REPORT

January 13, 2014
10-0014.3 S

Geotechnical Engineering Services

Proposed Blue Sky West Wind Power Project
Somerset and Piscataquis Counties
Bingham, Mayfield Township, and
Kingsbury Plantation, Maine

PREPARED FOR:
Reed & Reed, Inc.
Attention: Dustin Littlefield
Route 128
PO Box 370
Woolwich, Maine 04579

PREPARED BY:
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S.W.COLE
ENGINEERING, INC.

- Geotechnical Engineering
- Construction Materials Testing
- GeoEnvironmental Services
- Ecological Services

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ATTACHMENTS

10-0014.3 S

January 31, 2014

Reed & Reed, Inc.
Attention: Dustin Littlefield
Route 128
PO Box 370
Woolwich, Maine 04579

Subject: Geotechnical Engineering Services
Proposed Blue Sky West Wind Power Project
Bingham, Brighton Plantation, and Mayfield Township
Somerset and Piscataquis Counties, Maine

Dear Dustin:

In accordance with our Proposal dated September 13, 2013, S. W. Cole Engineering, Inc. (S.W.COLE) has completed geotechnical engineering services for the Proposed Blue Sky West Wind Power Project in Bingham, Mayfield Township and Kingsbury Plantation, Maine. This report summarizes the findings of our subsurface explorations and geotechnical engineering recommendations relative to foundation design and earthwork associated with the proposed foundation construction. The contents of this report are subject to the limitations set forth in Attachment A.

1.0 INTRODUCTION

1.1 Scope of Services

The purpose of our services was to explore the subsurface conditions at fifty-six (56) of the proposed sixty-two (62) turbine locations, operations and maintenance (O&M) building and along certain areas of proposed access roads and underground collector routes in order to develop geotechnical recommendations relative to foundation design and earthwork construction associated with construction.

Our services included fifty-six (56) test boring explorations at proposed wind turbine locations, two (2) test borings at the proposed O&M building, thirty-five (35) test probes along access roads and within areas of anticipated deep cuts, seventy-nine (79) test pit explorations along proposed access roads and underground collector routes, laboratory soils and bedrock testing, field soil resistivity testing at fifty-six (56) wind turbine locations, geophysical testing and a preliminary geotechnical evaluation of the findings as they relate to the proposed construction. It should be noted, the location of T12 moved approximately 250 feet northwest since our preliminary subsurface investigation. The proposed new location for T12 was explored during this investigation.

It should be understood the explorations and laboratory and field testing were made at selected locations. It is understood the recommendations provided herein are based on widely spaced explorations and laboratory test data and may need to be revised during construction when additional information becomes available.

1.2 Previous Services

S.W.COLE completed a preliminary geotechnical engineering services report dated December 14, 2012 for Reed & Reed, Inc. and preliminary geological investigation report dated December 21, 2010 for First Wind, LLC in association with evaluation of the proposed Blue Sky West Wind Power Project.

Our preliminary geologic services included reconnaissance of geological hazards and bedrock outcrops, verification of geologic mapping, laboratory acid base assessment (ABA) and evaluation of the earthquake seismic potential of the area.

Our preliminary geotechnical engineering services included six (6) test boring explorations at proposed wind turbines T5, T12, T28, T36, T47 and T56, nineteen (19) test pit explorations near proposed wind turbine locations and at the proposed substation site, laboratory soils and bedrock testing, field soil resistivity testing, geophysical testing and a preliminary geotechnical evaluation of the findings as they relate to the proposed construction.

Data and recommendations from our previous services have been incorporated and are discussed herein.

1.3 Proposed Construction

We understand the overall project includes the construction of sixty-two (62), Siemens SWT 3.0 wind turbines situated in about six strings oriented along the approximately northeast to southwest trending ridgeline of Johnson Mountain. The site extends about 6 miles south of State Route (SR) 16 along the ridgeline of Johnson Mountain and about 4 miles north of SR 16. Based on information provided on the “Permit Plan Submission” (Civil Plans) prepared by DeLuca-Hoffman Associates, Inc. (DHA) dated April 9, 2013, the proposed tower elevations vary from 1350 to 1800 feet above mean sea level. In addition, we understand the proposed development includes the construction of a substation, O&M building, underground collection system with areas of overhead collection, and about 21.3 miles of new and reconstructed gravel roadway for site access.

We understand twenty-six (26) wind turbines are planned south of SR 16 and thirty-six (36) wind turbines are planned north of SR 16. We understand the proposed substation will be located about 2.5 miles north of SR 16 and about 0.3 miles east of Hayden Pond and the O&M building will be located approximately 100 feet south of SR 16 and about one mile west of the intersection of State Routes 16 and 151. The general project location is shown on the “Site Location Map and Index”, attached as Sheet 1.

Based on the “Civil Plans,” we understand tapered cuts of up to about 40 feet and fills of up to about 30 feet high will be needed to achieve proposed turbine pad areas and roadway grades. The proposed substation yard will be approximately 300 by 400 feet in plan dimensions and will have a finish elevation of about 1492 to 1496 feet. The existing grade within the proposed substation area slopes downward from southwest to northeast from about elevation 1458 to 1506 feet requiring tapered fills up to 34 feet and cuts up to 10 feet. The proposed O&M building will be approximately 70 by 84 feet in plan dimensions with a FFE at about 1322 feet. We understand the proposed finish ground elevation will be about 1321 feet. The existing grade within the proposed O&M building area slopes downward from south to north from about elevation 1328 to 1320 feet requiring tapered fills of about 1 foot and cuts up to 7 feet high. Based on the “Civil Plans,” we understand fill slopes will generally be constructed with slopes of 2(H):1(V) or flatter and cut slopes will generally be constructed with slopes of 1(H):1(V) or flatter.

2.0 EXPLORATIONS

2.1 Test Boring and Test Probe Explorations

Fifty-six (56) test borings and fifty (50) test probes (P-1 through P-50) were made at the site between October 28 and December 10, 2013 by Northern Test Boring, Inc. of Gorham, Maine and Maine Test Borings of Hermon, Maine working under subcontract to S.W.COLE. The test boring locations were established in the field by S.W.COLE, using a mapping-grade Trimble GPS receiver, based on coordinates provided by Reed & Reed, Inc. (Reed & Reed).

The test borings and test probes were made using auger, cased rotary-wash drilling and NQ rock coring techniques. Soil sampling was generally performed, in the test borings, at 5-foot intervals using a split spoon sampler and Standard Penetration Testing (SPT) in general accordance ASTM D1586-84. Upon encountering a refusal surface, test borings were continued by rock core methods. Rock core sampling was performed in general accordance with ASTM D2113-83.

2.2 Test Pit Explorations

Seventy-nine (79) test pit explorations (TP-101 through TP-179) were made at the site between November 4 and 19, 2013 by Sargent Corp. of Stillwater, Maine under subcontract to Reed & Reed. Test pit locations were selected and located in the field by S.W.COLE using a mapping grade Trimble GPS receiver.

2.3 Previous Subsurface Explorations and Testing

We reviewed our "Preliminary Geotechnical Engineering Report" for the Blue Sky West Wind Power Project dated December 14, 2012. Our previous services included six (6) test borings at proposed Turbines T5, T12, T28, T36, T47, and T56, nineteen (19) test pit explorations (TP-1 through TP-19) near proposed wind turbine locations and at the proposed substation site, and laboratory and field testing. It should be noted, our "Preliminary Geotechnical Engineering Report" is superseded by this report. .

2.4 Exploration Locations and Elevations

Exploration elevations shown on the exploration logs are approximate and estimated based on ground surface contours as shown on the grading plans prepared by DHA dated April 9, 2013. The approximate exploration locations are shown on the "Exploration Location Plan(s)" attached as Sheets1A through 1M and the "Notes, Legend and Key to Testing" is attached as Sheet 1N.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Location and General Conditions

Based on the provided "Civil Plans," we understand the project site is orientated approximately southwest to northeast between Johnson Mountain in Bingham extending north across SR 16 toward Kingsbury Plantation. The site includes about 6 miles along the ridgeline of Johnson Mountain located south of SR 16 and extends approximately 4 miles along an unnamed ridgeline north of SR 16. The proposed turbine base elevations vary from about 1350 to 1800 feet above mean sea level. Based on our current and previous field services at the site, the site is currently wooded but has been cut in the past. Various tree cutting, woods logging roads and gravel roads traverse the project area.

3.2 General Geological Conditions

The *Surficial Geologic Map of Maine*[1] (Thompson and Borns, 1985) published by the Maine Geological Survey (MGS) indicates the Blue Sky West Wind Power Project area generally consists of thin (generally less that 10 feet) glacial drift (till) overlying and with multiple exposures of bedrock. Observations during our field reconnaissance and test pit and test boring explorations are generally consistent with the mapped surficial geology however, the depth to bedrock was observed to range from the ground surface to depths of more than 55 feet (the maximum depth explored).

The bedrock geology of the Blue Sky West Wind Power project area has been mapped as a portion of the *Bedrock Geologic Map of Maine*[2] (Osberg et al., 1985). The *Geologic Map of Western Interior Maine*[3] by Moench and Pankiwskyj (1988) provides a more detailed compilation of bedrock mapping by Ludman[4] (1978), Newell[5] (1978) and others, describing the bedrock as Devonian to Silurian age rocks of the Carrabasset and Madrid Formations within the Piper Pond Syncline. The younger Devonian, Carrabasset Formation is bounded to the north and south by the older Silurian, Madrid Formation. Ludman and Moench describe the Carrabasset Formation as a thinly layered to massive gray slate (pelite) and phyllite; the pelite is interbedded with metasandstone (arenite). The Madrid Formation is described as a variably bedded calcareous metamorphosed sandstone and pelite. The degree of regional metamorphism is mapped as increasing to the south and west, with staurolite grade metamorphism observed in the southwestern portion of the project area.

[1] Thompson, W. B. and Borns, H. B., eds., 1985, Surficial Geologic Map of Maine, Maine Geological Survey.
[2] Osberg, P. H., Hussey, A. M. , and Boone, G. M., eds., 1985, Bedrock Geologic Map of Maine, Maine Geological Survey.
[3] Moench, R. H. and Pankiwskyj, K. A., eds., 1988, Geologic Map of Western Interior Maine, Department of Interior, U. S. Geological Survey, Map I-1692.
[4] Ludman, A, 1978, Bedrock Geology of the Kingsbury Quadrangle, Maine; Maine Geological Survey Map Series GM-6.
[5] Newell, W. R., 1978, Geologic Map and Structure of the Bingham Quadrangle, Maine: Syracuse, N.Y., Syracuse University, pl. 1 of M.S. thesis.

Osberg, Moench and Ludman have mapped an inferred northeast to southwest trending strike-slip fault located just south of Otter Pond, Mayfield Pond, and Kingsbury Pond south of SR 16, approximately 6,000 to 18,000 feet southeast of the proposed turbine locations. This fault is mapped as being roughly parallel to the orientation of the primary bedding and foliation direction for bedrock in the region (30 to 35 degrees). The orientation of drainage patterns in the Carrabasset Formation is interpreted as a surficial expression of well developed bedrock joints oriented at angles of 50 to 80 degrees to the bedding and foliation.

3.2.1 Seismic – Faulting Data

Seismic activity can impact a site from two sources: ground rupture directly beneath a site or shaking produced at the site from seismic activity. There are no documented cases of ground rupture that can be definitely attributed to seismic activity in New England since the departure of glaciers more than 10,000 years ago. Bedrock deformation has occurred over geologic time; however documented faulting in the project area is limited to the single inferred northeast to southwest trending strike-slip fault noted above.

Ground motion or shaking is produced by seismic activity. The intensity of the ground motion decreases as the distance from a seismic event increases due to the absorption of energy by the earth. The table below lists the earthquake events and intensities within an approximately 10,000 kilometer^2 area centered on the proposed site.

According to the United States Geological Survey, 1,511 seismic (earthquake) events have been recorded within 1,000 km of the site since 1534. The nearest recorded earthquake event within a 10,000 km^2 area around the site (USGS, December 2013) was approximately 11.3 miles from the site. Based on available records, this event would have exhibited an approximate Peak Ground Acceleration[6] (PGA) of 0.0019g (0.19%g). This event was recorded in 1978. The maximum PGA calculated for the site was 0.0450g associated with an event visually recorded in 1755 approximately 170 miles from the site, this would convert to 4.5%g for the horizontal acceleration. This is consistent with 2008 USGS Seismic Hazard mapping (Sheet 2) for the region having a 10% probability for a Peak Horizontal Acceleration of 3 to 4%g in the next 50 years.

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[6] Peak ground acceleration (PGA) is a measure of earthquake acceleration on the ground and is not a measure of the total size of the earthquake, but rather how hard the earth shakes in an area. PGA is expressed in *g*, the acceleration due to earth's gravity (1g = 9.81 m s^{-2})

RECORDED SEISMIC EVENTS WITHIN A 10,000 KM^2 AREA OF THE PROPOSED BLUE SKY WEST WIND POWER PROJECT AREA					
Date	Richter Intensity	Event to Site Location Distance (miles)	(km)	log PGA	PGA
1978	2.3	11.3	18.2	0.2803	0.0019
1943	4.3	17.5	28.2	1.3222	0.0214
1967	3.0	18.2	29.3	0.5286	0.0034
1967	3.0	18.2	29.3	0.5286	0.0034
1967	4.3	18.2	29.3	1.3086	0.0208
1999	3.0	19.6	31.5	0.5023	0.0032
1947	4.3	19.7	31.6	1.2810	0.0195
1817	4.3	19.7	31.6	1.2810	0.0195
1929	2.3	21.8	35.1	0.0435	0.0011
1948	3.0	24.4	39.2	0.4234	0.0027
1948	3.7	24.4	39.2	0.8434	0.0071
1885	3.0	24.4	39.2	0.4234	0.0027
1930	2.3	25.0	40.2	0.0140	0.0011
1983	3.7	26.2	42.1	0.8175	0.0067
1980	2.3	26.6	42.8	0.0140	0.0011
1983	3.0	27.1	43.6	0.3849	0.0025
1948	2.3	27.2	43.7	0.0140	0.0011
1948	3.7	27.2	43.7	0.8035	0.0065
2004	2.0	27.8	44.8	0.0140	0.0011
2005	2.4	28.7	46.1	0.0140	0.0011
1940	3.0	30.6	49.2	0.3403	0.0022
1983	3.7	31.4	50.6	0.7501	0.0057
1983	3.7	31.4	50.6	0.7501	0.0057
1926	4.3	32.6	52.4	1.0969	0.0128
2008	2.5	33.3	53.6	0.0140	0.0011
1855	3.0	34.4	55.3	0.2970	0.0020
1888	3.7	35.2	56.6	0.7087	0.0052

Note: Data from USGS Earthquake database through 12-30-2013.

