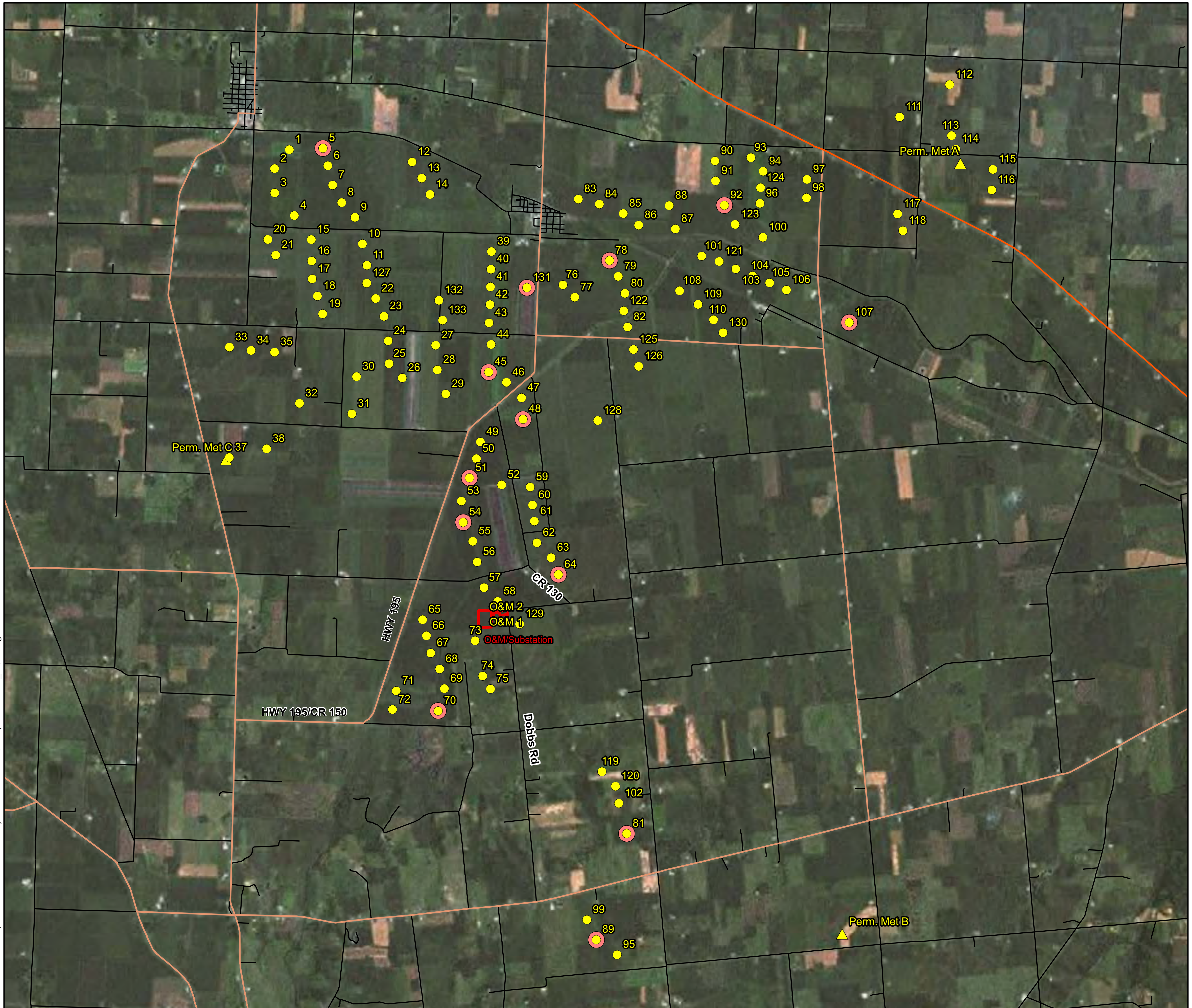
1 0 1 2

Miles

1 0 1

Figure 9

BOREHOLE TEST AND
PIEZOMETER LOCATIONS
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

DMT Location

Switchyard Boundary (Approximate)

