Appendix C:

East Cable Terminal Geotechnical Report

REVISED GEOTECHNICAL FOUNDATION REPORT
for a
230kV, 30 MVAR SHUNT REACTOR
to be installed at
CHUGACH ELECTRIC ASSOCIATION'S SIX-MILE EAST CABLE TERMINAL SUBSTATION JOINT BASE ELEMENDORF-RICHARDSON, ALASKA

Prepared for:

Electric Power Systems, Inc.
3305 Arctic Blvd., Suite 201
Anchorage, AK 99503-4575

Prepared by:

Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing

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MAY 2023

May 12, 2023 NGE-TFT Project # 6238-21(R1)

Electric Power Systems, Inc.
3305 Arctic Blvd., Suite 201
Anchorage, AK 99503-4575

Attn: Tim Conrad, P.E.

RE: REVISED GEOTECHNICAL FOUNDATION REPORT FOR THE SITE OF A PLANNED 230KV, 30 MVAR SHUNT REACTOR TO BE INSTALLED AT CHUGACH ELECTRIC ASSOCIATION'S SIX-MILE EAST CABLE TERMINAL – JOINT BASE ELEMENDORF-RICHARDSON, ALASKA

Tim,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed a geotechnical engineering assessment of the site of the planned 230kV, 30 MVAR Shunt Reactor which is to be installed at Chugach Electric Association's Six-mile East Cable Terminal Substation. We have revised our initial report to include recommendations for H-Piles. The soils underlying the project site consist of medium dense well graded gravel deposits that are suitable for supporting the proposed site improvements provided that proper engineering controls are incorporated into the design and construction of the proposed site improvements.

In the following report we provide a summary of our field and laboratory testing programs, as well as provide our conclusions regarding the suitable of the project site to support the proposed the proposed site improvements. We also provide recommendations for the design and construction of the proposed site improvements, including recommendations for the design of the foundations.

We greatly appreciate the opportunity to provide you with our professional service. Please contact us directly with any questions or comments you may have regarding the information that we present in this report, or if you have any other questions, comments, and/or requests.

Sincerely,
Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing

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1.0 INTRODUCTION

In this report, we (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) present the results of a geotechnical engineering assessment that we conducted at Chugach Electric Association's (CEA) Six-mile East Cable Terminal (ECT) Substation; which we hereafter refer to as "the project site". We provided our professional service in accordance with our service fee proposal #21-215 which we submitted to our client, Electric Power Systems, Inc. (EPS), on October 26, 2021. EPS authorized our proposed scope of service on November 23, 2021 via Purchase Order No. 210478-001.

EPS contracted us to evaluate the subsurface conditions adjacent to an existing shunt reactor located at the project site to aid in the design and construction of a foundation for a new shunt reactor which is to replace the existing reactor. In the following report, we present the findings of our subsurface exploration and laboratory testing programs, as well as our conclusions regarding the suitability of the existing subgrade to support the planned replacement reactor. We also present our engineering recommendations for the design and construction of the foundation for the planned replacement reactor.

2.0 PROJECT OVERVIEW

As we detail in Figure 1 of this report, the project site is located on Joint Base Elmendorf-Richardson (JBER), approximately 1.3 miles due north of Six-Mile Reservoir. The project site is the location of the CEA Six-Mile ECT Substation, which consists of an earthen fill pad approximately 0.5 acres in area which supports an array of various electrical transmission equipment. We have included a plot plan of the Six-Mile ECT Substation in Figure 2 of this report for reference (from CEA drawing SMET-SS-000 – Sheet 1).

The existing plot plan suggests that the earthen pad on which the substation is sited was cut into the existing slope, with cut depths beneath the substation pad ranging from approximately 0 to 10 feet and the deepest cut along the east side of the pad. Some fill (either imported or re-worked native material) was likely used to level the site and bring the pad to finished grade.

CEA plans to replace an existing shunt reactor that is located at the southeast corner of the project site with a new 230 kV, 30 MVAR shunt reactor. The new reactor will be located within the footprint of the existing reactor (once it has been removed). The new reactor will weigh approximately 85 tons and occupy an area of approximately 17-ft by 18-ft.