3.3 Subsurface Conditions

3.3.1 Wind Turbines

Test borings for turbines were made to depths of about 55 feet below the existing ground surface except at turbine location T11 which was abandoned within probable bedrock at a depth of about 40 feet due to very difficult drilling conditions consisting of numerous cobbles and boulders.

In general, the test borings encountered several inches to about 2 feet of surficial forest duff and/or soil with organics except at B-T36 where about 1 foot of sandy gravel with organics (fill) was observed. The surficial soils were underlain by a variable thickness of glacial till followed by bedrock that has been weathered to varying extent. In B-T5, B-T21, B-T34, and B-T53, zones of highly weathered bedrock (saprolite) varying from about 4 to 30 feet in thickness was observed overlying more competent bedrock. The depth to more competent[7] bedrock generally ranged from at or near the ground surface to a depth of 53 feet except at B-T9, B-T24, B-T45, and B-T55 where bedrock was not observed within the 55 foot exploration depth. The glacial till generally consists of medium dense to very dense gravelly silt and sand to gravelly silty sand with cobbles and boulders.

The recovered bedrock core is generally consistent with the mapped bedrock geology, consisting of variations within the Carrabasset Formation (metasandstone, interbedded pelite and metasandstone and pelite) and Madrid Formation (calcareous pelite). The calcareous nature of the Madrid Formation was confirmed by the Acid Base Assessment (ABA) testing described in Section 4.4. At B-T21, highly weathered bedrock associated with extensive fracturing was observed to be sufficiently extensive to allow split spoon sampling of the bedrock between layers of more competent rock. The Rock Quality Designation (RQD) of the cores ranges from 0 to 100 percent corresponding to a rock mass quality of very poor to excellent. The apparent depth to competent bedrock at each turbine exploration is presented in following table:

[7] Competent bedrock is interpreted by S.W.COLE to occur when it is possible to continuously collect bedrock core using standard core drilling methods.

APPARENT DEPTH TO COMPETENT[7] BEDROCK			
Exploration	**Approximate Depth (feet)**	**Exploration**	**Approximate Depth (feet)**
B-T1	0.7	B-T32	40.0
B-T2	8.5	B-T33	50.5
B-T3	3.8	B-T34	46.0
B-T4	0.8	B-T35	23.0
B-T5	17.3	B-T36	5.3
B-T6	7.1	B-T37	15.0
B-T7	6.0	B-T38	3.0
B-T8	6.0	B-T39	3.0
B-T9	Not Encountered	B-T40	3.0
B-T10	51.3	B-T41	2.0
B-T11	>40.0	B-T42	28.1
B-T12	52.5	B-T43	2.0
B-T12NEW	44.0	B-T44	4.2
B-T13	53.0	B-T45	Not Encountered
B-T14	29.6	B-T46	4.1
B-T15	19.5	B-T47	10.0
B-T16	6.3	B-T48	10.2
B-T17	5.9	B-T49	1.3
B-T18	2.5	B-T50	15.0
B-T19	3.9	B-T51	Not Drilled
B-T20	20.7	B-T53	34.0
B-T21	17.0	B-T54	4.8
B-T22	3.0	B-T55	Not Encountered
B-T23	4.1	B-T56	15.0
B-T24	Not Encountered	B-T57	2.0
B-T25	49.5	B-T58	7.5
B-T26	32.0	B-T59 (7ALT)	Not Drilled
B-T27	15.0	B-T73	4.6
B-T28	9.0	B-T74	4.8
B-T29	4.0	B-T75	8.8
B-T30	6.0	B-T76	24.0
B-T31	24.0	B-T77	35.0

Note: Test borings were made to depths of about 55 feet below the existing ground surface.

For more detailed descriptions of the findings at the turbine sites, please refer to the test boring logs located in Appendix A.

3.3.2 Operations and Maintenance Building

Test borings (O&M B1 and O&M B2) made for the proposed O&M building generally encountered a surface layer of forest duff overlying loose to medium dense sand with varying amounts of silt and gravel to depths of about 14 to 21.5 feet followed by a 3.5 to 4-foot thick layer of glacial till overlying probable bedrock or boulders. For more detailed descriptions of the findings at the O&M building, please refer to the test boring logs located in Appendix A.

3.3.3 Substation

Test pits (TP-2 through TP-7) made at the proposed substation during the 2012 investigation generally encountered a surface layer of forest duff and soils with organics (roots and rootlets) followed by brown to gray-brown silt with varying amounts of sand and gravel with cobbles and boulders (glacial till). Below the glacial till, refusal surfaces (probable bedrock) were encountered at depths of 2.5 to 5.6 feet except at TP-6 which was terminated in the glacial till at a depth of 8.8 feet. For more detailed descriptions of the subsurface findings, please refer to the exploration logs in Appendix C

3.3.4 Access Roadways and Utilities

Test pits TP-1, TP-8 through TP-19, and TP-101 through TP-179 were made for access roadways and utilities. These test pits generally encountered about 0.2 to 3.5 feet of forest duff and soils with organics (roots and rootlets) followed by glacial till with areas of relatively shallow refusal. Refusal surfaces were encountered at depths of 0.2 to 18.8 feet and interpreted to be probable bedrock or boulders. For more detailed descriptions of the subsurface findings, please refer to the exploration logs in Appendices A, B and C.

3.3.5 Groundwater Conditions

Saturated soils and groundwater seepage were observed in numerous test pit explorations at the time of excavation from October 29 to November 5, 2012 (TP-1 through TP19) and November 4 to 19, 2013 (TP-101 through TP-179). We observed groundwater seepage at depths ranging from about 1.5 to 11.5 feet. The seepage depths are indicated on the test pit logs in Appendix B.

We measured the depth to groundwater in the test borings upon completion of drilling, with the measurements recorded on the boring logs (Appendix A). The initial measured depths to water ranged from the ground surface to 21.3 feet. Follow-up water level measurements (as available) are included on the test boring logs. These follow-up water level measurements ranged for near the ground surface to a maximum depth of 21.7 feet. Test borings were advanced using “drive-and-wash” methods in the overburden and duel tube NQ rock coring in the bedrock. Both drilling methods use water pressure to remove drill cuttings from the borehole. Therefore, these drilling methods generally result in higher water levels than would be observed under natural conditions. The test borings were left open subsequent to drilling to make additional water level measurements. The additional borehole water level measurements indicate the depth to water varies with time, showing both increasing and decreasing depths to water subsequent to drilling. While potentially more representative of natural conditions, the water levels obtained from open boreholes are still influenced by the addition of drilling water and could be influenced by water bearing strata and may not reflect actual groundwater in an undisturbed setting. Oxidation of iron minerals at depths of greater than 50 feet were noted after a review of the recovered rock core. This depth of oxidation is interpreted to indicate historic seasonal low water levels. In areas with thick overburden (B-T11, B-T12 and B-T24), the observed soil saturation observed during drilling may be indicative of the seasonal water table.

We anticipate the shallow overburden will be saturated on a seasonal basis based on the low permeability of the glacial till. Groundwater elevations will vary seasonally. Elevated groundwater conditions may exist at changes in slope or at the base of hills.

4.0 LABORATORY AND FIELD TESTING

4.1 Laboratory Testing

Soil and bedrock samples recovered from the borings were visually examined by S.W.COLE in our laboratory. Laboratory testing was performed on selected samples recovered from the explorations. Laboratory testing is attached in Appendix D and includes the following:

Soil

- 23 Moisture Tests, ASTM D2216 (10 in 2012 and 13 in 2013)
- 26 Grain Size Analyses, ASTM C117/C136 (7 in 2012 and 19 in 2013)
- 1 Grain Size Analysis with Hydrometer, ASTM D422 (1 in 2012)
- 19 Moisture-Density Tests, ASTM D1557-09 (3 in 2012 and 16 in 2013)
- 3 Direct Shear Tests, ASTM D3080 (1 in 2012 and 2 in 2013)
- 11 Atterberg Limits, ASTM D4318 (4 in 2012 and 5 in 2013)

Rock Core

- 137 Unconfined Compressive Strength Tests, ASTM D7012 Method C (18 in 2012 and 119 in 2013)
- 137 Unit Weight Tests, ASTM C127 (18 in 2012 and 119 in 2013)

4.2 Soil Thermal Resistivity Testing

Thermal resistivity testing was performed on bulk soil samples obtained from test pits. Thirteen (13) laboratory soil thermal resistivity tests (three in 2012 and thirteen in 2013) with soil dry back curves in accordance with IEEE-442 were performed by Geotherm USA (Geotherm). In addition, five (5) laboratory thermal resistivity tests with soil dry back curves were conducted by S.W.COLE. The thermal resistivity testing results are included in Appendix E.

In-situ field soil thermal resistivity tests were performed in twenty-three (23) of the seventy-nine (79) test pits excavated in 2013. Data from these tests are shown on the test pit logs attached as Appendix B and summarized in Appendix E.

The field and laboratory thermal resistivity testing indicated similar results. We interpreted variations to be the result of differences in the density and moisture content between the field and the laboratory tests. In addition, laboratory samples were screened to remove coarser material, in order to perform the tests, which will also change the character of the soils.

This data will need to be reviewed by the electrical engineer for the project to evaluate their applicability to design criteria.

4.3 Laboratory Soil Chemistry Testing

Thirteen (13) soil samples (three in 2012 and ten in 2013) were submitted to Katahdin Analytical Services for determination of pH (SW846 9045D), water soluble chloride content (SW846 9251) and water soluble sulfate content (SW846 9038) testing. Results of the pH and water soluble chloride and sulfate testing as well as sulfate exposure classifications in accordance with ACI 318 Table 4.3.1 are included in Appendix F and shown in the following table:

Exploration	pH Testing	Chloride Testing (ppm)	Sulfate Testing (ppm)	Sulfate Exposure Classification (ACI 318 Table 4.3.1)
TP-1	4.8	< PQL	< PQL	Negligible
TP-2	4.7	< PQL	< PQL	Negligible
TP-9	5.7	25	< PQL	Negligible
TP-104/105	5.8	23	< PQL	Negligible
TP-128	5.8	< PQL	< PQL	Negligible
TP-129	6.5	21	< PQL	Negligible
TP-144	6.4	30	< PQL	Negligible
TP-154	5.9	22	< PQL	Negligible
TP-159	6.0	24	< PQL	Negligible
TP-161	6.1	< PQL	< PQL	Negligible
TP-163	6.0	23	< PQL	Negligible
TP-175	6.7	35	< PQL	Negligible
TP-178	6.5	< PQL	< PQL	Negligible

Notes

ppm = parts per million
PQL – Procedure Quantification Limit
PQL for chloride testing is 20 ppm
PQL for sulfate testing is 10 ppm

4.4 Laboratory Acid Rock Testing

A total of twenty-eight (28) bedrock samples (20 rock core and 8 bedrock outcrop) were submitted to Sturm Environmental Services (SES) of Bridgeport, West Virginia for acid base assessment (ABA[8]). These bedrock samples included:

- 7 outcrop samples from the preliminary geological services in November 2010;
- 1 outcrop sample from the preliminary geotechnical evaluation in November 2012;
- 5 core samples from the preliminary geotechnical evaluation in November 2012; and
- 15 core samples from the rock core samples in November and December of 2013.

The 1 outcrop and 5 rock core samples from 2012 were forwarded by SES to REI Consultants, Inc. (REIC) of Beaver, West Virginia for sulfate and chloride analyses. The results of the ABA, sulfate and chloride testing are included in Appendix G.

[8] ABA analysis includes analyses for Fizz, Color, Paste pH, Neutralization Potential (NP) and total sulfur, and are used to calculate Maximum Potential Acidity (MPA) and Net Neutralization Potential (NPP). NP, MPA and NPP are expressed in calcium carbonate equivalent Tons/100 Tons of Material.

We interpreted the ABA testing results to indicate the acid rock drainage (ARD) potential in the project area, with the exception of proposed turbine T21, is low (Appendix G). This interpretation is based on:

- Rock core paste pH values ranging from 4.1 (B-T21) to 8.8 (B-T56), with only one sample having a pH of less than 5.0;
- Total sulfur concentrations ranging from less than 0.001% to 0.798%, with only 2 samples having greater than 0.1% sulfur (outcrop sample T18BLR, and core from B-T21). The core sample from B-T21 was the only sample to have greater than 0.5% total sulfur[9];
- With the exception of the B-T21 core, rock samples were found to have excess calcium carbonate equivalent neutralizing potentials (NP) to preclude the development of ARD; and
- Only B-T21 has an NP/MPA ratio of less than 2 (calculated to be 0.3), indicating the potential to generate ARD.

We interpret the bedrock tested from boring B-T21 to have the potential to generate ARD based on the presence of sulfide minerals in combination with a fractured zone as a contributing factor to the high degree of weathering observed at this location. However, this ARD potential is based primarily on the limited buffering capacity (lack of carbonate minerals in the bedrock), as the limited amount of pyritic sulfur (0.372%) is generally not interpreted to be ARD producing (0.5% sulfur is generally accepted as the minimum criteria to produce ARD). Based on limited bedrock data available and the proposed construction cut and fill plans for turbine T21, it is our interpretation that the ARD potential at this location is sufficiently low to preclude the need for site specific ARD mitigation. However, we recommend that site conditions be reviewed during construction to confirm this interpretation.

4.5 Field Soil Resistivity Testing

S.W.COLE personnel performed Wenner array soil/rock resistivity testing at each of the proposed wind turbine sites and at the proposed substation location in general accordance with ASTM G57. Two Wenner Array test spreads with A-spacing's of 1, 2, 3, 5, 10, 15, 25, 35, 50, 75 and 100 feet were tested at each location[10], and were

[9] This sample was subsequently tested for sulfur forms, with 0.372% pyritic sulfur reported. Pyritic sulfur is interpreted as the primary form of sulfur contributing to ARD.

[10] Limited exceptions to the A-spacings and center point location are noted on the field forms.

configured with a common center point generally less than 15 feet from the boring made at that location. The results of the testing and test spread orientations with graphs of apparent resistivity and cumulative apparent resistivity are included in Appendix H.

The test spreads at the proposed substation site had a maximum A-spacing of 300 feet on spread A (oriented at magnetic N 31^{o}W) and a maximum A-spacing of 200 feet on spread B (oriented at magnetic N 31^{o}W).