Kilometers

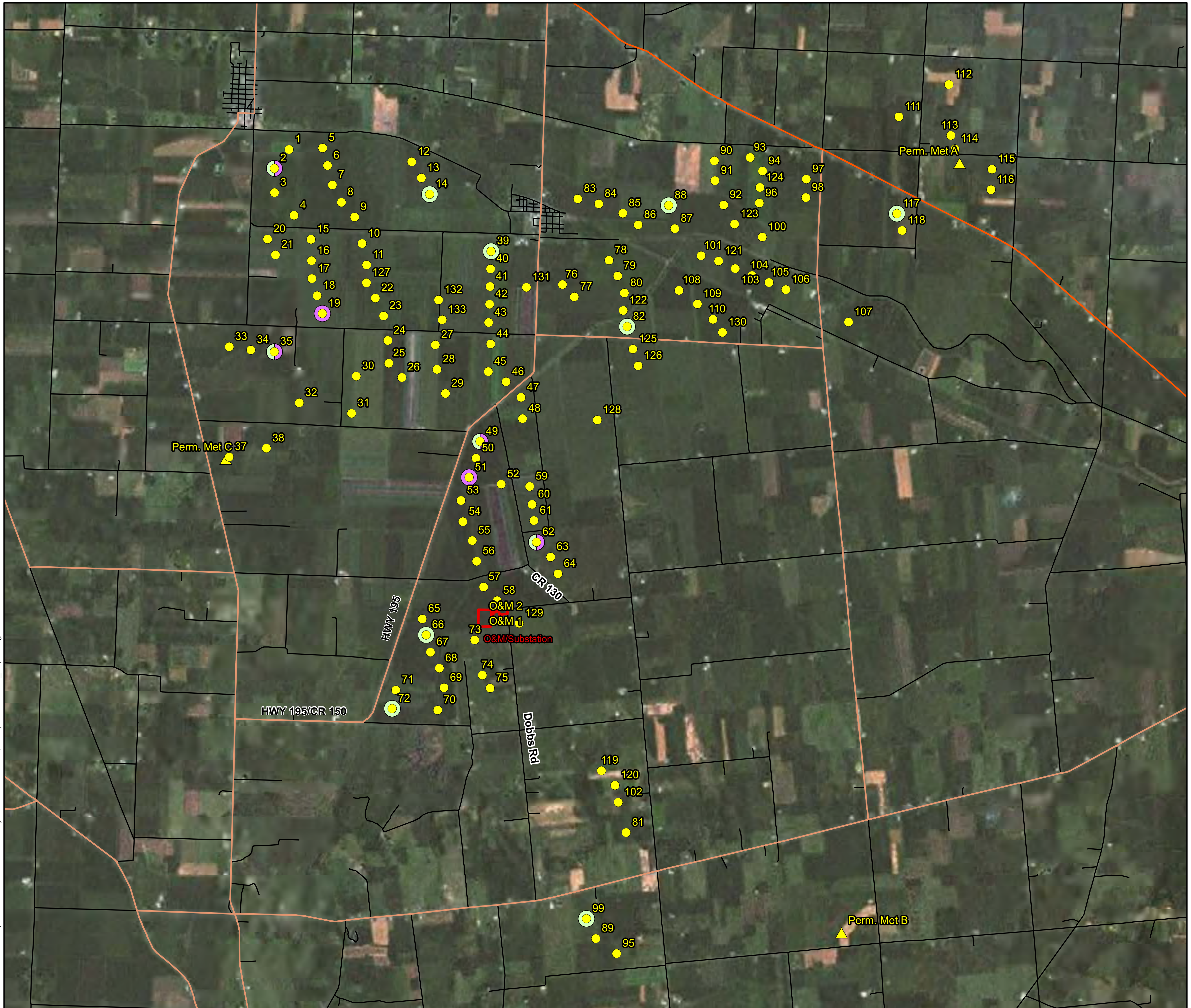
1 0 1 2

Miles

1 0 1

Figure 10

DMT TEST LOCATIONS
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



Turbine Location (7/7/2011 Coordinates)

Met Tower Location

Electrical Location

Thermal Location

Electrical & Thermal Location

Switchyard Boundary (Approximate)

N

Kilometers

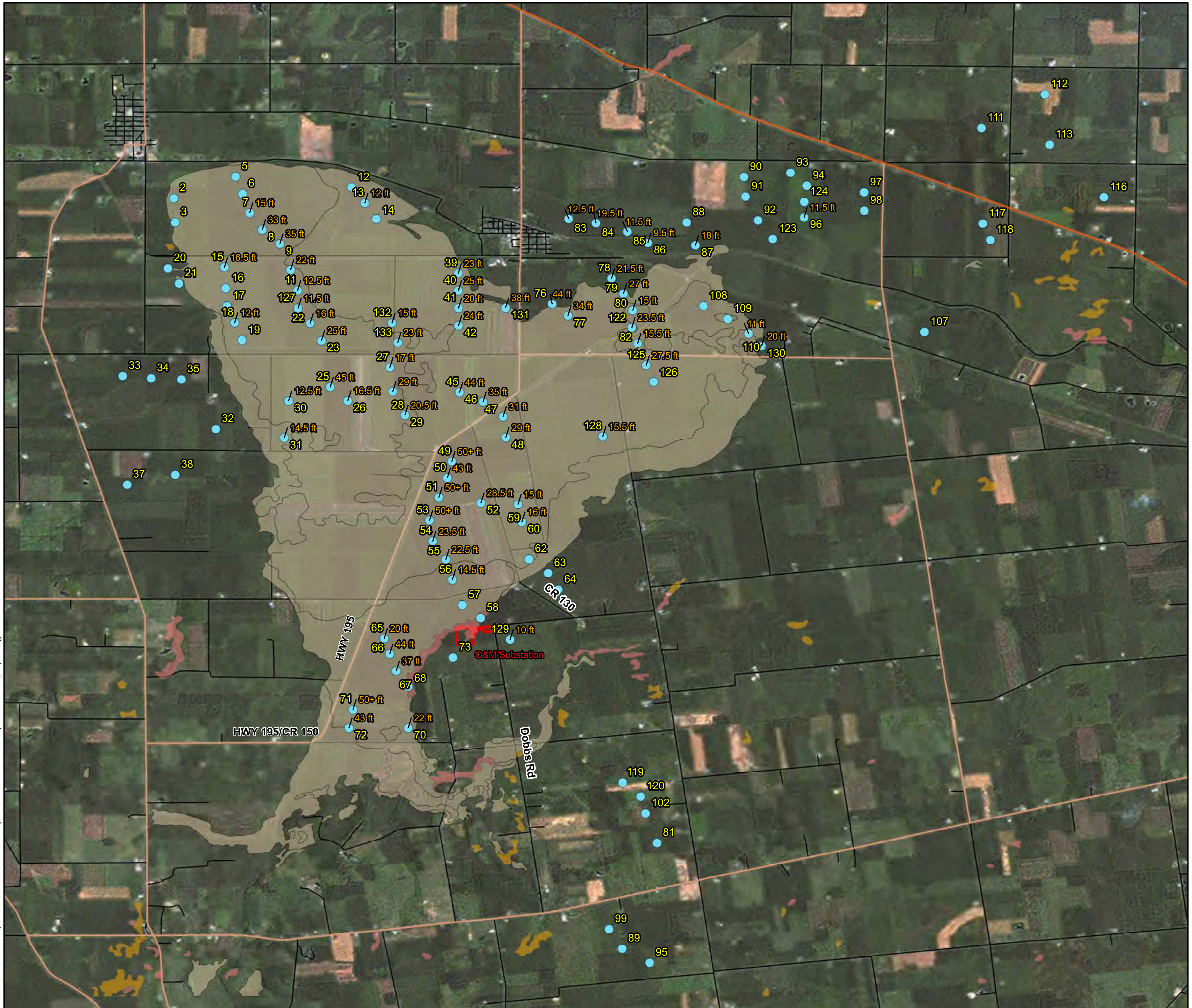
1 0 1 2

Miles

1 0 1

Figure 11

ELECTRICAL AND THERMAL TEST LOCATIONS
Hardin County Wind Project
Inenvergy
Hardin County, Ohio



CPT Test Locations

15 Required Mitigation Depth (Feet)

Switchyard Boundary (Approximate)

USCS

CL-ML (Lean Clay/Silt)

ML (silt)

PT (peat)

Kilometers

1 0 1 2

Miles

1 0 1

Figure 12

RECOMMENDED SOIL REMEDIATION SITES
Hardin County Wind Project
Inenergy
Hardin County, Ohio