The existing reactor bears upon a poured concrete slab, however, we do not have any information regarding the subgrade components/configuration/design of the existing reactor foundation slab. The foundation system for the planned replacement reactor has yet to be determined but will likely consist of either a conventional shallow foundation (e.g., reinforced poured concrete slab) or a deep foundation system (e.g., driven steel piling, etc.), or some combination of both. It is

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understood that the new foundation will incorporate a fluid containment system as a mitigation for potential spills.

3.0 SITE CHARACTERIZATION ACTIVITIES

3.1 Subsurface Exploration

We conceived, coordinated, and directed a subsurface exploration program at the project site in an effort to characterize the subsurface conditions adjacent to the existing reactor as they currently exist. A qualified representative from our office was present onsite during the entire exploration program to select the exploration locations, direct the exploration activities, log the geology of each exploration, and collect representative samples for further identification and laboratory analysis.

We subcontracted Discovery Drilling, Inc (our drilling contractor) to provide the necessary geotechnical exploration services. Under our direction, our drilling contractor advanced a total of two soil borings at the project site on January 7, 2022 to depths ranging from approximately 20 to 40 feet bgs. We have plotted the approximate location of both soil borings in Figure 3 of this report. Prior to any drilling, we subcontracted Alaska Directional to excavate an approximately 12-inch diameter pilot hole (to a depth of approximately five feet bgs) at each at each boring location using a vacuum truck and high-pressure water jetting to ensure that there were no buried utility conflicts.

Our drilling contractor performed a Modified Penetration Test (MPT) at regular intervals during the drilling of each borehole. A MPT can be used to assess the consistency of a soil interval and to collect representative soil samples. A MPT is performed by driving a 3.0-inch O.D. (2.4-inch I.D.) split-spoon sampler at least 18 inches past the bottom of the advancing augers with blows from a 340-lb drop-hammer, free-falling 30 inches onto an anvil attached to the top of the drill rod stem. Our field representative recorded the hammer blows required to drive the split-spoon sampler the entire length of each sample interval, or until sampler refusal was encountered. We have provided the field blow count data for each sample interval (in six-inch increments) on the graphical borehole logs contained in Appendix A of this report.

We corrected the field blow count data for both boreholes for standard confining pressure, drill rod length, and drop-hammer operation procedure to estimate a standard $(N_1)_{60}$ value for each sample interval. $(N_1)_{60}$ values are a measure of the relative density (compactness) and consistency (stiffness) of cohesionless or cohesive soils, respectively. Our estimate of the $(N_1)_{60}$ values is based on the drop-hammer blows required to drive the spilt-spoon sampler the final 12-inches of an 18-inch MPT. We have provided our estimated $(N_1)_{60}$ values for each sample interval on the graphical borehole logs contained in Appendix B of this report. The automatic drop-hammer that our drilling contractor used for this project is not standard, so we applied a correction factor of 1.1 to the $(N_1)_{60}$ values to account for the efficiency of the automatic drop-hammer used. We have provided a graphical plot of the field blow count corrections that we used to correct for confining pressure and drill rod length in Figure 3 of this report.

Our field representative photographed each split-spoon sample that our drilling contractor collected during our exploration program and we have included these photographs in Appendix A of this report. Our field representative sealed each sample that they collected during our subsurface exploration program inside of an air-tight bag, to help preserve the moisture content of each sample, and then submitted each sample to our laboratory for further identification and analysis.

Once the exploration activities were complete, we directed our drilling contractor to backfill the annulus of each borehole with its respective drill cuttings and/or appropriate amounts of granular fill. Prior to backfill, we directed DDI to install one-inch diameter, open-ended PVC pipe from the ground surface down to the bottom of borehole B2 in order to provide a conduit (i.e., monitoring wells) for future groundwater level monitoring.

4.0 LABORATORY TESTING

We collected a total of 13 soil samples from the two soil borings that DDI advanced at the project site and submitted all of the soil samples to our laboratory for further identification and geotechnical analysis. We tested select soil samples in accordance with the respective ASTM standard test methods including:

- moisture content analysis (ASTM D-2216);
- determination of fines content (a.k.a. P200 – ASTM D-1140);
- grain size sieve and hydrometer analysis (ASTM D-6913 & D-7928);
- Atterberg limits (ASTM D-4318).