Rain and snow events occurred at various times prior to and during the course of testing, resulting in generally moist to saturated surface soils. Care was taken, when frost was observed, to make sure test pins were in good contact with soil free of frost. Differences in results between the collocated test spreads are interpreted to be related to variables associated with depth to water, depth to bedrock, extent of bedrock weathering, bedrock type and the orientation of the bedrock fabric (bedding and/or foliation). Testing on subgrade crushed stone or during low water (drought) conditions will provide different results. In general, greater soil thickness, in combination with increased moisture content results in lower soil resistivity.

4.6 Geophysical Testing

Eight (8) seismic refraction and multichannel analysis of shear waves (MASW) surveys were conducted at turbine locations T5 and T28 in 2012 and T12, T18, T22, T28, T36, T56, and T73 in 2013 by Northeast Geophysical Services (Northeast Geophysical) of Bangor, Maine under subcontract to S.W.COLE. The purpose of the MASW surveys was to collect pressure wave (P-wave) and shear wave (S-wave) velocities at each survey location. The P-wave and S-wave velocities provided by Northeast Geophysical are summarized in the following table. Reports of the testing are attached as Appendix I.

P-WAVE AND S-WAVE VELOCITIES				
Turbine Site	Material	P-wave, V_p (fps)	S-wave[1], V_s (fps)	Depth to Apparent Bedrock (ft)
T5	Soil	3290	1370	17.3
	Rock	11270	5550	
T12	Soil	4170	2070	52.5
	Rock	12310	5790	
T18	N/A	N/A	N/A	2.0
	Rock	13180	6550	
T22	N/A	N/A	N/A	3.0
	Rock	11840	5740	
T28	N/A	N/A	N/A	9.0
	Rock	18570	8730	
T36	N/A	N/A	N/A	5.3
	Rock	12290	6710	
T56	Soil	2040	1050	12.4
	Rock	14740	7230	
T73	N/A	N/A	N/A	4.6
	Rock	12690	6650	

The provided P-wave and S-wave velocities were used to calculate the following dynamic soil and bedrock properties at each turbine location: Young's modulus, Poisson's ratio and shear modulus. The calculated values are presented in the following table.

DYNAMIC SOIL AND ROCK PROPERTIES					
Turbine	Material	Density, γ [1,2] (pcf)	Poisson Ratio ν	Shear Modulus, G_o (ksi)	Young's Modulus, E_o (ksi)
T5	Soil	120	0.40	50	130
	Rock	166	0.34	1100	2960
T12	Soil	120	0.34	110	300
	Rock	168	0.36	1210	3300
T18	Soil	N/A	N/A	N/A	N/A
	Rock	173	0.34	1600	4280
T22	Soil	N/A	N/A	N/A	N/A
	Rock	168	0.35	1190	3210
T28	Soil	N/A	N/A	N/A	N/A
	Rock	172	0.36	2830	7700
T36	Soil	N/A	N/A	N/A	N/A
	Rock	173	0.29	1680	4330
T56	Soil	120	0.32	30	75
	Rock	173	0.34	1950	5230
T73	Soil	N/A	N/A	N/A	N/A
	Rock	173	0.31	1650	4330

Note: 1) Soil density was estimated based on average N-values recorded for glacial till.
2) Rock density was estimated based on the average density from laboratory testing for each boring except for T36 and T56 were based on the average density from rock core testing.
3) Surface seismic methods can not adequately model a thin layer of soil; therefore the shallow velocities can not be used to reliably calculate these parameters.

5.0 PRELIMINARY GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

5.1 Geotechnical Considerations

Based on the subsurface findings and our understanding of the proposed construction, the proposed development appears feasible from a geotechnical standpoint. We offer the following summary of geotechnical consideration for design and construction:

- Wind Turbine Foundations: In general, the test borings encountered several inches to about 2 feet of surficial forest duff and soil with organics underlain by a variable thickness of glacial till with cobbles and boulders followed by bedrock and areas of relatively shallow bedrock. The depth to bedrock ranged from at the ground surface to a depth of greater than 55 feet, the maximum depth explored.

  We understand proposed foundation options will consist of either gravity spread footings or rock anchored mat foundations bearing directly on a lean concrete leveling pad underlain by undisturbed native glacial till soils, bedrock or compacted fill (Structural Fill or Crushed Stone) placed on the undisturbed native glacial till soils or bedrock.

  The foundation design engineer will need to make final determination of the selected foundation type. However, based on the proposed turbine pad grading and subsurface conditions, we anticipate the foundations at T1 through T4, T6, T8, T16 through T19, T23, T29, T30, T36, T38 through T41, T43, T44, T49, T54, T57, T73 and T74 may bear on bedrock. Turbines T7, T22, T28, T56, T58, and T75 may bear on glacial till overlying bedrock within about 5 feet of the foundation subgrade elevation. Turbines T5, T9 through T15, T20, T21, T24 through T27, T31, T32 through T35, T37, T42, T45, T48, T50, T51, T53, T55, T76, and T77 may bear on glacial till overlying bedrock at depth. We recommend the foundation designer assess potential dynamic considerations for shallow foundations bearing on bedrock. Additional foundation design considerations for turbines are discussed in Section 5.3.

- Substation Foundations: In general, the test pits encountered about 1.5 to 3.5 feet of surficial forest duff and soil with organics underlain by glacial till with cobbles and boulders followed by relatively shallow refusal. The refusal surfaces were encountered at depths of 2.5 to 5.6 feet and were interpreted as shallow bedrock.

Based on the existing and the proposed grading, we anticipate substation foundations will bear on bedrock or a thin layer of native glacial till overlying shallow bedrock in the southern portion transition to as much as 35 feet of compacted fill overlying native soils and bedrock in the northern portion. Foundation design considerations for the substation are discussed in Section 5.4.

- O&M Building Foundations: In general, the test borings encountered a surface layer of forest duff overlying loose to medium dense sand with varying amounts of silt and gravel to depths of about 14 and 21.5 feet followed by a 3.5 to 4-foot thick layer of glacial till overlying probable bedrock or boulder. Based on the existing and the proposed grading, we anticipate the O&M building foundations will bear medium dense native soils. Foundation design considerations for the O&M building are discussed in Section 5.4.

- Bedrock Excavations: Based on the subsurface conditions encountered, we anticipate bedrock excavation and removal will require blasting or other techniques to achieve the necessary grades. Considerations for the excavation and removal of bedrock can be found in Section 5.5.1.

- Groundwater: The depth to groundwater upon completion of the test borings ranged from at the ground surface to a depth of about 21 feet below ground surface. In addition, groundwater seepage was observed in several test pits at depths ranging from about 1.5 to 11.5 feet below ground surface. We anticipate excavations for turbine foundations will require dewatering techniques to help control below foundation grade. Dewatering considerations are discussed further in Section 5.5.2.

- Reuse of Native Soils: We understand the native soils are being considered for re-use as Common Borrow. We anticipate excavated soils suitable for reuse will consist of silt and sand with varying amounts of gravel and cobbles (glacial till) with a Plasticity Index of 6 or less. In our opinion, the native soils can be utilized provided they are at a moisture content that is workable for achieving the required compaction. Given the range in grain-size distribution and plasticity properties, the glacial till soils are moisture sensitive and will be very difficult to compact when above the optimum moisture content. Therefore, we do not recommend reuse of the native soils during wet and freezing conditions. Considerations for reuse of native soils as fill can be found in Section 5.7.

- Reuse of Blasted Bedrock: The bedrock is a resource for production of embankment fills to achieve proposed finish grade for the access roads and turbine areas. The blasted bedrock can be used as embankment fill provided the maximum particle size is less than 24 inches and used in appropriate size lifts. Ideally, the rock borrow should be mixed with sand and finer rock particles to reduce the percentage of voids in the fill. However, where there is a lack of overburden soil available or the blasting and/or crushing operations create a poorly graded borrow; the use of a geotextile separation fabric and/or surface choke stone material will be required to reduce the potential for loss of the fine material into the blasted bedrock. Considerations for reuse of blast rock as fill can be found in Section 5.7.

5.2 Site Preparation

We recommend an erosion control system be installed prior to clearing and grubbing activity at the site to help protect adjacent wetlands, drainages and areas outside of the construction limits. The soils that will be exposed will be subject to erosion. To reduce the potential for erosion, as much vegetation as possible should remain undisturbed adjacent to the construction site. Construction areas should be cleared and grubbed of all topsoil and soil with organics including roots.

Based on the subsurface findings, the thickness of forest duff and/or topsoil varies across the site from several inches to about 2 feet. The contractor should anticipate areas where roots and soils containing organics will extend several feet into the underlying soil. The methods used by the contractor for removal and the moisture condition of the site will affect the volume of material removal required. Topsoil and organics may be stockpiled and screened for reuse as a new topsoil layer in landscape areas. Suitability of the topsoil re-use from a nutrient and plant grown standpoint should be evaluated by a soil scientist prior to its use.

Upon completion of site preparation and prior to placing new fill soils, we recommend the exposed subgrade soils be observed by a representative of S.W.COLE and evaluated for the presence of soft, loose, disturbed or unsuitable materials.

5.3 Wind Turbine Foundation Considerations

5.3.1 Seismic and Frost Conditions

According to IBC 2009, Table 1613.5.2, we interpret the following Seismic Site Classes using the shear wave velocity method:

- Seismic Site Class A (for foundations on bedrock)
- Seismic Site Class B (for foundations on glacial till less than 35 feet thick)
- Seismic Site Class C (for foundations on glacial till greater than 35 feet thick)

We recommend consideration of the following parameters for seismic ground motions:

Seismic Site Class	S_S	S_1	S_{DS}	S_{D1}
A	0.273g	0.081	0.146	0.043
B	0.273g	0.081	0.182	0.054
C	0.273g	0.081	0.219	0.092

Based on the subsurface conditions encountered, in our opinion liquefaction is not a design consideration.

According to *Design and Construction of Frost-Protected Shallow Foundations* (SEI/ASCE 32-01), the design air-freezing index for the Bingham area is about 2,000 Fahrenheit degree-days, which corresponds to a frost penetration depth on the order of 5.5 feet. We recommend foundations exposed to freezing be covered with at least 5.5 feet of soil for frost protection.

5.3.2 Gravity Spread Footing Foundations

We recommend the proposed turbines supported on gravity spread footing foundations be founded directly on a lean concrete leveling pad underlain by undisturbed native glacial till soils, bedrock or compacted fill (Structural Fill or Crushed Stone). We recommend the following geotechnical parameters for preliminary foundation design consideration:

- Net Allowable Soil Bearing Pressure = 4.5 ksf or less (on undisturbed native glacial till or compacted Structural Fill or Crushed Stone)
- Maximum Edge of Foundation Toe Pressure = 6.0 ksf or less (on undisturbed native glacial till or compacted Structural Fill or Crushed Stone)
- Net Allowable Soil Bearing Pressure = 20.0 ksf or less (on bedrock)
- Base Friction Factor = 0.4 (Concrete to undisturbed native glacial till or compacted Structural Fill or Crushed Stone)
- Base Friction Factor = 0.6 (Concrete to bedrock)
- Total Unit Weight of Backfill = 125 pcf (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Internal Friction Angle of Backfill = 32 degrees (Structural Fill)
- Total Unit Weight of Backfill = 120 pcf (native glacial till)
- Internal Friction Angle of Backfill = 36 degrees (densified native glacial till)
- Ultimate Passive Soil Pressure Coefficient = 3.0 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- At-Rest Soil Pressure Coefficient = 0.5 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)

Note: The parameters provided above assume drained conditions above bottom of footing grade.
Geotechnical parameters must be assessed by the foundation designer.

Due to the estimated size of the gravity spread footing foundations, variations in the foundation subgrade conditions should be anticipated in portions of the foundation area not explored. S.W.COLE should be on site to observe the foundation excavation and subgrade preparation prior to placement of the lean concrete working mat.

5.3.3 Rock Anchor Mat Foundations

We understand rock anchors will be used to resist overturning for turbine foundations where bedrock is less than about 10 to 15 feet from finish grade. We anticipate rock anchor mat foundations will be founded on a lean concrete leveling pad underlain by undisturbed native glacial till soils, bedrock or compacted fill (Structural Fill or Crushed Stone) placed on the undisturbed native glacial till soils underlain by relatively shallow bedrock. We recommend the following geotechnical parameters for preliminary rock anchor design consideration:

- Net Allowable Soil Bearing Pressure = 4.5 ksf or less (on native glacial till or compacted Structural Fill or Crushed Stone)
- Maximum Edge of Foundation Toe Pressure = 6.0 ksf or less (on native glacial till or compacted Structural Fill or Crushed Stone)
- Net Allowable Soil Bearing Pressure = 20.0 ksf or less (on bedrock)
- Base Friction Factor = 0.4 (Concrete to native glacial till or compacted Structural Fill or Crushed Stone)
- Base Friction Factor = 0.6 (Concrete to bedrock)
- Total Unit Weight of Backfill = 125 pcf (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Internal Friction Angle of Backfill = 32 degrees (Structural Fill)
- Total Unit Weight of Backfill = 120 pcf (native glacial till)
- Internal Friction Angle of Backfill = 36 degrees (densified native glacial till)
- Ultimate Passive Soil Pressure Coefficient = 3.0 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- At-Rest Soil Pressure Coefficient = 0.5 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)

Note: The parameters provided above assume drained conditions above bottom of footing grade.
Geotechnical parameters must be assessed by the foundation designer.

Based on the subsurface conditions and guidance from the Post-Tensioning Institute's manual entitled *Recommendations for Prestressed Rock and Soil Anchors* (PTI, 2004), we recommend the use of prestressed, Class I corrosion protection, grouted rock anchors be considered by the foundation designer. We recommend the following geotechnical parameters for preliminary rock anchor design consideration:

RQD (see boring logs – Appendices A and C)	0 to 100%
Average Dry Unit Weight of Bedrock	173 pcf
Rock Cone Pull-Out Angle (from vertical)	45 degrees (from vertical)
Average Ultimate Grout to Bedrock Bond Strength	120 psi

Based on guidance from the *Recommendations for Prestressed Rock and Soil Anchors* (PTI, 2004) we recommend a minimum unbonded length (free-stressing length) of 15 feet for strand tendons and 10 feet for bar tendons be considered for preliminary rock anchor design. The bonded length will depend upon the uplift load and the diameter of the drill hole. Rock anchor spacing should be at least 1.2 times the free-stressing length; closer spacing will reduce allowable anchor loads. Rock anchors installed in groups should be designed with consideration of pullout resistance from overlapping failure surfaces extending from the midpoint of the anchor bond zone to the bedrock surface.