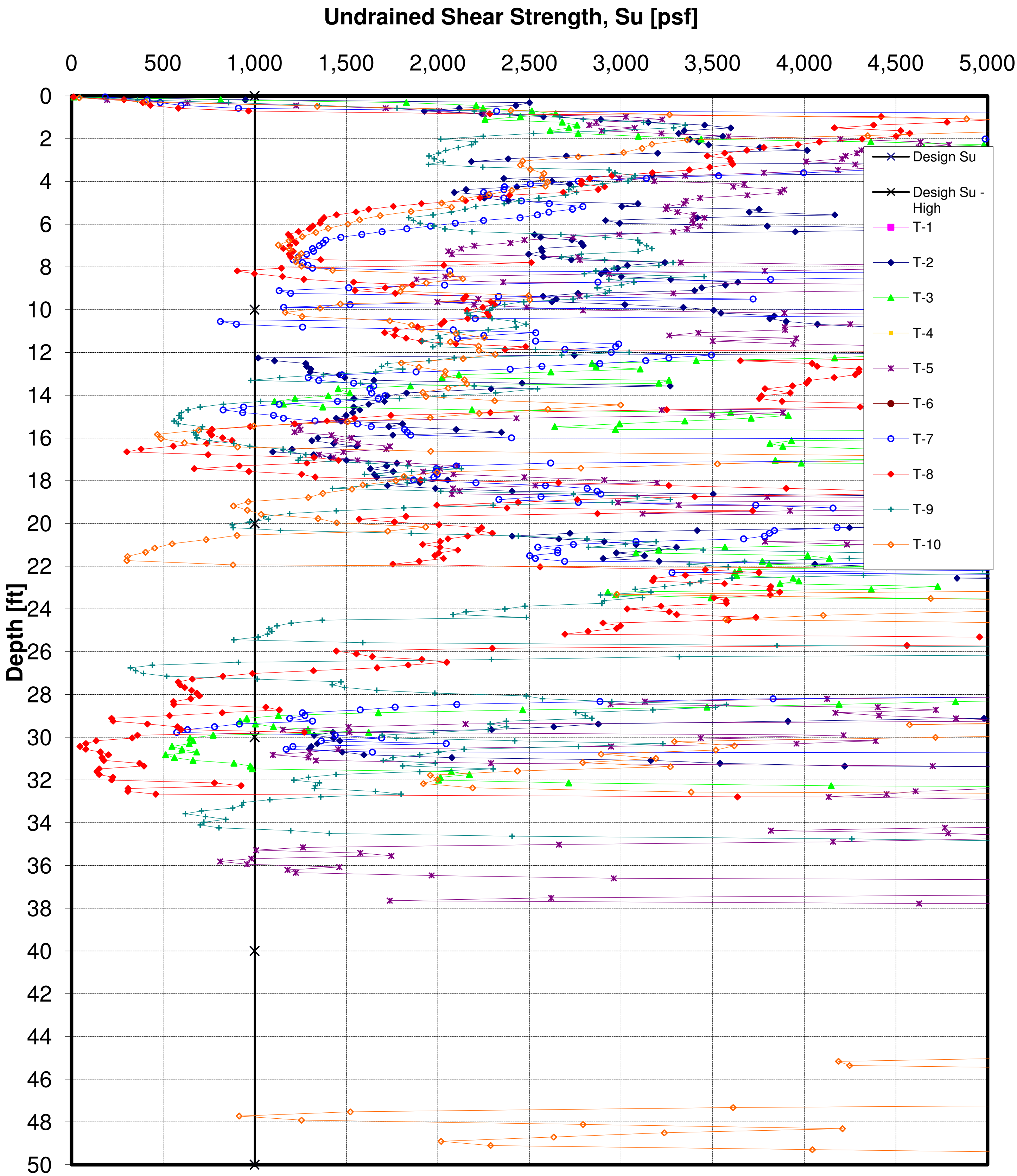


Figure 13. Undrained Shear Strength from CPT vs. Depth Turbines 1-10

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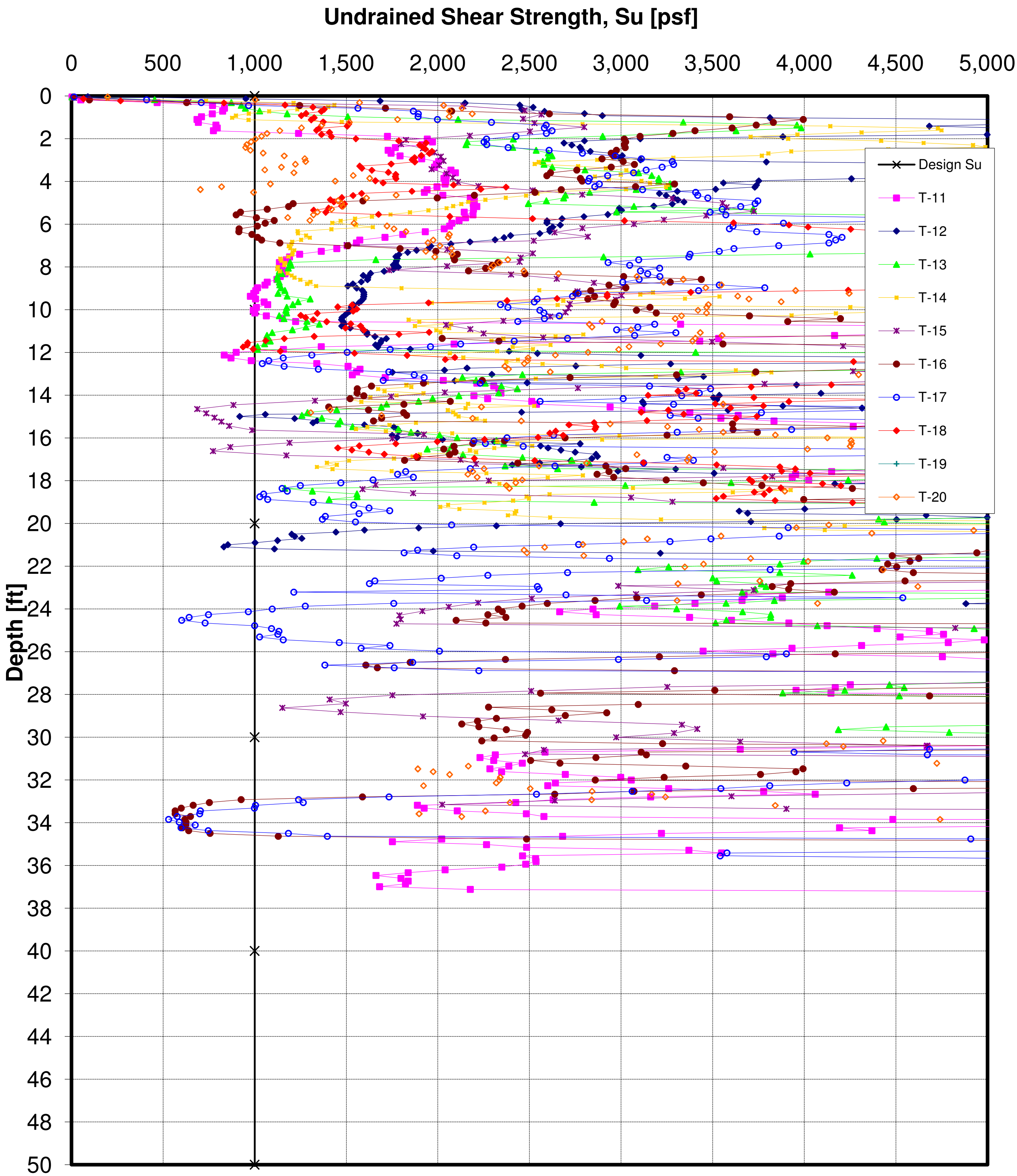