It is important to note that ASTM test method D-6913 requires that any soil sample specimen which is to be submitted for gradational analysis (by ASTM D-7928 or other methods) must satisfy a minimum mass requirement based on the maximum particle size of the sample specimen. Split-spoon sampling techniques (standard or modified), as well as other small-diameter soil sampling techniques (e.g., macro-core, etc.), typically recover anywhere from approximately 1 to 10 pounds of sample specimen. The amount of sample specimen recovered can be influenced by (amongst other variables) the soil gradation, soil density, sample interval, sampler tooling, and soil moisture content. As a result, samples of coarse-grained soils (with individual soil particles greater than approximately 0.75 inches in diameter) collected with small-diameter sampling methods (e.g., split-spoons, macro-core, etc.) may not meet the minimum mass requirement specified by Table 2 of ASTM D-6913. This may result in gradational and frost classification results which are not representative of the actual (i.e., in-situ) soil gradation and/or frost classification. The use of small-diameter sampling devices in coarse-grained soils (e.g., sand and gravel) can result in the collection of unrepresentative samples due to: the exclusion of oversized particles (larger than the opening of the sampler) from the sample; and the mechanical breakdown/degradation of coarse-grained particles by the sampling process (producing an unrepresentative increase in smaller-diameter particles in the sample). Both of these sampling biases can skew laboratory test results towards the fine-grained end of the gradational spectrum.

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The laboratory test results, along with the observations we made during our subsurface exploration efforts, aid in our evaluation of the subsurface conditions at the project site and help us to assess the suitability of the subsurface materials located at the project site to support the planned replacement reactor. We have included the results of our geotechnical laboratory analyses on the graphical exploration logs contained in Appendix A of this report and on the laboratory data sheets contained in Appendix B of this report.

5.0 DESCRIPTION OF SUBSURFACE CONDITIONS

We compiled our field observations with the results from our laboratory analyses to produce graphical logs of each subsurface exploration (Appendix A). The graphical exploration logs depict the subsurface conditions that we identified at each exploration location and help us to interpret/extrapolate the subsurface conditions for areas adjacent to, and immediately surrounding, each exploration location across the project site.

5.1 General Subsurface Profile

In general, the project site is underlain by a varying thickness of gravel which contains varying fractions of silt and sand-sized particles and generally classifies as a well-graded gravel with silt and sand (GW-GM) on the United Soil Classification System (USCS). Particles within the gravel material range up to three inches in diameter, and trace cobbles up to four inches in diameter were visible in the annulus of each pilot hole. In the area immediately surrounding (and underlying) the existing/proposed reactor the gravel material appears to be reworked (and placed as fill), and this fill likely ranges in thickness from approximately 1 to 2 feet; although the boundary between any fill and the underling native gravel deposits was difficult to discern in the field. The gravel deposits (below a depth of approximately five feet bgs) exhibit a moderately dense consistency and our laboratory testing suggest that these deposits classify as F1 on the US Army Corps of Engineers (USACOE) frost classification scale.

The gravel deposits are underlain by deposits of sand with varying fractions of silt and gravel-sized particles which classifies as a poorly-graded sand (SP to SP-SM) by the USCS. These sand deposits extend to depths of approximately 14 to 15 feet bgs and exhibit a moderately dense consistency.

The sand/gravel deposits are underlain by interbedded deposits of silt and silty sand which extend to a depth of at least 35 feet bgs. These silt/silty sand deposits exhibit a moderately stiff/dense consistency and the silt deposits exhibit some plasticity.

The silt/silty sand deposits appear to be underlain by deposits of silty gravel and sand which are consistent with local glacial till deposits. The silty gravel deposits contain gravel particles ranging up to at least 2.5 inches in diameter (and may contain varying percentages of larger gravel/cobble/boulder-sized particles) and exhibit a relatively dense consistency.

5.2 Groundwater

We observed indications of groundwater in both soil borings during our exploration efforts at an estimated depth of approximately six feet bgs. We measured the static groundwater level in soil boring B2 on January 18, 2022 and determined it to be 3.79 feet bgs. The groundwater appears to be perched above the underlying silt deposits and likely occurs at similar depths in the vicinity of the existing/proposed reactor, and likely deepens to the west across the project site.