The drill-hole for each rock anchor should be cleaned of any drilling fines and tightness tested to determine the need for pre-grouting. Rock anchors should be installed, tested and locked-off according to the design engineer's recommendations.

5.3.4 Settlement

Post-construction settlements for foundations bearing on properly prepared soil are estimated to be less than 1 inch. Settlements for foundations on properly prepared bedrock are estimated to be less than ½ inch. Differential settlements are estimated to be half of the anticipated total settlements. S.W.COLE should observe the foundation subgrades prior to placing new fill or lean concrete leveling pads, formwork, reinforcing steel and concrete placement.

5.3.5 Foundation Drainage

We recommend perimeter underdrains be installed at the foundation subgrade elevation. Underdrain pipe should consist of 4 or 6-inch diameter perforated foundation drain pipe enveloped in at least 6-inches of Crushed Stone wrapped in a geotextile filter fabric, such as Mirafi 160N or equivalent. The underdrains may have long runs to get to gravity outlet. The underdrains and pipe runs must be protected from freezing and have positive gravity outlets protected from freezing and clogging.

5.3.6 Buoyancy

If foundation drainage is not practical to daylight, we recommend foundation design considering buoyancy due to hydrostatic fluid pressure (i.e. 62.4 psf/foot) extending from finish grade.

5.3.7 Dynamic Foundation Soil Stiffness

Foundation soil stiffness parameters were estimate based on generalized soil profiles and guidance from the DNV/Risφ design manual entitled *Guidelines for Design of Wind Turbines* (2002) to assist the foundation designer. Variations in the foundation soils and engineering properties should be anticipated. We recommend the following geotechnical parameters for design consideration of the overall foundation stiffness:

Parameter	Glacial Till	Bedrock
Shear Wave Velocity[1] (V_s)	1200 fps	6500 fps
Low-Strain Shear Modulus (G_o)	40 ksi	1600 ksi
Small Strain Elastic Modulus (E_o)	110 ksi	4250 ksi
Large Strain Shear Modulus (G)	15 ksi	630 ksi
Large Strain Elastic Modulus (E)	45 ksi	1700 ksi
Poisson's Ratio	0.40	0.35

Notes: 1) Average shear wave velocities from geophysical testing completed at Turbines 5, 12, 18, 22, 28, 36, 56, and 73.

We recommend the foundation designer review the information provided in this report and select appropriate parameters for design.

5.4 Substation and O&M Building Foundation Considerations

5.4.1 Substation Foundations

Based on the subsurface conditions and proposed grading, we anticipate foundation subgrades will consist of bedrock in the southern portion and up to 35 feet of tapered compacted fill overlying glacial till and bedrock in the northern portion. We recommend the following geotechnical parameters for substation foundation design consideration:

- Seismic Site Class B (based on IBC, 2009 N-value method)
- Design Frost Depth of Footings on Soil = 5.5 feet
- Design Frost Depth of Footings on sound, intact, Bedrock = 2.5 feet
- Net Allowable Soil Bearing Pressure = 4.0 ksf or less (on undisturbed native glacial till or compacted Structural Fill or Crushed Stone)
- Net Allowable Soil Bearing Pressure = 20.0 ksf or less (on prepared bedrock)
- Estimated Post Construction Settlement = ½ inch or less (total settlement)
- Base Friction Factor = 0.4 (concrete to native glacial till, compacted Structural Fill or Crushed Stone)
- Base Friction Factor = 0.6 (concrete to bedrock)
- Total Unit Weight of Backfill = 125 pcf (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Internal Friction Angle of Backfill = 32 degrees (Structural Fill)
- Total Unit Weight of Backfill = 120 pcf (native glacial till)

- At-Rest Soil Pressure Coefficient = 0.5 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Active Soil Pressure Coefficient = 0.3 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Ultimate Passive Soil Pressure Coefficient = 3.0 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Modulus of Subgrade Reaction = 200 pci (Structural Fill)

Note: The parameters provided above assume drained conditions above bottom of footing grade.
Geotechnical parameters must be assessed by the foundation designer.

Spread footings should be at least 24 inches in width regardless of the bearing pressure. We recommend spread footings be placed on 12 inches of compacted Structural Fill (if overlying native or fill soils) or either 12 inches of Crushed Stone or 12 inches of Structural Fill overlying a geotextile fabric (if overlying fractured bedrock or blasted bedrock fills).

5.4.2 Operations and Maintenance Building Foundation

Based on the subsurface conditions and proposed grading, we anticipate the O&M building foundation subgrade will consist of medium dense native soils overlying bedrock with depth. We recommend the following geotechnical parameters for O&M building foundation design consideration:

- Seismic Site Class B (based on IBC, 2009 N-value method)
- Design Frost Depth of Footings on Soil = 5.5 feet
- Net Allowable Soil Bearing Pressure = 3.0 ksf or less (on native soils)
- Estimated Post Construction Settlement = ½ inch or less (total settlement)
- Base Friction Factor = 0.4 (concrete to native glacial till, compacted Structural Fill or Crushed Stone)
- Total Unit Weight of Backfill = 125 pcf (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Internal Friction Angle of Backfill = 32 degrees (Structural Fill)
- Total Unit Weight of Backfill = 120 pcf (native glacial till)
- At-Rest Soil Pressure Coefficient = 0.5 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)

- Active Soil Pressure Coefficient = 0.3 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Ultimate Passive Soil Pressure Coefficient = 3.0 (Structural Fill compacted to 95% of modified Proctor - ASTM D1557)
- Modulus of Subgrade Reaction = 200 pci (native subgrade)

Note: The parameters provided above assume drained conditions above bottom of footing grade.
Geotechnical parameters must be assessed by the foundation designer.

Spread footings should be at least 24 inches in width regardless of the bearing pressure. We recommend spread footings be placed on 12 inches of compacted Structural Fill overlying native soils.

5.5 Excavations and Dewatering

5.5.1 Excavations

Excavations will generally encounter forest duff and topsoil, sandy silt to sand and silt with varying amounts of gravel and cobbles and boulders, and shallow bedrock. Care must be exercised during construction to reduce potential for disturbance of foundation and roadway subgrades. We recommend a smooth-edged bucket be utilized to excavate to final soil subgrades. Construction traffic on soil subgrades should be avoided when practical. Should subgrades become disturbed, the subgrade should be over-excavated to expose suitable soil and replaced with compacted Structural Fill or Crushed Stone or moisture conditioned glacial till and recompacted. Because site soils are generally wet to saturated, the use of a woven geotextile fabric should be anticipated particularly in cut areas prior to placement of roadway material.

Based on the proposed grading and subsurface conditions, we anticipate bedrock removal will be needed to achieve the required subgrade elevation throughout the project. We anticipate some of the surficial weathered bedrock may be able to be excavated using a large excavator with ripping teeth and hoe-ramming techniques. However, we anticipate most of the bedrock removal will require drilling and blasting techniques. We recommend an experienced drilling and blasting contractor be retained to perform bedrock removal. In addition, we recommend the subcontractor submit a detailed drilling and blasting plan with qualifications and references prior to blasting.

5.5.2 Dewatering

Sumping and pumping dewatering techniques should be adequate to control water inflow into excavations above the groundwater table. Controlling the water levels below the groundwater table may require sheeting and extensive dewatering in order to maintain a stable excavation. Temporary, unsupported soil excavations should be sloped back to 1½(H):1(V) or flatter. In all cases, excavations must be properly shored and/or sloped according to OSHA regulations to prevent sloughing and caving of the sidewalls during construction.

When working at the bottom of slopes, temporary dewatering may require construction of uphill cut-off swales and/or diversion berms to direct upgradient runoff water away from the work areas.

5.6 Embankment Construction

The "Civil Plans" prepared by DHA indicate fill soil slopes for turbine pads and roadways will generally be constructed with slopes of 2(H):1(V) or flatter and cut slopes will generally be constructed with slopes of 1(H):1(V) or flatter.

5.6.1 General

Fill slopes should be constructed as level benches, which are overbuilt to facilitate compaction. The final slope face should be constructed by cutting back into the compacted core prior to placing slope surface materials. Fill slopes constructed on existing terrain steeper than 3(H):1(V) should be keyed into the existing ground surface with continuous level benches. Fill slopes constructed on existing slopes flatter than 3(H):1(V) do not need continuous benching. We recommend a 10 foot wide bench be cut into the native soil beneath the toe of fill slopes for installation of a 1-foot thick drainage blanket consisting of Gravel Borrow or Rock Borrow mixed with Gravel Borrow prior to placing fill soils. The drainage blanket should be day-lighted for gravity drainage.

5.6.2 Fill Slopes 2(H):1(V) or Flatter

Backfill materials needed to construct fill slopes at inclinations of 2(H):1(V) or flatter should consist of compacted Common Borrow, Gravel Borrow, Rock Borrow, Structural Fill or Crushed Stone. Exposed soil slopes will be susceptible to surface erosion, slumping and sloughing, particularly during heavy rain and freeze/thaw events. Exposed slopes should be surfaced with an erosion control blanket and loam and seed, as soon as practicable, to create a vegetated mat. In areas of concentrated surface water, we recommend 8-inch minus rip-rap overlying a geotextile fabric be used in lieu of the erosion blanket and loam and seed. We recommend cross-slope stone lined drainage channels underlain with geotextile fabric be construct into the slope face when the height of the embankment exceeds 25 feet.

5.6.3 Fill Slopes Steeper than 2(H):1(V)

Although not anticipated, if proposed fill slopes are to be constructed steeper than 2(H):1(V), we recommend these slopes be constructed with compacted Rock Borrow and the slopes be covered with at least 2 feet of compacted rip-rap. Further, lateral edges where the riprap terminates along the face of the embankment should be similarly keyed into the ground surface. We recommend slopes be constructed no steeper than 1.5(H):1(V). Rock Borrow should be controlled to maximum particle size of 24 inches and be placed in horizontal lifts not exceeding 36 inches. The Rock Borrow should be placed in a manner to reduce the potential for voids by infilling with sand and smaller stone particles to create a well graded matrix. If overburden soil is not available for infilling or the blasting operations create a course poorly graded rock borrow lacking fines, a geotextile separation fabric and a surficial choke stone layer will be required at the top of subgrade prior to placing aggregate road base products.

5.6.4 Cut Slopes

We recommend proposed cut slopes less than 15 feet in height consider slope inclinations of 2H:1V or flatter since the depth to bedrock is unknown between exploration locations and areas of outcropping bedrock. The final slope inclination will be dependant on the subsurface conditions (soil or bedrock) encountered during construction. Cut slopes in bedrock should be sloped back to a stable condition, which will depend on rock fracturing, as well as bedrock formation strike and dip in relation to slope orientation. We recommend a representative from S.W.COLE observe the bedrock slopes during construction.

We recommend a rock fall catchment zone be provided at the toe of rock cut slopes. We developed the sizing of the catchment zone utilizing slope height and inclination following FHWA Publication No. HI-99-007 *Rock Slopes Reference Manual*. We recommend a rock fall catchment zone measuring 15 feet horizontally from the toe of slope and 5 feet below the adjacent finish road or turbine pad grade.

In addition, we recommend a minimum 5-foot wide bench be constructed at the interface of the overburden soil and bedrock to reduce potential erosion that could cause soils, cobbles and boulders to wash down the rock slopes potentially clogging drainage swales and causing blocking hazards.

In areas of concentrated surface water or locations of groundwater seeps, rip-rap should be used in lieu of the erosion blanket and loam/seed. We recommend cross-slope stone lined drainage channels underlain with geotextile fabric be constructed into the slope when the height of the slope exceeds 25 feet.

5.6.5 Slope Surface Erosion Control

Unprotected and un-established slopes, regardless of inclination, will be susceptible to surface erosion, slumping, and sloughing especially during precipitations and freeze/thaw events. Topsoil and seed should be installed, as soon as practicable, to create a vegetated mat over the entire surface of the slope. We recommend the use of UV resistant synthetic erosion control mesh to reinforce the surface soils until the vegetated mat is established, particularly if constructed during the winter or spring seasons.

Groundwater seepage and upgradient runoff water will make establishment of soil slopes difficult. In areas where surface water may be concentrated and discharged over the slope or where groundwater seepage is encountered, we recommend locally covering the slope with a small diameter rip-rap placed over a layer of crushed gravel and a woven filter fabric.

5.7 Backfill and Compaction

Although a wide range of soil materials can be used successfully, it has been our experience granular soils with good drainage characteristics provide significant advantages particularly in wet conditions and during cold weather construction. We have made recommendation for materials that are suitable for support of the proposed construction from a geotechnical standpoint. However, the electrical designer must develop parameters for fill to achieve proper compatibility between the fill soils and the electrical grounding system. In general, we recommend the following materials for consideration:

Common Borrow: We anticipate on-site glacial till soils generated from mass excavations may be used as Common Borrow for embankment fill. However, the glacial till soils are not well-suited for reuse during wet or freezing conditions. Common Borrow should meet the requirements of MaineDOT Standard Specification 703.18 “Common Borrow”. Gradation testing indicated the glacial till consists generally of sandy silt to sand and silt with varying amounts of gravel with cobbles and boulders.

Laboratory soil moisture contents on samples of the glacial till ranged from about 9 to 21 percent. Based on the completed moisture-density testing, optimal moisture contents of the glacial till soils ranged from about 5.5 to 13.5 percent. Therefore, it should be anticipate that the glacial till soils may be saturated requiring aeration and moisture conditioning to attain compactable moisture content. To promote the workability and compaction, glacial till may need to be stockpiled and dried prior to its use as fill. The suitability of re-use of the native glacial till will be dependent upon weather conditions and soil moisture content at the time of use.

Common Borrow is recommended for use as:

- Embankment fill to raise general turbine pad areas and access roads

Gravel Borrow: Depending on cut and fill quantities and amount of re-use of native soil, some import material may be needed. Imported materials consisting of a well-graded mixture of sand, gravel and silt or reclaimed concrete, brick, and crushed rock that is crushed and blended with sand to create a compactable fill. Gravel borrow should be the requirements for MaineDOT Standard Specification 703.20 “Gravel Borrow”. The maximum particle size should not exceed two-thirds of the loose lift thickness.

Gravel Borrow is recommended for use as:

- Drainage blanket at the base of embankment fills
- Initial lift over wet areas requiring fill
- Embankment Fill

Rock Borrow: On-site rock generated from mass excavation and on-site blasting activities may be used as Rock Borrow for embankment fill. Excavated blast rock should be broken to various sizes that will form a compact fill with a minimum of voids. Blasted rock fill should meet the gradation requirements for MaineDOT Standard

Specification 703.21 "Rock Borrow" with a maximum particle size of 2 feet. Rock Borrow fills should be choked with a crushed stone layer such that embankment fills or road surfacing materials placed above do not infiltrate into the rock fill.