Figure 14. Undrained Shear Strength from CPT vs. Depth Turbines 11-20

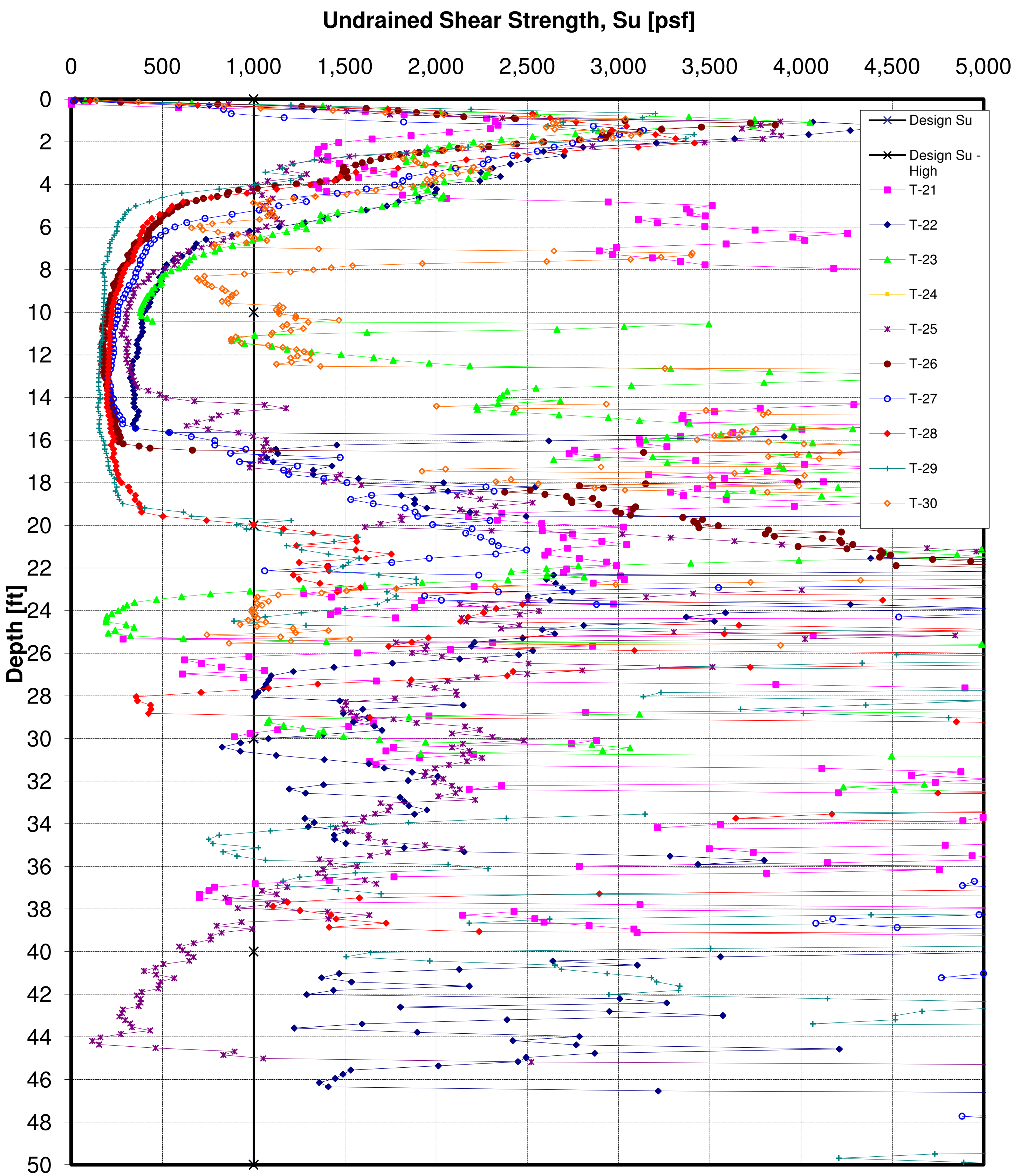

Figure 15. Undrained Shear Strength from CPT vs. Depth Turbines 21-30

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