5.3 Frozen Soils

We observed some seasonal ground frost during our subsurface exploration effort down to depths of approximately three feet bgs, and seasonal ground frost depths likely vary across the project site depending upon variations in seasonal snow cover and groundwater levels. We did not, however, observe any indications of permafrost and we do not expect permafrost to occur anywhere across the project site.

6.0 ENGINEERING CONCLUSIONS

6.1 General Site Conclusions

Based on the findings of our field and laboratory testing efforts, it is our conclusion that the existing gravel soils which we observed across the project site are generally suitable to support the proposed improvements; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

6.2 Earthworks

Earthworks conducted at the project site should be relatively minimal. Any excavated coarse-grained material may be reused on-site as structural fill, assuming that the material is free of any organic material (or other deleterious debris) and that the material is compactable. We discuss our earthworks in more detail in Sections 7.1 and 8.1 of this report.

6.3 Foundations

While the gravel samples resulted in some frost susceptibility (F1 on the USACOE frost classification scale) the shallow ground-water table (approximately 4 feet bgs) reduces the depth of freeze at the project site. We do not expect the soils below the ground water table to freeze. As such, a conventional pad foundation is suitable to support the proposed improvements assuming that the subsurface conditions are similar across the project site and that our recommendations are followed.

An alternative to conventional, poured-concrete foundations is a deep foundation system which transfers the foundation loads down through the active layer to the dense and continuously thawed bearing soils, and protects the foundation from seasonal frost-related ground movements. We detail our design and construction recommendations for both conventional poured-concrete and driven steel pipe pile foundations in greater detail in Sections 7.2/8.2 and 7.3/8.3 of this report.

6.4 Foundation Settlements

Settlements for shallow foundations should be within tolerable limits, provided that they are placed directly onto the undisturbed gravel soils (or properly placed structural fill located directly above the undisturbed gravel soils). We anticipate a total settlement for shallow concrete foundations placed on either the undisturbed gravel soils and/or or structural fill placed above the undisturbed gravel soils (as we discuss in Section 7.1 of this report) to be less than three-quarters (3/4) of an inch, with differential settlements comprising about one-half (1/2) of the total anticipated settlement. Settlement amounts could increase substantially if the structural fill material used to bring any foundation pads to grade is not properly compacted. Most of the settlements should occur as the structure loads are applied, such that additional long-term settlements should be relatively small and within tolerable limits. Settlements for deep foundations (as we discuss in Section 7.3 of this report) should be negligible.

6.5 Seismic Design Parameters

The International Building Code (IBC) 2018 is slowly being adopted by various state and local governmental regulatory agencies throughout Alaska. However, the on-line seismic site design query tool that we use to estimate seismic site design parameters has not been updated from IBC 2015 to IBC 2018. Additionally, IBC 2018 does not explicitly state that any changes have been made to the 2015 IBC seismic design code for locations with site specific geotechnical information. As such, we feel comfortable using the seismic site design parameters using IBC 2015.

We have assumed that the International Building Code (IBC) 2018 will be used for the design of the proposed structure and that the seismic risk category for the proposed structure will be Category II. The seismic site classification for the project site is D based on the $(N_1)_{60}$ values that we calculated for the subgrade soils that occur at the project site. We utilized the Structural Engineers Association of California (SEAOC) Seismic Design Maps tool (https://seismicmaps.org/) to calculate the seismic design parameters for the project site, which are $F_a = 1$ ($S_s = 1.5$ g) and $F_v = 1.5$ ($S_1 = 0.678$ g). A copy of the SEAOC Design Maps report for the project site is contained in Appendix C of this report.

Based on our findings, we expect there to be a low potential for soil liquefaction at the site. The potential for earthquake-induced lateral spreading and pressure ridges is unlikely.

7.0 DESIGN RECOMMENDATIONS

We have presented our design recommendations in the general order that the project site will most likely be developed. Our design recommendations can be used in parts (as needed) for the final design configuration.

7.1 Earthworks

Our recommendations assume that any shallow foundations (i.e., poured-concrete slabs) will be founded either directly onto the undisturbed gravel deposits or compacted structural fill pads constructed directly above the undisturbed gravel deposits. Any structural fill materials used on-site should be compacted to a minimum of 95 percent of the modified Proctor density.