Rock Borrow is recommended for use as:

- Embankment Fill
- Backfill of overexcavated areas
- Drainage blanket below toe of new embankments

Structural Fill: Fill to raise grades for buildings and turbines and backfill for over-excavations and foundations should be clean, non-frost susceptible sand and gravel meeting the gradation requirements for Structural Fill as given below.

STRUCTURAL FILL	
Sieve Size	**Percent Finer by Weight**
4 inch	100
3 inch	90 to 100
¼ inch	25 to 90
#40	0 to 30
#200	0 to 5

Structural Fill is recommended for use as:

- Backfill adjacent to turbines and building foundations
- Backfill within frost zone for foundations
- Backfill for repair of soft yielding areas above water table

Crushed Stone: Slab base materials and drainage aggregate around foundation underdrains should meet the requirements for MaineDOT Standard Specification 703.22 Type C "Underdrain Aggregate".

CRUSHED STONE **MaineDOT 703.22 Underdrain Backfill Material (Type C)**	
Sieve Size	**Percent Finer by Weight**
1 inch	100
¾ inch	90-100
⅜ inch	0-75
#4	0-25
#10	0-5

Crushed Stone is recommended for use as:

- Fill 12 inches below turbines, buildings and substation foundations
- Choke Stone over Rock Borrow (fill voids over the rock borrow fill and in fractured bedrock surfaces)
- Backfill for repair of soft yielding areas below water table or on wet soils
- Use around underdrains

Placement and Compaction: Fill should be placed in horizontal lifts and compacted such that the desired density is achieved throughout the lift thickness with 3 to 5 passes of the compaction equipment. Loose lift thicknesses for grading, fill and backfill activities should not exceed 12 inches, except for Rock Borrow fills which should not exceed 36 inches provided large compaction equipment is being used.

The moisture content of soils should be as such to achieve project compaction specifications. Regardless of the percent compaction achieve, the surface of the compacted soil should not rut, pump, or weave under the compaction or construction equipment. This will require use of a narrow fluctuation of moisture content from the optimum moisture content as determined by ASTM D-1557. The more fines a soil contains the narrower the moisture content range. We recommend fill and backfill adjacent to and below turbines be compacted to at least 95 percent of its maximum dry density as determined by ASTM D-1557. Crushed Stone below foundations, if needed, should be compacted to 100 percent of its dry rodded unit weight in accordance with ASTM C-29.

5.8 Earthwork Weather Considerations

5.8.1 General Weather Considerations

The site soils are likely at or near saturation much of the year. The site soils are moisture and frost sensitive and will lose strength when disturbed during wet and freezing conditions. Site work and construction activities should take appropriate measures to protect exposed soils and subgrades.

The native glacial till material will be difficult to re-use during wet and cold weather, and the amount of fine sand and silt will create difficulties for reuse during freezing conditions. The contractor should anticipate the need for moisture conditioning fills to facilitate compaction during dry or wet weather. We recommend filling be limited during wet and cold conditions or alternative materials that have better drainage characteristics and are non-frost susceptible be used.

5.8.2 Winter Weather Considerations

Construction activity during cold weather should be undertaken in a manner that considers construction schedule relative to frozen soils. If foundation construction takes place during cold or freezing weather conditions, subgrades must be protected from freezing conditions.

If earthwork occurs during freezing conditions, we recommend subgrades soils and fills be allowed to thaw before fill placement activities can continue. Subsequent lifts of soil must not be placed on frozen soil and once placed; the soil must be protected from freezing. If soils become frozen, the frozen soils should be allowed to thaw and then recompacted prior to placing subsequent lifts. Alternatively, frozen soils may be excavated to reveal unfrozen soil prior to placing subsequent lifts of fill or foundations.

5.9 Access Roadways and Crane Pads

The access roadways and crane pads will be subject to construction vehicles, transport vehicles carrying turbine parts and assembly cranes. Based on the “Civil Plans,” we understand the typical construction roadway section will consist of 12 to 24 inches of compacted 6-inch minus Rock Borrow as indicated on Plan Sheet C5.0 overlying densified native soils or compacted fill (Common Borrow, Rock Borrow, Structural Fill or Crushed Stone). We anticipate associated drainage ditches, swales and culverts will be designed by others. We recommend the civil engineer design ditches and swales deep enough to allow continuous drainage of the roadway section materials and subgrade soils.

In addition, we anticipate the site civil engineer is considering the construction of a “rock sandwich” roadway detail. These roadways will likely consist of compacted roadway fill overlying a 12 inch minimum rock drainage layer (rock sandwich layer) with stones having a D_{50} (grain diameter at 50 percent passing) of 4 inches. Typically the rock drainage aggregate is wrapped in a non-woven geotextile fabric, such as Mirafi 160N or equivalent. We understand culverts within the rock sandwich shall be placed such that a minimum of 6 inches of rock drainage aggregate be included below the culvert. We anticipate H-20 loading will be the same magnitude as on related projects; therefore, we recommend culverts being designed for H-20 loading have at least 2 feet of gravel cover.

We recommend roadway aggregate surfacing and base materials be compacted to at least 95 percent of its maximum dry density as determined by ASTM D-1557. To reduce the loss of surface material during heavy rain events, the roadway should be sloped to allow the water to flow to adjacent ditches or swales as soon as possible. Design of access roadway section thicknesses should be re-evaluated once the final turbine layout is selected as shared roadways will have higher traffic loadings and will require thicker sections.

Based on our previous experience, we estimate the anticipated maximum track contact pressures for assembly cranes will be about 8.0 ksf. Based on the subsurface conditions at the proposed turbine locations, subgrade preparation recommendations and an assumed track width of 3.5 feet, we recommend the crane pads consist of 2.0 feet of compacted Crushed Stone or 1 foot of MaineDOT Type A Gravel underlain by 1 foot of MaineDOT Type D Gravel underlain by undisturbed native glacial till. This bearing capacity analysis is based on a safety factor of 1.2 and an allowable settlement of ½ inch for the temporary loading condition.

5.10 Utility Trenches and Poles

Based on the proposed grading, we anticipate that utility trenches and poles could be constructed in a variety of soils ranging from excavated native soil or bedrock to a variety of embankment fills. Installation of utility poles will either require open excavation and backfill or auger drilling. We anticipate auger drilling will be difficult in areas where embankment fills are comprised of blasted Rock Borrow. In area of shallow intact bedrock drilled installation of the utility poles may require rock coring.

We recommend utility trenches made in native soil or embankment fills be backfilled with material as similar to the soils in the sidewalls a practical to reduce the differential effects that may occur with respect to subsurface water flow and frost action. If the utility trench backfill is comparably more permeable than the adjacent trench sidewalls, the trench will become a conduit for subsurface water flow.

Where utility trenches are cut into bedrock and in areas sensitive to collection or flow of water, consideration should be given to use of trench dams using low permeable soil, low-strength control density flowable fill, or other suitable method's as directed by the civil engineer.

5.11 Corrosion Potential

5.11.1 Concrete Corrosion Potential

The primary chemical in soil and water with respect to disintegration of concrete is sulfate (SO_4). The American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* (ACI 318-11) considers sulfate exposure negligible where the water-soluble sulfate (SO_4) is less than 150 ppm.

Results of the sulfate testing on selected samples indicated sulfate concentrations were less than 150 ppm, therefore ASTM Type I Portland cement appears appropriate for use in below grade concrete foundations.

5.11.2 Steel Corrosion Potential

The Post-Tensioning Institute publication *Recommendations for Prestressed Rock and Soil Anchors* (PTI, 2004) indicates ground environments may be classified as aggressive if one of the following conditions are present or may be present in the ground during the service life of the ground anchor:

- Soil or groundwater pH value of less than 4.5;
- Resistivity of less than 2000 ohm-cm;
- Presence of sulfides;
- Presence of stray currents; or
- Observation of corrosion or direct chemical (acid) attack on adjacent buried concrete.

Results of the pH testing during this phase of investigation found the average bedrock and soil pH to be about 7, with sulfur concentrations generally less than 0.01%. Bedrock testing during our preliminary phase did not detect sulfate or chloride. With the exception of the core from T21, bedrock was not tested for sulfate or chloride during this phase of investigation. The rock core from T21 was found to contain 0.391% sulfate sulfur, and we recommend these data be reviewed by the corrosion specialist for the project.

Results of our soil resistivity testing indicate the apparent resistivity varies from a geometric mean of 291 ohm-meters (29,100 ohm-cm) at T13 to 9,954 ohm-meters at T39. The geometric mean of the apparent resistivity for all soil resistivity locations is about 3,950 ohm-meters.

Corrosion protection of ferrous materials should be provided per the manufacture's specifications and upon the recommendations of a corrosion specialist.

5.12 Design Review and Construction Services

We request S.W.COLE be provided the opportunity to review the final design and specifications to determine that our earthwork and foundation recommendations have been properly interpreted and implemented.

We recommend a quality assurance testing program be implemented during construction to observe compliance with the design concepts, plans, specifications and design recommendations and to allow design changes in the event that subsurface conditions found differ from those anticipated prior to the start of construction. S.W.COLE should be on-site to observe subgrades prior to the placement of fill or foundations. S.W.COLE would be pleased to provide a scope of services and budget to observe the foundation construction and subgrade preparation, as well as to provide field and laboratory testing services at the appropriate time.

6.0 CLOSURE

The data and conclusions contained in this report are based on the findings from the explorations and laboratory and field testing made to date to provide a general characterization of the overall site conditions. Geotechnical parameters and recommendations have been provided for consideration of the foundation designer, other design professionals and contractors involved with the project. The foundation designer and other designers or contractors will need to select appropriate parameters.

It has been a pleasure to be of assistance to you with this phase of your project. If you have any questions please do not hesitate to contact us. We look forward to being of assistance to you on the next phase of your project.

Sincerely,

S. W. Cole Engineering, Inc.

STATE OF MAINE
CLIFFORD R. LIPPITT
GE292
GEOLOGIST

Clifford R. Lippitt, C.G.
Senior Geologist

STATE OF MAINE
MICHAEL A. ST. PIERRE
No. 12519
LICENSED
PROFESSIONAL ENGINEER

Michael A. St. Pierre, P.E.
Geotechnical Engineer

MAS-CRL:pfk-gwb

Attachment A
Limitations

This report has been prepared for the exclusive use of Reed & Reed, Inc. for specific application to the Proposed Blue Sky West Wind Power Project located in Bingham, Brighton Plantation, and Mayfield Township, Maine. S. W. Cole Engineering, Inc. has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

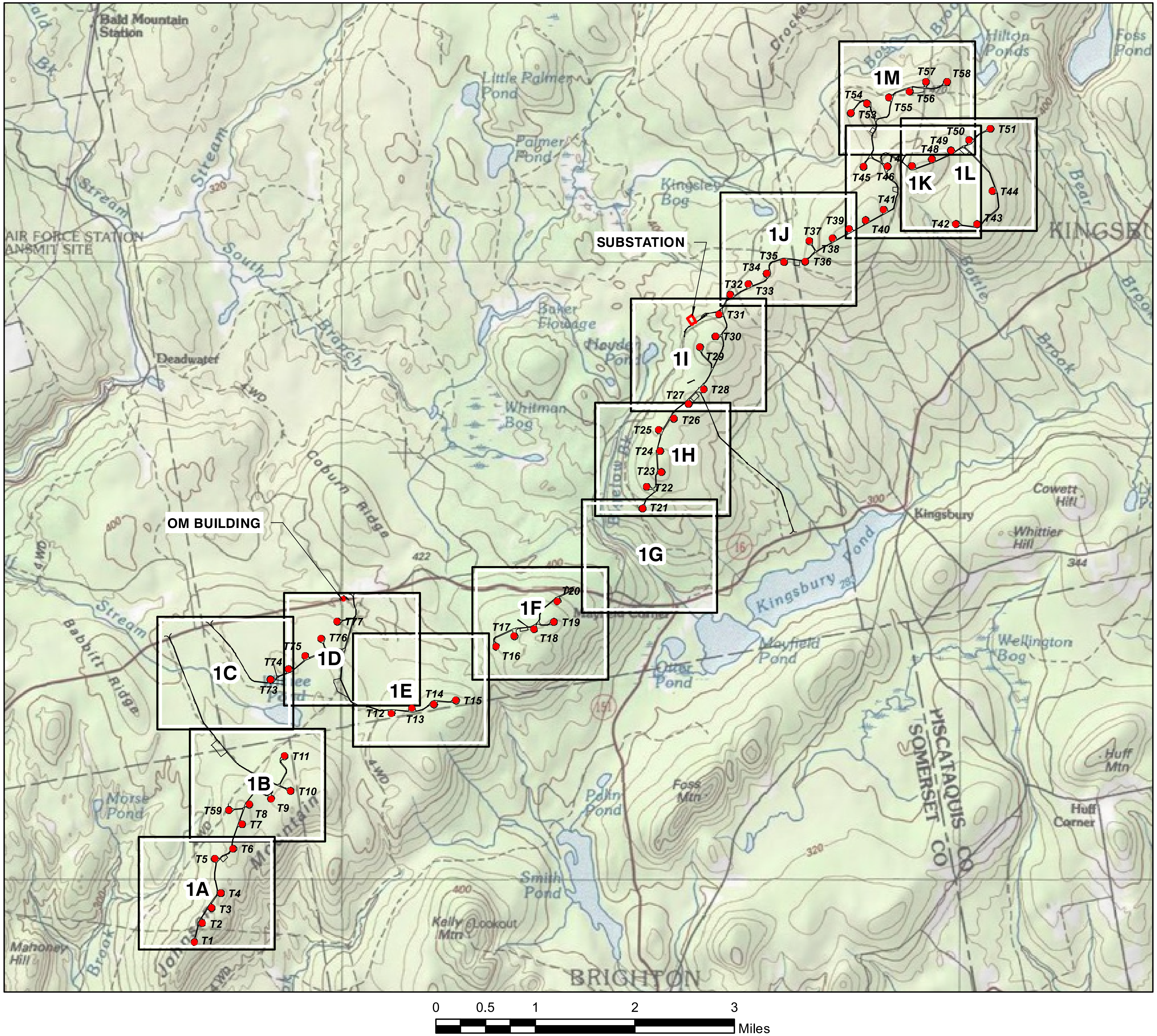
The soil profiles described in this report are intended to convey general trends in subsurface conditions. The boundaries between strata are approximate and are based upon interpretation of exploration data and samples.