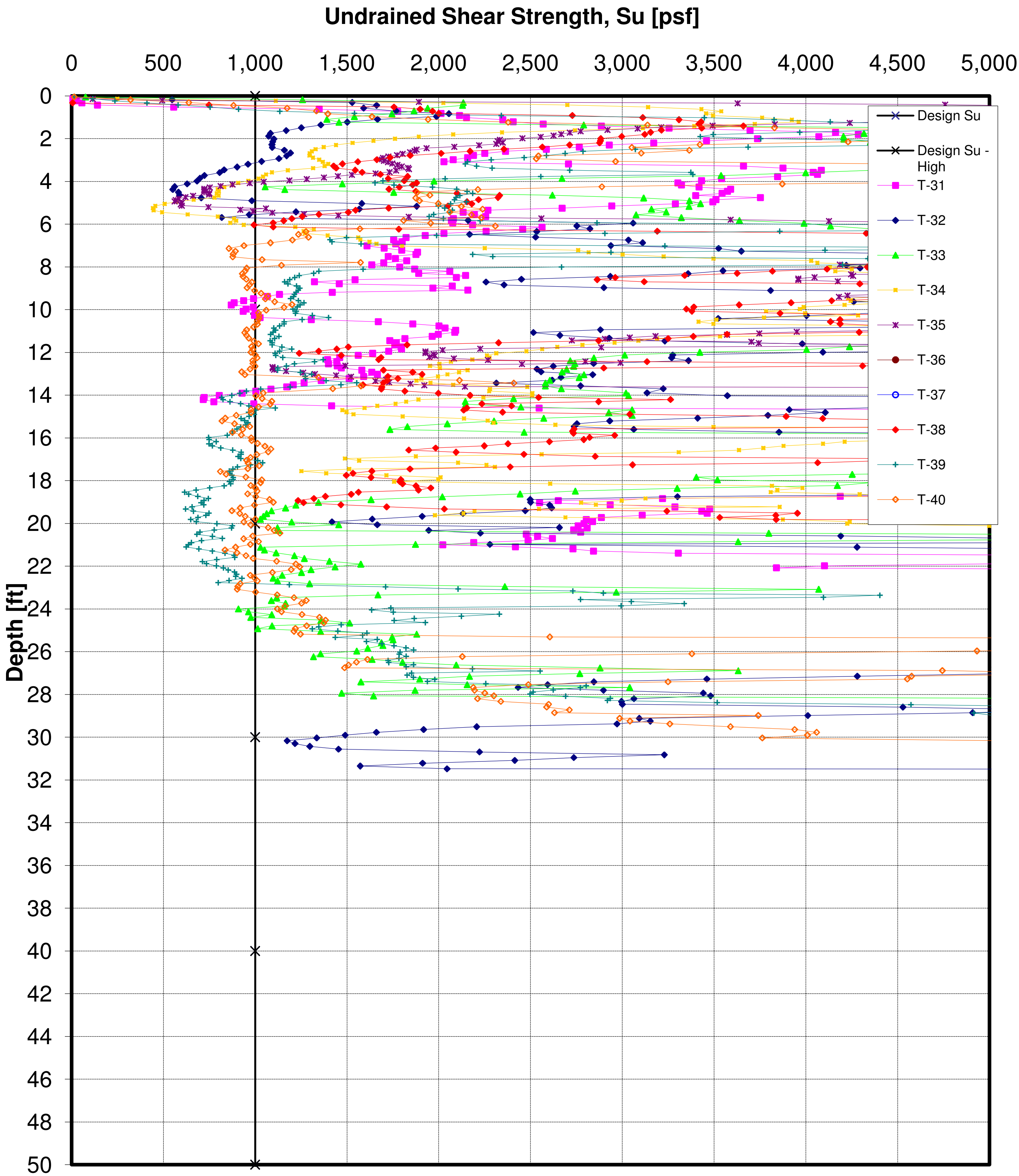

Figure 16. Undrained Shear Strength from CPT vs. Depth Turbines 31-40

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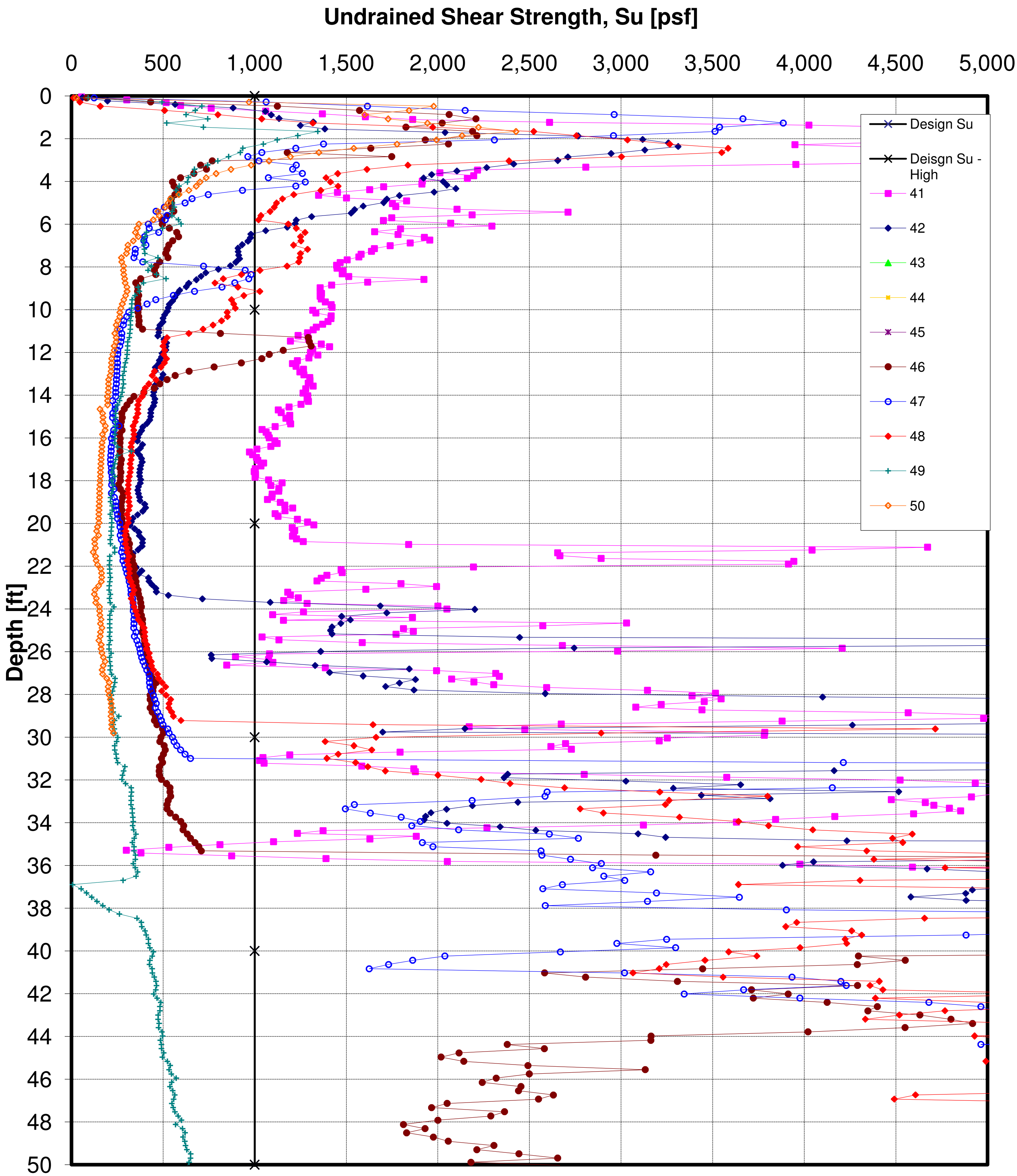