Any slopes built/constructed at the project site should not exceed a 2:1 slope. If fill is placed onto an existing slope, the fill needs to be properly keyed into the native slope. We can provide more detailed recommendations for keying into the slope upon request. All fill material should be compacted to a minimum of 95% of the modified Proctor density as determined by ASTM D-1557. Additionally, erosion control should be placed on all slopes.

Any material removed during the initial site grading and excavation activities, which does not contain any organic/deleterious material, and has relatively low silt content (less than approximately 15 percent passing the #200 sieve), can be re-used on-site as structural fill. Proper placement and compaction techniques need to be applied during the backfill process (see Section 8.1 of this report for more details). Additional laboratory testing may be required to verify the frost susceptibility of any excavated soil for use in shallow fill applications.

All earthworks should be completed with quality control inspection, including: bottom-of-hole inspections; fill gradation classification; and in-situ compacting testing. A bottom-of-hole inspection should be conducted by a qualified geotechnical engineer, geologist, or special inspector following site excavation activities (and before any foundation construction begins) in order to visually confirm the findings of this report and provide recommendations for any non-conforming conditions encountered during the excavation activities.

7.2 Cold Shallow Foundations

For the purposes of this report, we consider a cold shallow foundation to be any shallow foundation whose subgrade is subjected to freezing temperatures for any amount of time.

Deep foundation systems such as driven piling, helical piers, under-reamed concrete piers, or other deep foundation systems can serve as an alternative means of cold foundation support. We provide a more detailed description of cold deep foundation systems in Section 7.3 of this report. Cost and constructability will typically be the driving forces behind which type of cold foundation system is ultimately selected for a given project.

7.2.1 Soil Bearing Capacity

Concrete slabs placed on either the undisturbed gravel or on structural fill pads (constructed directly above the undisturbed gravel) may be designed for an allowable soil bearing capacity based on the smallest horizontal dimension of the slab (Figure 5). The soil bearing capacity may be increased by one-third (1/3) to accommodate short-term wind and/or seismic loads.

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7.2.2 Concrete Slabs

Concrete slabs can also be founded directly onto the undisturbed gravel or properly placed structural fill located directly above the undisturbed gravel. Concrete slabs should extend a minimum of 16 inches below the finished exterior grade to achieve the recommended allowable soil bearing capacity and help resist any lateral forces. As we mention in Section 7.1 of this report, the upper structural fill material (at or above the footing grade) used to construct the structural pad for a warm foundation should be relatively free draining (sands and gravels) with less than 15% of the fill material passing through a #200 sieve. Furthermore, the top four to six inches of the structural pad located beneath the slabs should be free draining, with less than 3% passing the #200 sieve. This "blanket" will serve as a capillary break to help maintain a dry slab.

Concrete slabs constructed directly on the undisturbed gravel or on properly constructed granular fill pads (located directly above the undisturbed gravel), as we described above, may be designed using a modulus of subgrade reaction of k_1=330 pci (k_1 is the value for a 1-ft × 1-ft rigid plate). For this project, the following equations can be used (with standard English units) to calculate the appropriate modulus of subgrade reaction for load footprints bearing onto the undisturbed native silt/silty sand or on properly placed granular structural fill located directly above the undisturbed native silt/silty sand:

$$k_{(B\ x\ B)} = k_1 \left(\frac{B+1}{2B}\right)^2 \quad (1)$$

Where:

B = the load footprint width of a square load in feet
k_1 = the modulus of subgrade reaction for a 1-ft × 1-ft rigid plate in pci
$k_{(B\ x\ B)}$ = the modulus of subgrade reaction for a square load footprint of width B in pci

7.2.3 Lateral Loads for Foundation

Lateral forces exerted by wind or seismic activity may be resisted by passive-earth pressures against the sides of the foundation. Seismic loading on foundations generally increases the lateral pressures on the wall and decreases the passive resistance. The lateral soil pressures can be represented by equivalent fluid pressures. A value of 385 pcf can be used to represent the lateral soil pressure.

Lateral forces may also be resisted by friction between the concrete foundations and the underlying soil. The frictional resistance may be calculated using a coefficient of friction of 0.4 between the concrete and soil.