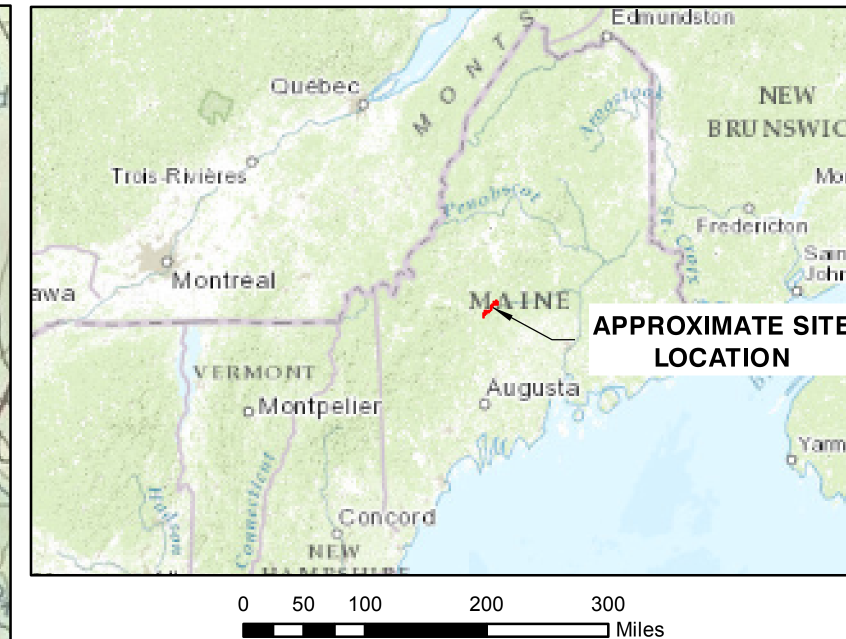
The analyses performed during this preliminary investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions will occur between explorations and may not become evident until construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and to review the recommendations of this report.

Observations have been made during the explorations to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

S. W. Cole Engineering, Inc.'s scope of services has not included the investigation, detection, or prevention of any Biological Pollutants at the project site or in any existing or proposed structure at the site. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and the byproducts of any such biological organisms.

Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S. W. Cole Engineering, Inc. should review such changes as they relate to analyses associated with this report. Recommendations contained in this report shall not be considered valid unless the changes are reviewed by S. W. Cole Engineering, Inc.





LEGEND

EXPLORATION LOCATION PLAN SHEET(S)

NOTES:

1. BASE MAP CREDITS: SOURCES: ESRI, DELORME, NAVTEQ, TOMTOM, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSTOPO, AND THE GIS USER COMMUNITY COPYRIGHT: © 2013 NATIONAL GEOGRAPHIC SOCIETY, I-CUBED.

2. TURBINE LOCATIONS PROVIDED BY REED & REED, OCTOBER 2013.

S.W.COLE ENGINEERING, INC.

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SITE LOCATION MAP AND INDEX

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

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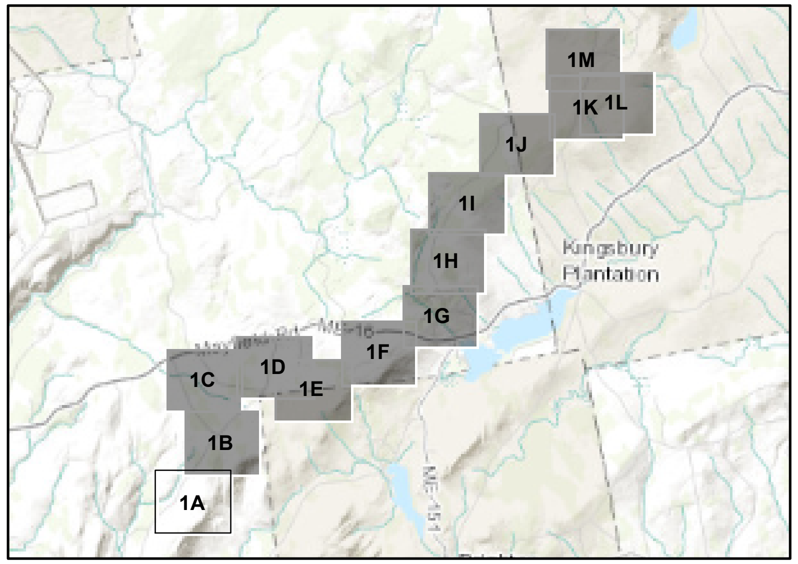

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Scale in Feet

S.W.COLE ENGINEERING, INC.

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EXPLORATION LOCATION PLAN - 1A

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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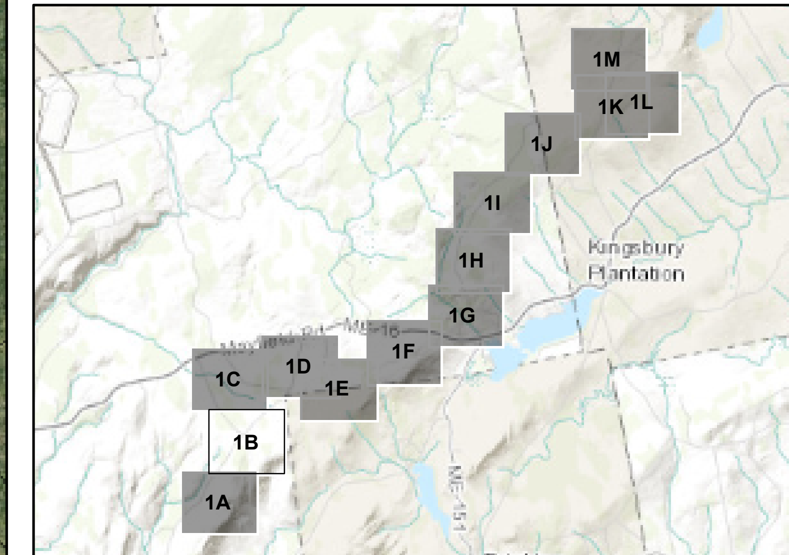

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Scale in Feet

S.W.COLE ENGINEERING, INC.

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EXPLORATION LOCATION PLAN - 1B

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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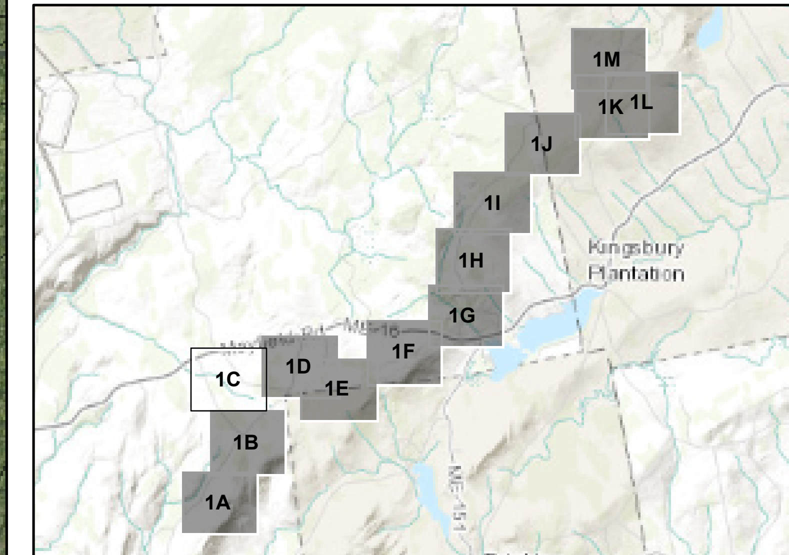

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Scale in Feet

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EXPLORATION LOCATION PLAN - 1C

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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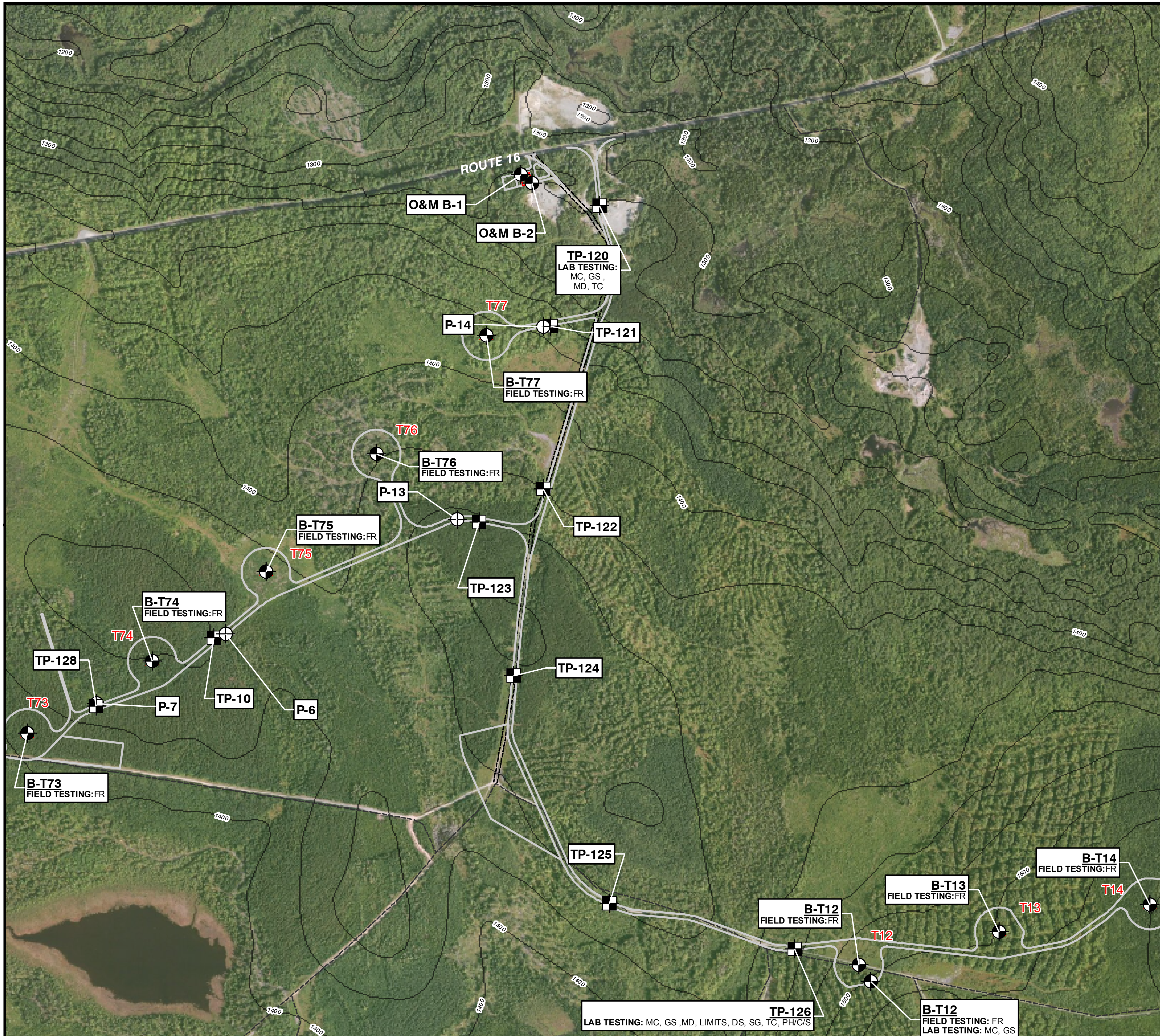

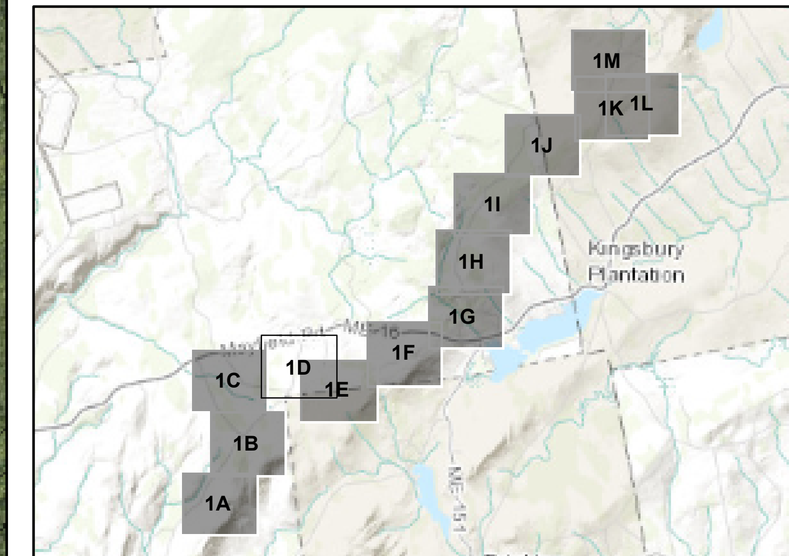

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Scale in Feet

S.W.COLE ENGINEERING,INC.

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EXPLORATION LOCATION PLAN - 1D

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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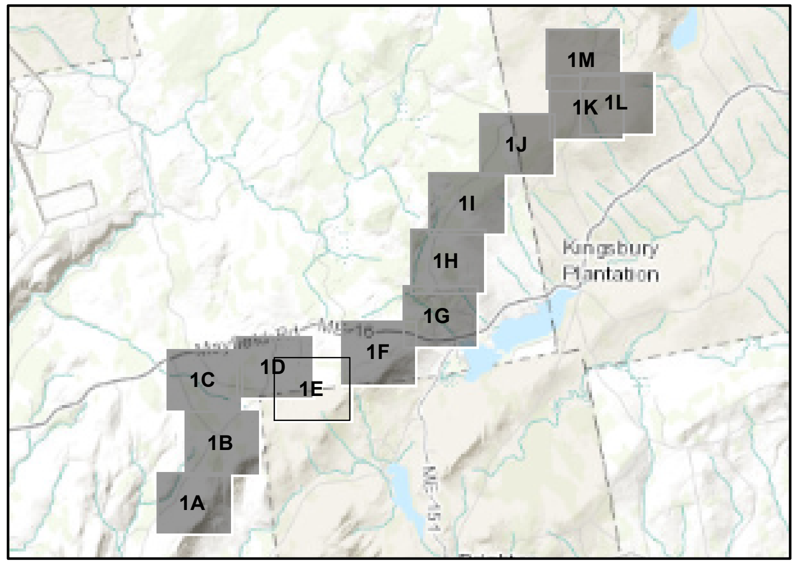

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Scale in Feet

S.W.COLE ENGINEERING, INC.