Figure 17. Undrained Shear Strength from CPT vs. Depth Turbines 41-50

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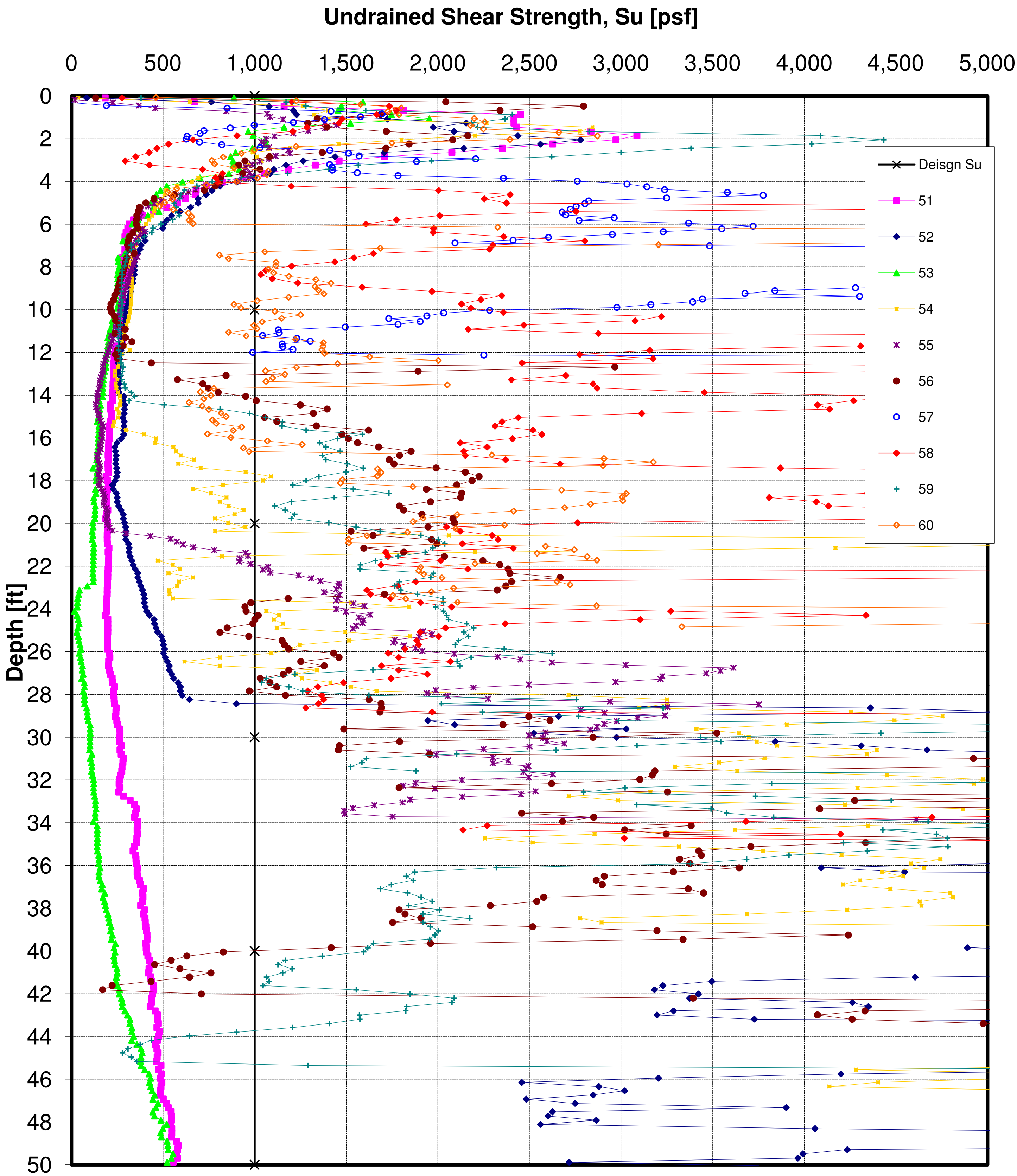

Figure 18. Undrained Shear Strength from CPT vs. Depth Turbines 51-60

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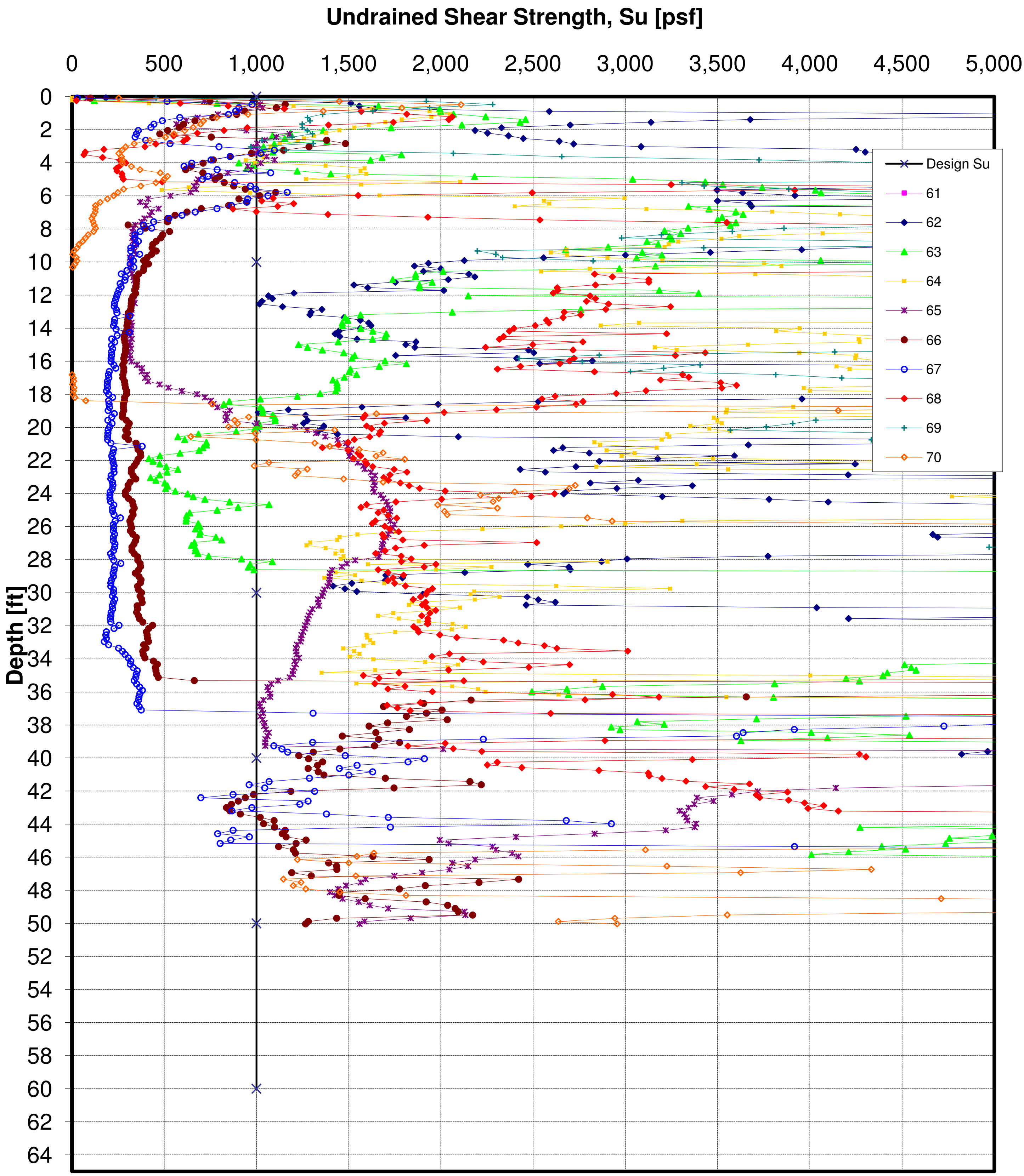