7.3 Deep Foundations

For the purposes of this report, a deep foundation can be considered any foundation which transfers foundation loads (both bearing and uplift) through the seasonal active layer and down into permanently thawed soils. As we discuss in Section 6.3 of this report, deep foundations (e.g., pile foundations, etc.) are a suitable alternative to conventional shallow foundations. In some instances, a combination of both a shallow and deep foundation may be employed to help reduce overall construction cost.

We only provide recommendations for driven steel piling in this report; however, helical piers, and drilled concrete are suitable deep foundation alternatives to driven steel piles. It is not feasibly to provide recommendations for all of the various deep foundation alternatives due to the numerous size configurations, etc., however, if an alternative deep foundation system is ultimately selected for the project (other than driven steel piling), then we can provide relevant recommendations for the alternative deep foundation system at that time.

7.3.1 Steel Piles

The most common type of deep foundation system in the Anchorage area consists of driven steel pipe piling; however, it is our understanding that the project site will most likely utilize H-Piles. Any pile installation should be completed with quality control inspection to verify the pile configuration and final penetration rate. The final penetration rate is used to determine that the individual piles have the required axial capacity.

7.3.2 Pile Bearing Capacity

In Figure 6 of this report, we have plotted our estimated individual allowable pile bearing capacities versus embedment depth for a range of steel pipe pile diameters as well as the specified H-pile size.

Final embedment depths should be verified utilizing a wave equation analysis to confirm that the allowable bearing capacity for each pile has been achieved. We can provide this service once the pile driving equipment, design load, and pile specifications are known.

7.3.2.1 Pile Uplift Capacity

The short-term uplift capacity of each pile may be taken as one-half (1/2) of the long-term bearing capacity as we detail in Figure 6 of this report. Our recommendation includes a typical one-third (1/3) increase for short-term wind and seismic loading. When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. We discuss pile group efficiency in further detail in Section 7.3.2.3 of this report.

7.3.2.2 Lateral Pile Capacity

We used the computer program ALLpile7 (developed by CivilTech software) to analyze the lateral capacity for each of the pile diameters/sizes presented in Figure 6 of this report. We assumed a free-head condition for the piles (i.e., the pile head is allowed to rotate/deflect) with the pile head level with the ground surface (i.e., no pile stickup). The ultimate and allowable lateral loads for each pile diameter/size at the ground surface (with no pile stickup) are listed in Table 2 of this report. The allowable lateral loads are ½ of the ultimate lateral loads. We can recalculate the lateral loads once the pile head elevation and connection design has been defined, as it is not feasible for us to provide an analysis for multiple design options. It should be noted that the lateral pile capacities significantly decrease as the pile stickup (above grade) increases. When multiple piles are installed in close proximately to one another, then pile group efficiency should be considered. We discuss group efficiency in Section 7.3.2.3 of this report.

Table 1: Free-Head Lateral Pile Capacity

PILE TYPE	MAX. DEFLECTION (in)	MIN. DEPTH (ft)	ULTIMATE CAPACITY (kips)*	ALLOWABLE CAPACITY (kips)*
8-in SCH. 80	1	15	15.0	7.5
10-in SCH. 80	1	15	21.0	10.5
12-in SCH. 80	1	15	27.2	13.6
14-in SCH. 80	1	15	32.0	16.0
HP14x89	1	15	42.8 (STRONG AXIS) 27.4 (WEAK AXIS)	21.4 (STRONG AXIS) 13.7 (WEAK AXIS)

*Lateral pile capacities calculated with pile head at grade (i.e., no pile stickup above grade)

We also ran a supplemental lateral pile analysis using the computer program LPILE (developed by Ensoft Inc.) to further estimate the lateral pile loads for each of the pile diameters/sizes that we detail in Figure 6 of this report. LPILE uses slightly different design variables and equations to calculate lateral pile loads than ALLPile7, so the output from each program is slightly different; although the differences in the calculated lateral pile loads for this project appear to be negligible. We have included the soil parameters that we used in our LPILE analysis in Table 2 of this report to aid our client in any future LPILE analyses (once the design pile loads and configurations have been established). Furthermore, we can be available to review any additional lateral pile analyses once they are complete to ensure that the analyses properly account for all of the geotechnical site conditions.