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EXPLORATION LOCATION PLAN - 1E

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

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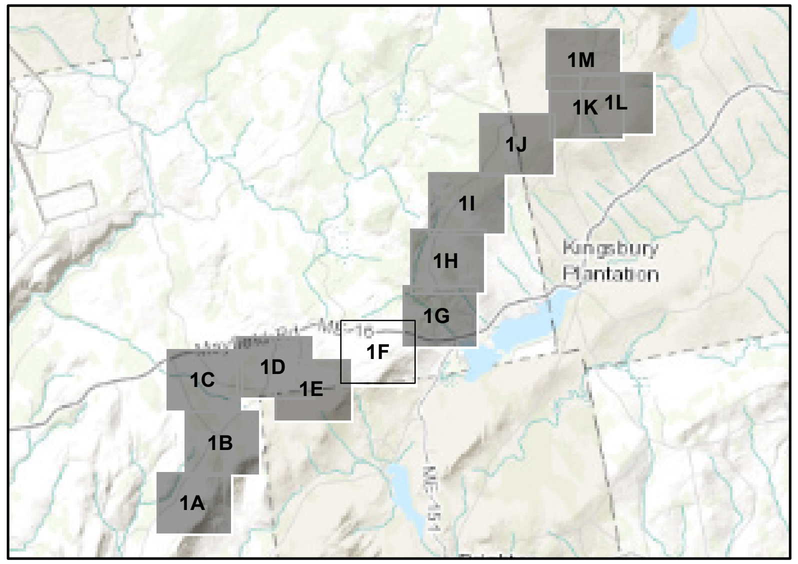

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Scale in Feet

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EXPLORATION LOCATION PLAN - 1F

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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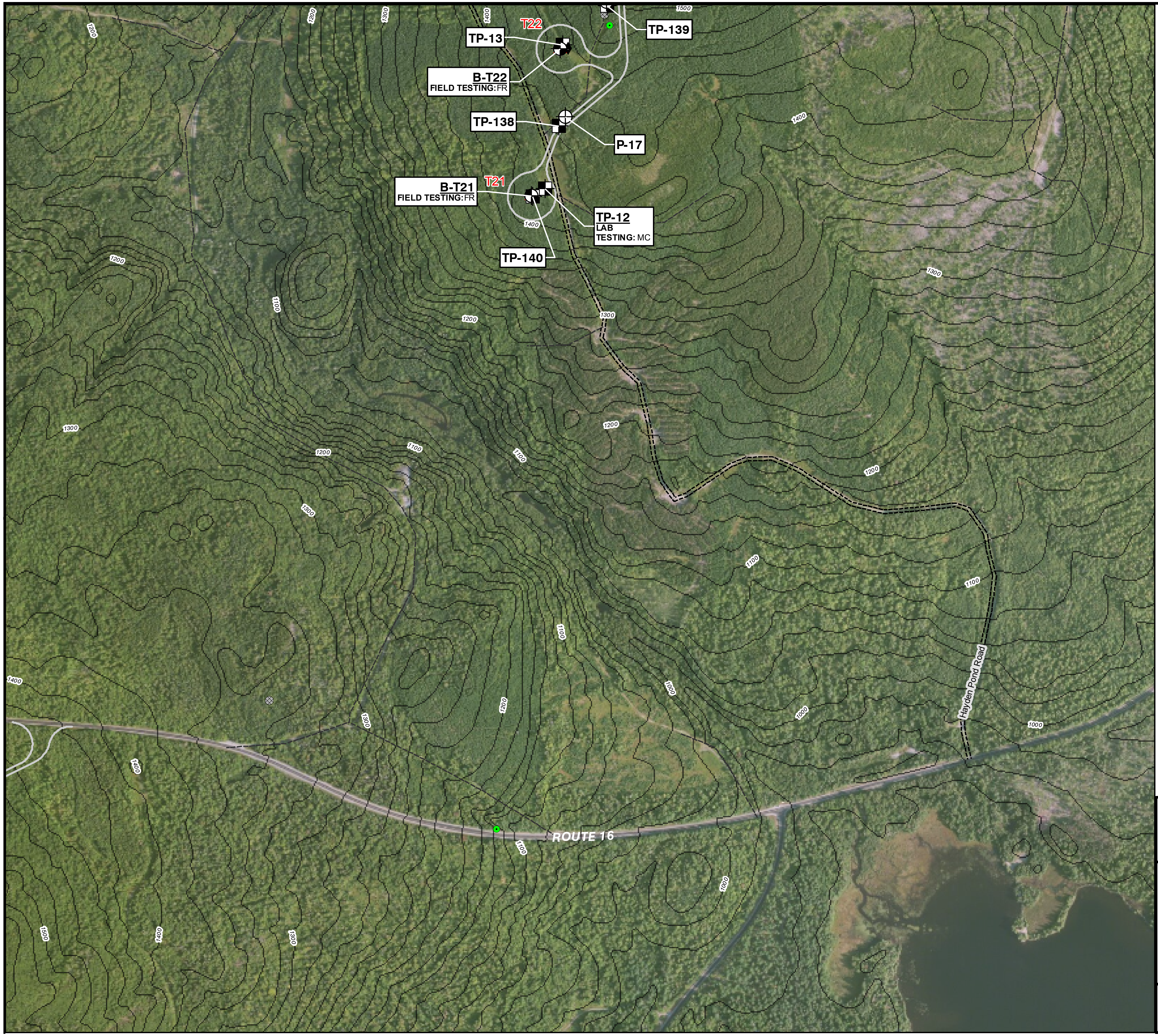

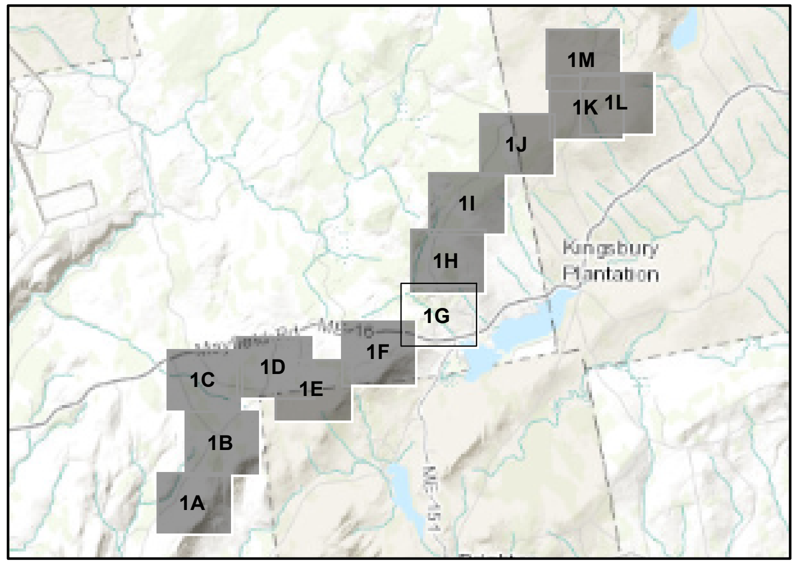

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Scale in Feet

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EXPLORATION LOCATION PLAN - 1G

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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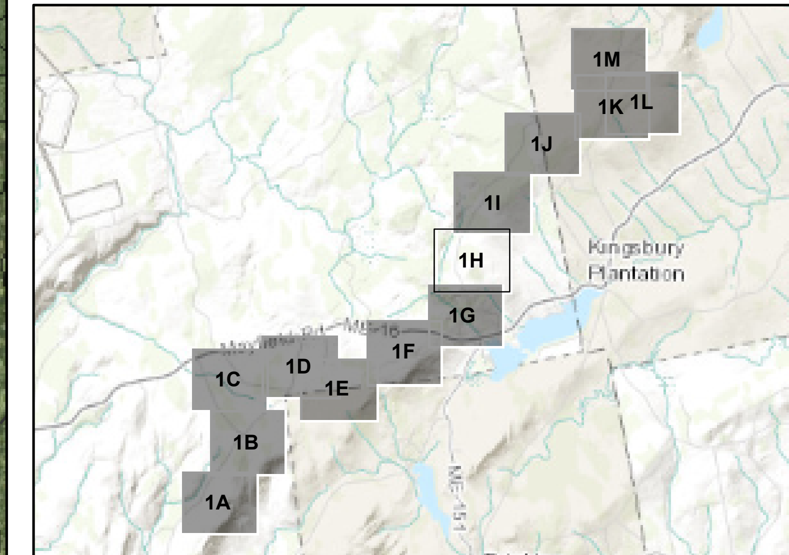

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Scale in Feet

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EXPLORATION LOCATION PLAN - 1H

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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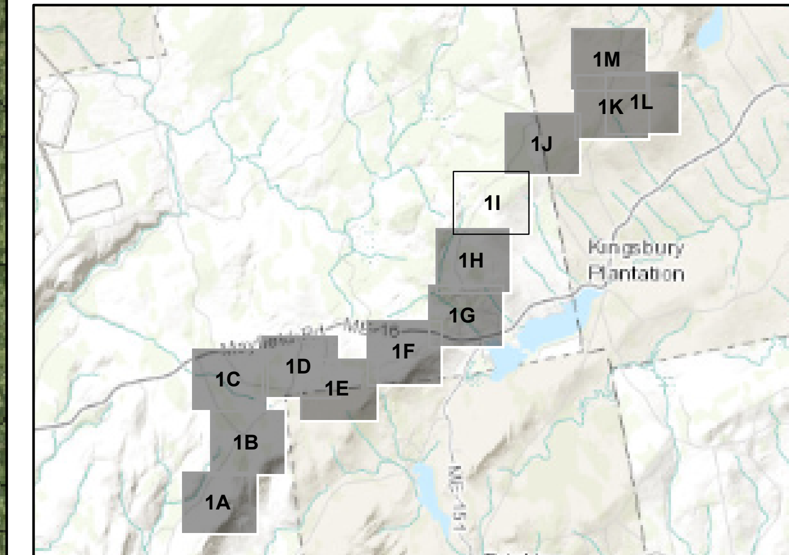

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Scale in Feet

S.W.COLE ENGINEERING, INC.

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EXPLORATION LOCATION PLAN - 1I

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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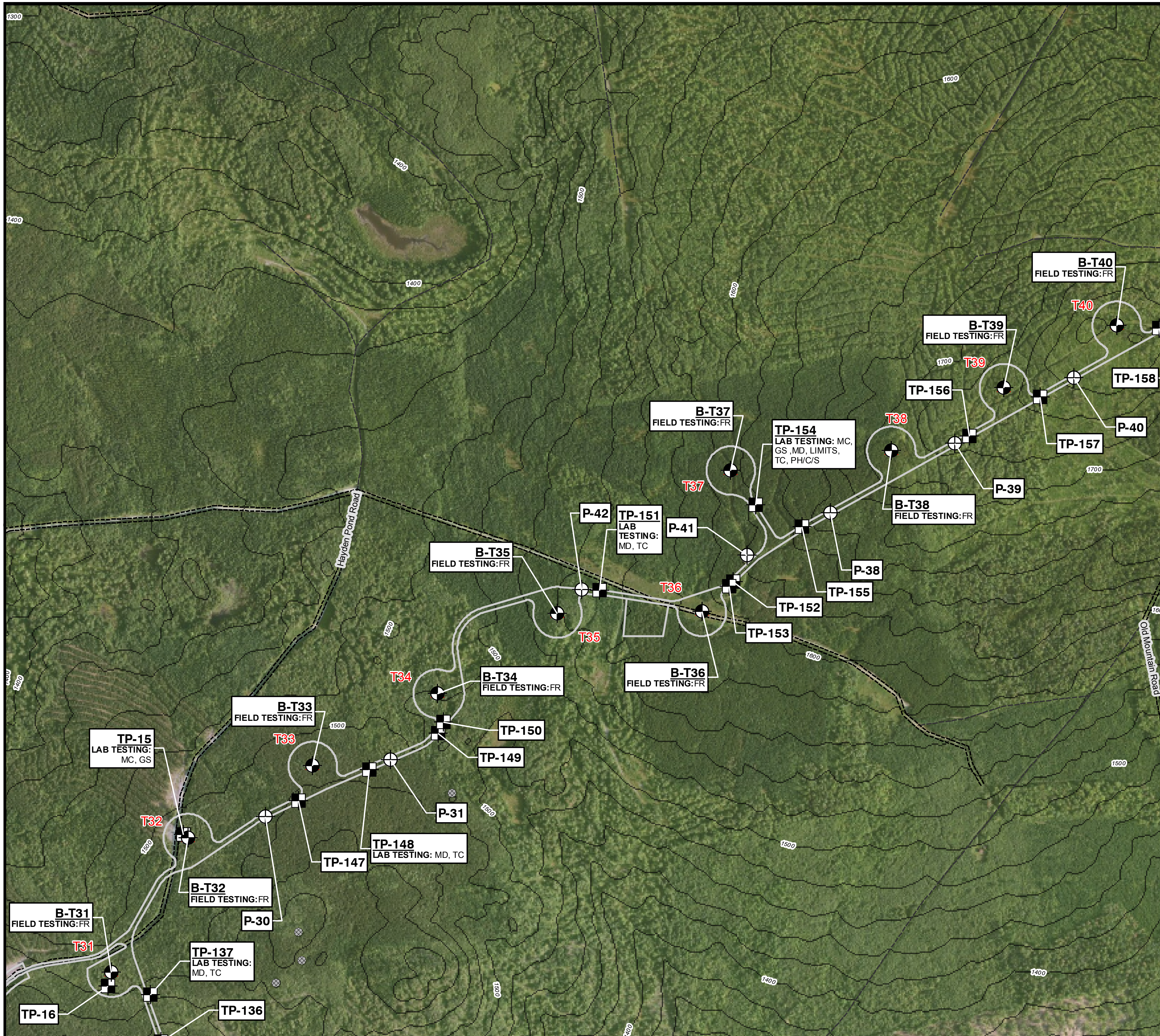

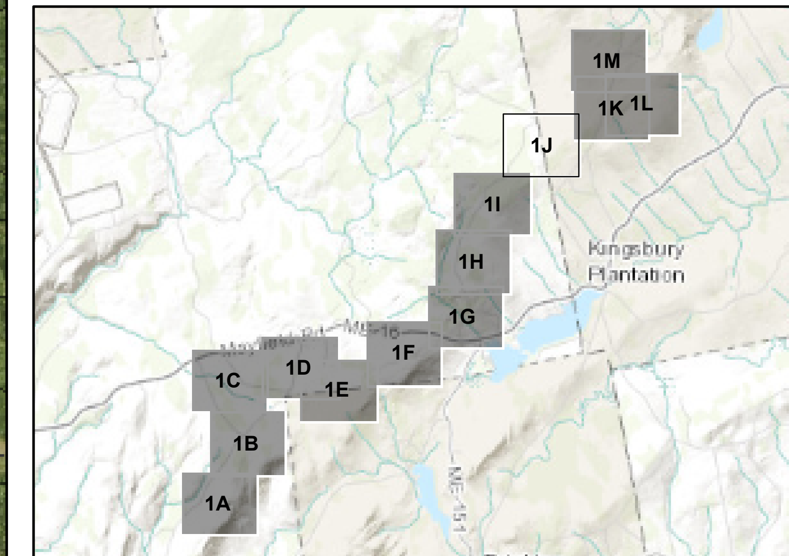

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Scale in Feet

S.W.COLE ENGINEERING, INC.