Figure 19. Undrained Shear Strength from CPT vs. Depth Turbines 61-70

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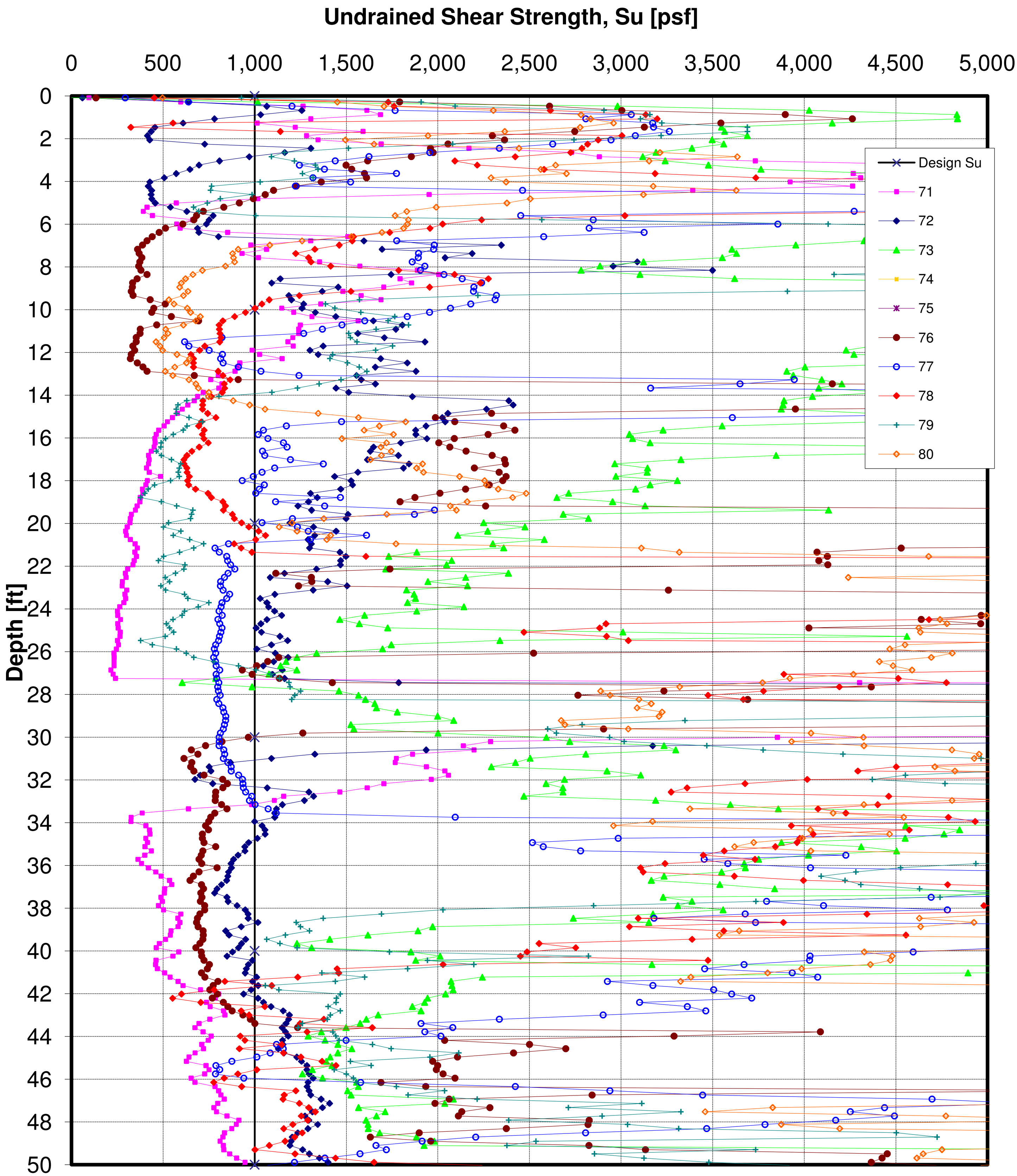