Table 2: LPILE Soil Parameters

SOIL TYPE	SOIL MODEL	EFFECTIVE UNIT WEIGHT (PCF)	INTERNAL FRICTION ANGLE (°)	LATERAL MODULUS OF SUBGRADE REACTION (pci)	COHESION (PSF)
GRAVEL (0-4 FT)	SAND (REESE)	140	33	115	N/A
GRAVEL (4-9 FT)	SAND (REESE)	87	33	70	N/A
SAND (9-14.5 FT)	SAND (REESE)	78	34	77	N/A
SILT (14.5-37 FT)	STIFF CLAY W/OUT FREE WATER (REESE)	69	30	272	550

7.3.2.3 Pile Group Efficiency

Group efficiency of steel piles is a function of the spacing of the individual piles. In Table 3 of this report, we present pile group efficiency parameters (as a function of pile diameter/depth). The allowable pile capacities provided in Figure 6 of this report should be adjusted as necessary according to the spacing of individual piles.

Table 3: Axial Pile Group Efficiency Values

PILE SPACING(S)	3B	4B	5B	6B	≥8B
GROUP EFFICIENCY (G_e)	0.70	0.75	0.85	0.90	1.00

*B = Largest Diameter of Pipe Pile / Depth of H-Pile

In Table 4 we provide pile group efficiency parameters for lateral loads. The allowable capacities provided in Table 5 should be adjusted as necessary according the spacing of individual piles.

Table 4: Lateral Pile Group Efficiency Values

PILE SPACING(S)	3B	4B	5B	6B	≥8B
GROUP EFFICIENCY (G_e)	0.50	0.60	0.68	0.70	1.00

*B = Diameter of Pipe Pile / Depth of H-Pile

8.0 CONSTRUCTION RECOMMENDATIONS

We have presented our construction recommendations in the general order that the project site will most likely be developed. Our construction recommendations are intended to aid the construction contractor(s) during the construction process.

8.1 Earthwork

Any and all fill material used should be placed at 95 percent of the modified Proctor density as determined by ASTM D-1557, unless specifically stated otherwise in other sections of this report. The thickness of individual lifts will be determined based on the equipment used, the soil type, and existing soil moisture content. Typically, fill material will need to be placed in lifts of less than one-foot in thickness. All earthworks should be completed with quality control inspection.

In our professional experience, structural fill should have less than approximately 10 to 15 percent passing the #200 sieve for ease of placement. Soils with higher silt contents can be used within the foundation footprint. However, the effort required to achieve proper compaction of silt-rich soils may be more costly than purchasing better grade materials. The time of year, existing moisture content, rainfall, air temperature, and fill temperature can all have an impact on the effort required to adequately compact silt-rich material.

Any excavated coarse grained soils (which are free of organic material and have relatively low silt contents) which are stockpiled on-site (for later use as structural backfill) should be protected from additional moisture inputs (precipitation, etc.) through the use of plastic tarps, etc. Additional

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moisture inputs can have detrimental effects on the effort needed to achieve proper compaction rates.

8.2 Cold Shallow Foundations

Care should be taken during foundation excavation activities to limit the disturbance of the bottom of any foundation excavations. The bottom of any foundation excavation should be moisture conditioned and proof-rolled as necessary to return the exposed soils to their original in-situ density.

Excess water will have a negative impact on any backfill and compaction efforts. Therefore, if surface water does accumulate in any open foundation excavations it can be controlled by excavating a shallow drainage trench around the perimeter of the excavation. The drainage trench will collect surface water and direct it to a sump area, which should be located outside of the foundation footprint. The excess water can then be pumped from the sump area and be discharged at an appropriate location away from the excavation and any other existing foundations.

8.3 Deep Foundations

A drive shoe is not required if the steel pipe pile wall thickness used is sufficient to help reduce the potential for buckling. Any drive shoe used during pipe pile installation should have an outside diameter smaller than the outside diameter of the pile so that it does not oversize the pile annulus and reduce the skin friction on the pile. Once the pile size, pile loading, and pile hammer are chosen, we can perform a pile analysis to determine a final driving rate for the allowable load required.