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EXPLORATION LOCATION PLAN - 1J

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

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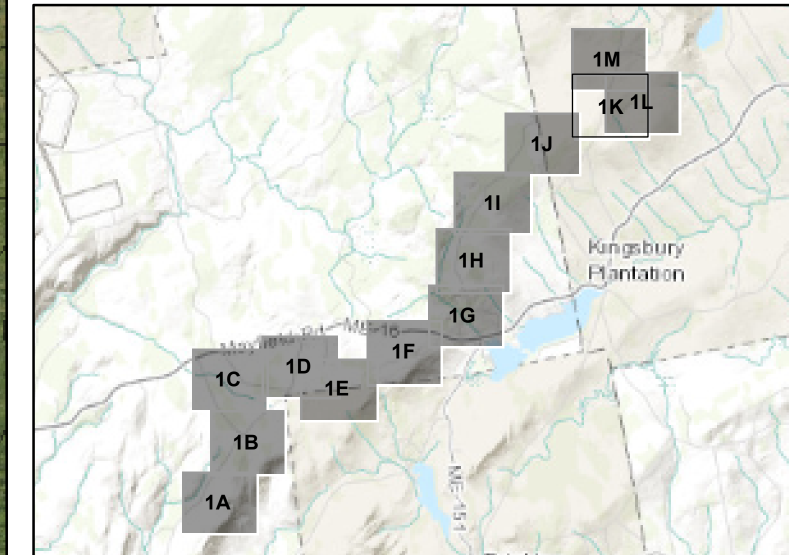

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Scale in Feet

S.W.COLE ENGINEERING, INC.

REED & REED

EXPLORATION LOCATION PLAN - 1K

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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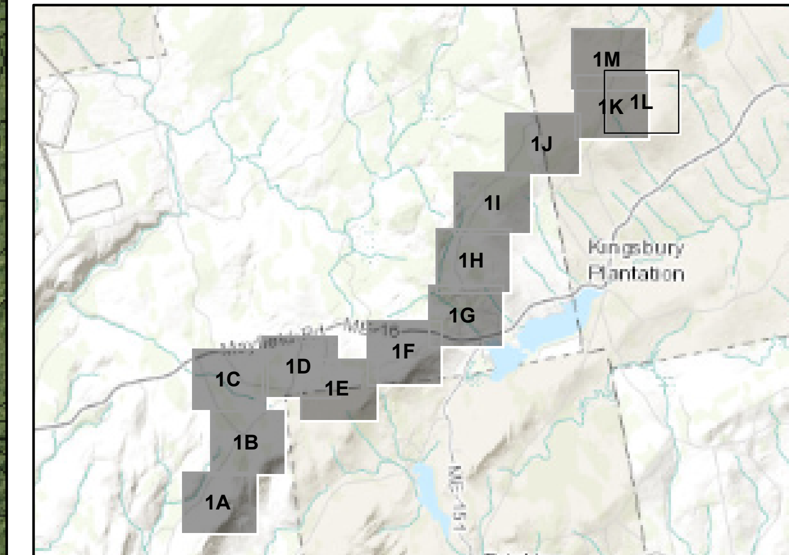

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Scale in Feet

S.W.COLE
ENGINEERING, INC.

REED & REED

EXPLORATION LOCATION PLAN - 1L

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
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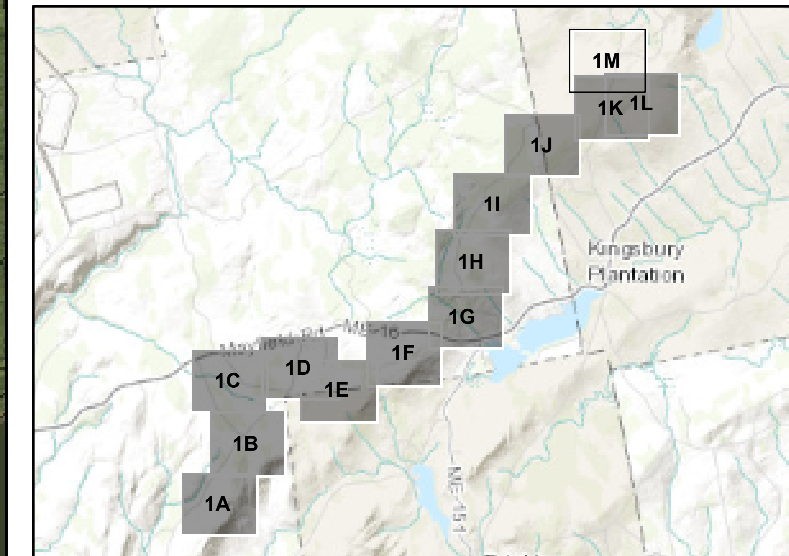

N
TRUE

NOTES:

1. SEE SHEET 1N, FOR EXPLORATION LOCATION PLAN NOTES, LEGEND AND KEY TO TESTING TERMS, AND GIS DATA REFERENCES

600 0 600 1,200

Scale in Feet

S.W.COLE
ENGINEERING,INC.

REED & REED

EXPLORATION LOCATION PLAN - 1M

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	1"=600'
Date:	01/30/2014	Sheet	1M

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NOTES:

1. PROPOSED TURBINE LOCATIONS PROVIDED BY REED & REED, OCTOBER 11, 2013.
2. PROPOSED EDGE OF ROAD, SUBSTATION AND OM BUILDING LOCATIONS PROVIDED BY DELUCA HOFFMAN ASSOCIATES, DATED MARCH 12, 2013.
3. EXPLORATION LOCATION PLAN IMAGERY FROM ESRI, DIGITALGLOBE, GEOEYE, I-CUBED, USDA, USGS, AEX, GETMAPPING, AEROGRID, IGN, IGP, SWISSTOPO, AND THE GIS USER COMMUNITY.
4. LOCUS MAP ESRI, DELORME, NAVTEQ, TOMTOM, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSTOPO, AND THE GIS USER COMMUNITY
5. TOPOGRAPHY FROM U.S. GEOLOGICAL SURVEY (USGS), MAINE OFFICE OF GEOGRAPHIC INFORMATION SYSTEMS (MEGIS) (ED.), PUBLISHED APRIL 30, 2000, ACCESSED AUGUST, 2012.
6. THE BORINGS WERE LOCATED IN THE FIELD BY GPS SURVEY BY S.W. COLE ENGINEERING, INC, BASED ON COORDINATES PROVIDED BY REED & REED, USING A MAPPING GRADE TRIMBLE GPS RECEIVER.
7. THE TEST PIT LOCATIONS WERE SELECTED BY S.W. COLE ENGINEERING, INC. AND REED & REED AND LOCATED IN THE FIELD BY GPS SURVEY USING A MAPPING GRADE TRIMBLE GPS RECEIVER.
8. APPARENT OUTCROP LOCATIONS WERE OBSERVED DURING OUR 2010 AND 2013 INVESTIGATIONS. THE APPARENT OUTCROPS WERE LOCATED IN THE FIELD BY GPS SURVEY BY S.W. COLE ENGINEERING, INC. USING A MAPPING GRADE TRIMBLE GPS RECEIVER. ADDITIONAL UNMARKED OUTCROPS EXIST IN THE PROJECT AREA.
9. THIS PLAN SHOULD BE USED IN CONJUNCTION WITH THE ASSOCIATED S.W. COLE ENGINEERING, INC. DOCUMENT ENTITLED "PRELIMINARY GEOTECHNICAL ENGINEERING SERVICES, PROPOSED BLUE SKY WEST WIND POWER PROJECT, SOMERSET AND PISCATAQUIS COUNTIES, BINGHAM, MAYFIELD TOWNSHIP AND KINGSBURY PLANTATION, MAINE," DATED JANUARY 09, 2014.
10. THE PURPOSE OF THIS PLAN IS ONLY TO DEPICT THE LOCATION OF EXPLORATIONS AND ROCK OUTCROPS IN RELATION TO EXISTING CONDITIONS AND PROPOSED CONSTRUCTION AND IS FOR PLANNING PURPOSES ONLY, IT IS NOT TO BE USED FOR CONSTRUCTION.

KEY TO LABORATORY TESTING TERMS

GS	GRAIN SIZE
GS/H	GRAIN SIZE WITH HYDROMETER
MC	MOISTURE CONTENT
MD	LABORATORY MOISTURE DENSITY
LIMITS	ATTERBERG LIMITS
DS	LABORATORY DIRECT SHEAR
PH/C/S	LABORATORY PH / CHLORIDES / SULFATES
TC	LABORATORY THERMAL CONDUCTIVITY

KEY TO FIELD TESTING TERMS

FR	FIELD RESISTIVITY
SV	FIELD SHEAR WAVE VELOCITY

LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE TEST PIT LOCATION
- APPROXIMATE PROBE LOCATION
- APPROXIMATE REISTIVITY TEST LOCATION
- APPARENT OUTCROP LOCATIONS 2010 AND 2013 OBSERVATIONS
- BEDROCK SAMPLE LOCATIONS 2010
- APPROXIMATE GEO-HAZARD LOCATION
- PROPOSED EDGE OF GRAVEL
- PROPOSED O&M AND SUBSTATION
- PROPOSED TURBINE LOCATION FROM REED & REED 10/11/2013
- GRAVEL ROADS FROM FIRST WIND
- 4WD WOODS ROADS FROM FIRST WIND

S.W.COLE ENGINEERING, INC.

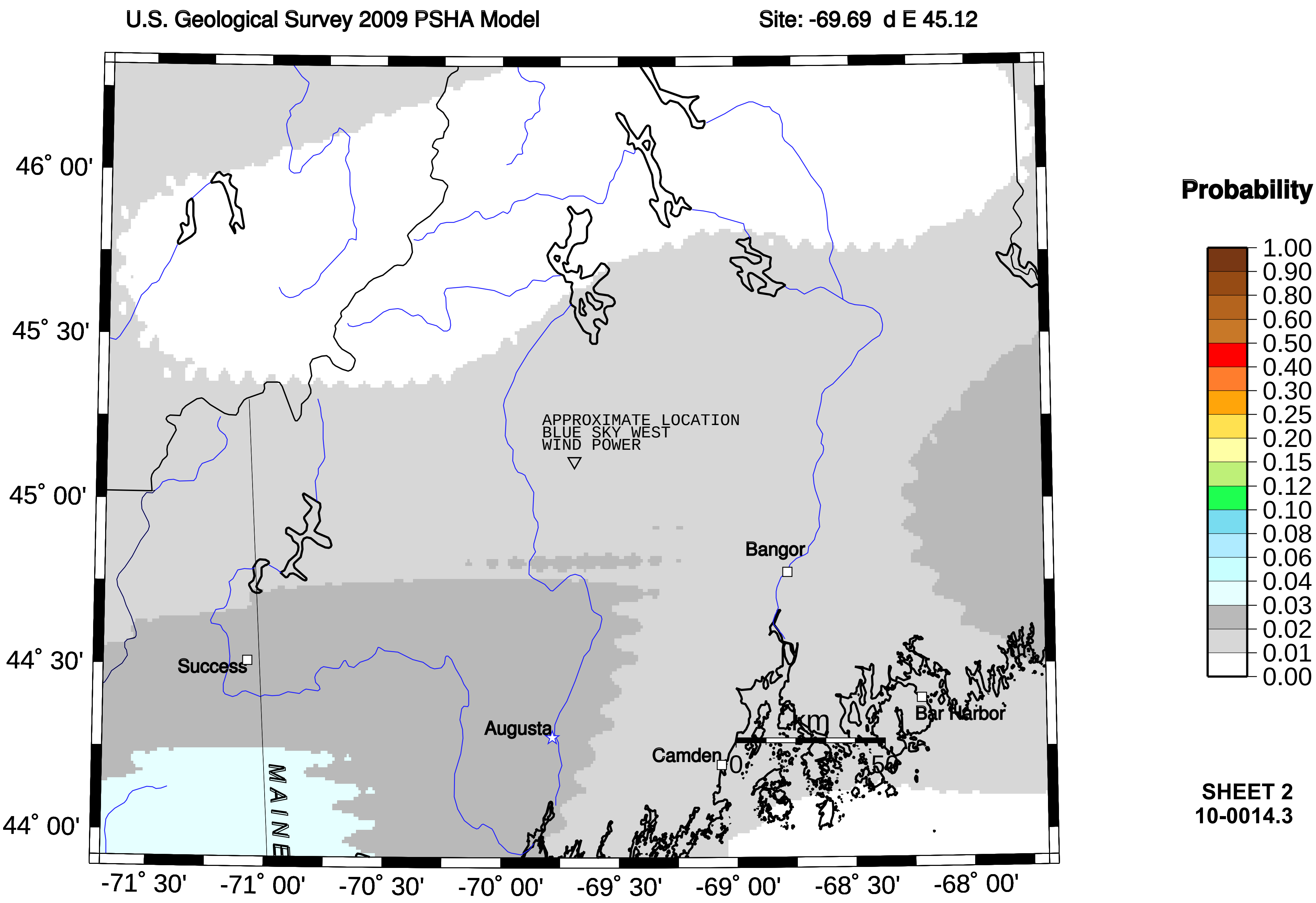
REED & REED

NOTES, LEGEND AND KEY TO TESTING

PROPOSED BLUE SKY WEST WIND POWER PROJECT
BINGHAM, MAYFIELD TOWNSHIP
AND KINGSBURY PLANTATION, MAINE

Job No.	10-0014.3	Scale	N.T.S.
Date:	01/30/2014	Sheet	1N

Probability of earthquake with M > 5.0 within 50 years & 50 km



SHEET 2
10-0014.3

GMT 2013 Dec 30 15:42:25

Earthquake probabilities from USGS OFR 08-1128 PSHA. 50 km maximum horizontal distance. Site of interest: triangle. Epicenters mb>5 black circles; rivers blue.