Figure 20. Undrained Shear Strength from CPT vs. Depth Turbines 71-80

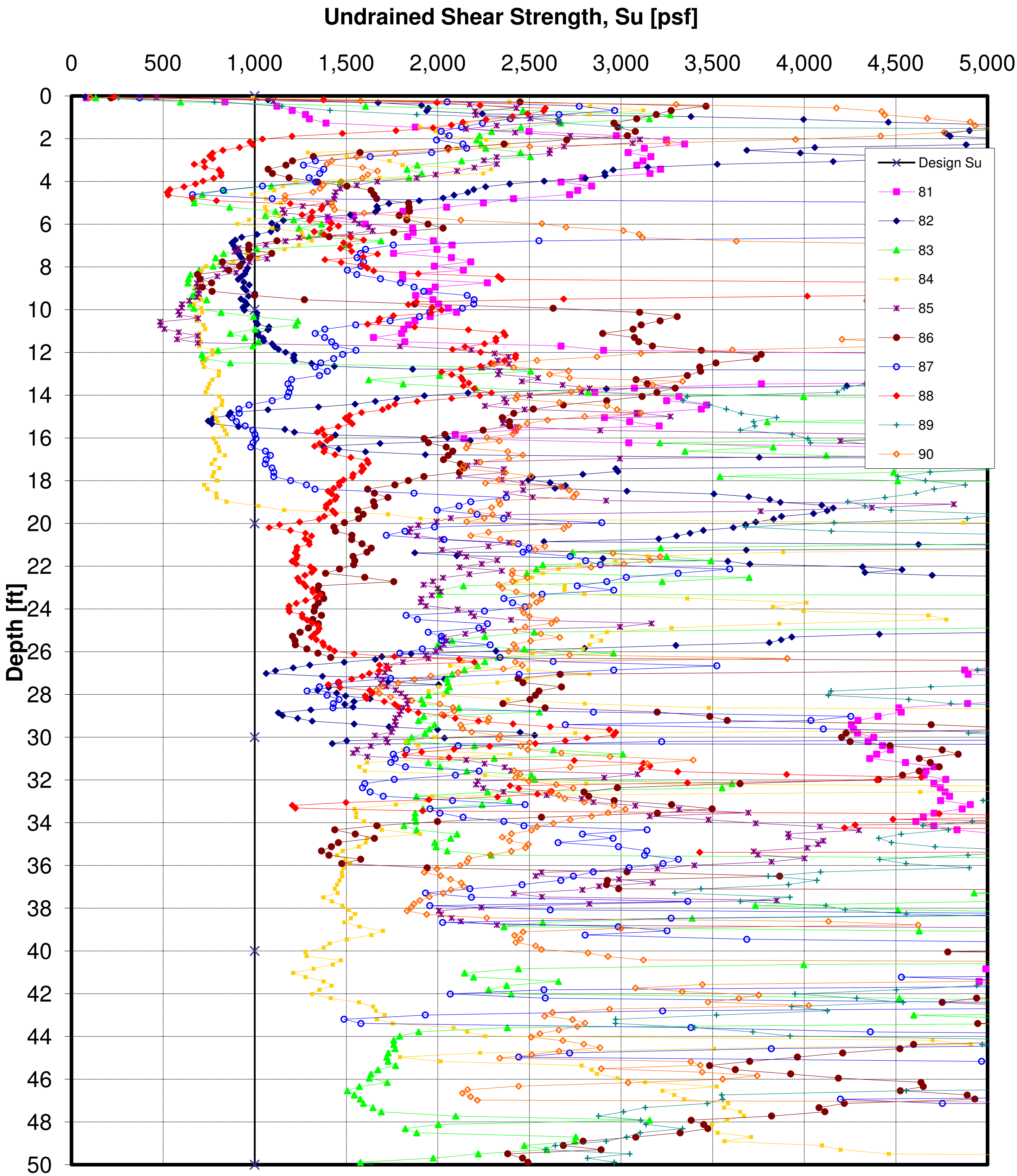

Figure 21. Undrained Shear Strength from CPT vs. Depth Turbines 81-90

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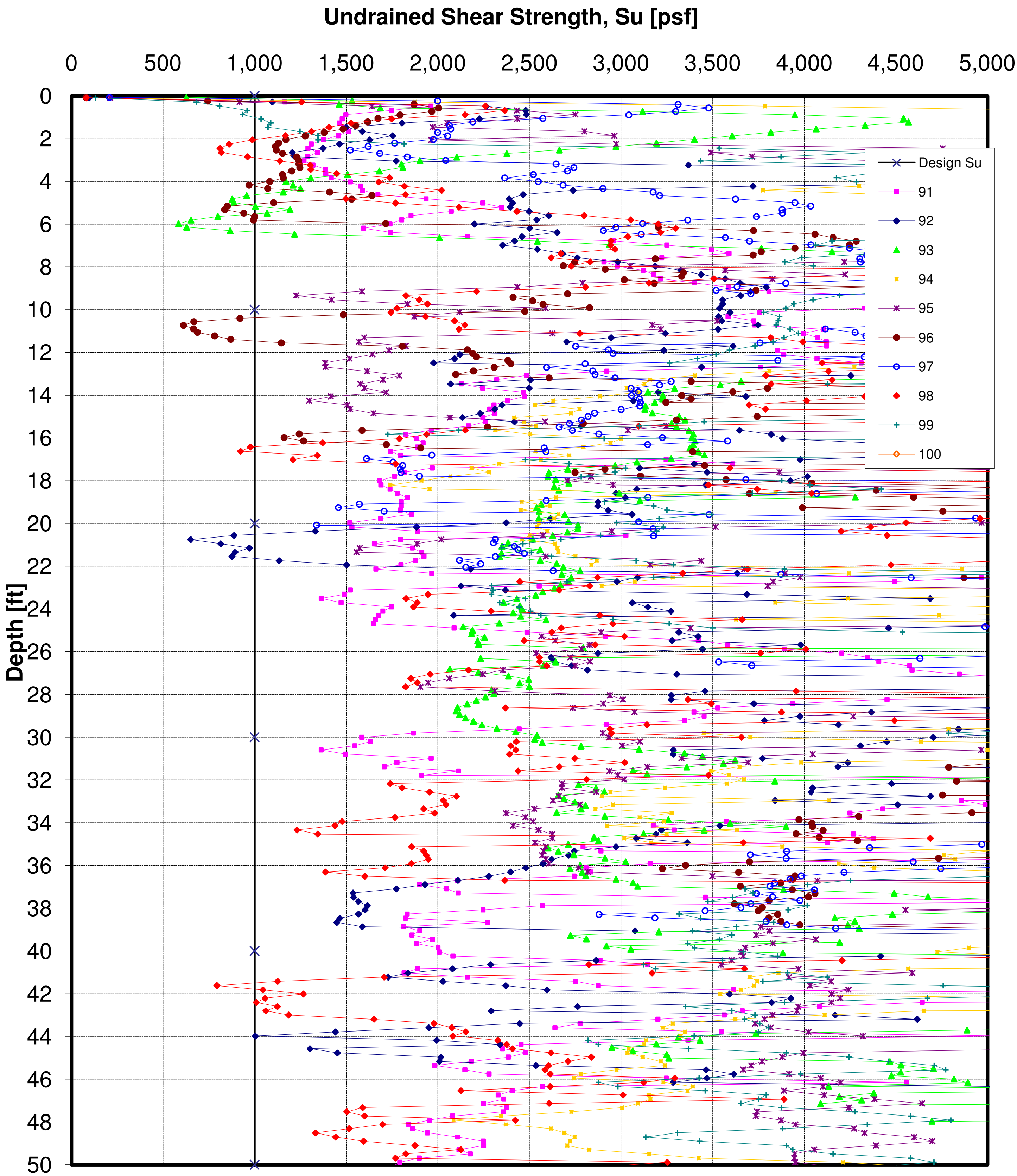

Figure 22. Undrained Shear Strength from CPT vs. Depth Turbines 91-100