8.4 Winter Construction

Proper placement and compaction of structural fill is not possible when fill material is frozen, and as such, frozen fill material should never be used for structural support unless it has been subsequently thawed and compacted to 95 percent of the modified Proctor density (throughout its vertical extent). Furthermore, subgrade soils (fill or native) need to be completely thawed prior to the placement and compaction of additional lifts of thawed fill material. In our professional experience, ambient soil temperatures need to be above 37 °F in order to achieve efficient compaction. It is extremely difficult to achieve compaction levels equal to 95 percent of the modified Proctor density in fill material that is between 32 °F to 37 °F.

9.0 THE OBSERVATIONAL METHOD

A comprehensive geoprofessional service (e.g., geotechnical, geological, civil, and/or environmental engineering, etc.) should consist of an interdependent, two-part process comprised of:

Part I - pre-construction site assessment, engineering, and design; and

Part II - continuous construction oversight and design support.

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This process, commonly referred to in the geoprofessional industry as "The Observational Method", was developed to reduce the costs required to complete a construction project, while simultaneously reducing the overall risk associated with the design and construction of the project.

In geotechnical engineering, Part I of the Observational Method (OM) begins with a geotechnical assessment of the site, which typically consists of some combination of literature research, site reconnaissance, subsurface exploration, laboratory testing, and geotechnical engineering. These efforts are usually documented in a formal report (e.g., such as this report) that summarizes the findings of the geotechnical assessment, and presents provisional geotechnical engineering recommendations for design and construction. Geotechnical assessment reports (and the findings and recommendations contained within) are considered provisional due to the fact that their contents are typically based primarily on limited subsurface information for a site. Most conventional geotechnical exploration programs only physically characterize a very small percentage of a given site, as it is typically cost prohibitive to conduct extensive (i.e. high density/frequency) exploration programs. As an alternative, geoprofessionals use the subsurface information available for a site to extrapolate subsurface conditions between exploration locations and develop appropriate provisional recommendations based on the inferred site conditions. As a result, the geoprofessional of record cannot be certain that the provisional recommendations will be wholly applicable to the site, as subsurface conditions other than those identified during the geotechnical assessment may exist at the site which could present obstacles and/or increased risk to the proposed design and construction.

Part II of the OM is employed by geoprofessionals to help reduce the risk associated with unidentified and/or unexpected subsurface conditions. Geoprofessionals accomplish Part II of the OM by providing construction oversight (e.g., construction observation, inspection, and testing). Part II of the OM is a valuable service, as the geoprofessional of record is available if unexpected conditions are encountered during the construction process (e.g., during excavation, fill placement, etc.) to make timely assessments of the unexpected conditions and modify their design and construction recommendations accordingly; thus reducing considerable cost resulting from potential construction delays and reducing the risk of future problems resulting from inappropriate design and construction practices.

Oftentimes, a client may be persuaded to use an alternative geoprofessional firm to conduct Part II of the OM for a given project; as some geoprofessional firms offer the same services at discounted prices in order to help them obtain the overall construction materials engineering and testing (CoMET) commission. The geoprofessional industry as a whole recommends against this practice. An alternative geoprofessional firm cannot provide the same level of service as the geoprofessional of record. The geoprofessional of record has (amongst other things) a unique familiarity with the project including; an intimate understanding of the subsurface conditions, the proposed design, and the client's unique concerns and needs, as well as other factors that could impact the successful completion of a construction project. An alternative geoprofessional firm is not aware of the inferences made and the judgment applied by the geoprofessional of record in developing the provisional recommendations, and may overlook opportunities to provide extra value during Part II of the geoprofessional service.

Clients that prevent the geoprofessional of record from performing a complete service can be held solely liable for any complications stemming from engineering omissions as a result of unidentified conditions. The geoprofessional of record may not be liable for any resulting complications that occur, as the geoprofessional of record was not able to complete their services. Furthermore, the replacement geoprofessional firm may also be found to have no liability for the same reasons.

We are available at any time to discuss the OM in more detail, or to provide you with an estimate for any additional construction observation and testing services required.

10.0 CLOSURE

We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of Electric Power Systems and their consultants/contractors/etc. for use in the design and construction of the planned replacement reactor. We should be notified if significant changes are to occur in the nature, design, or location of the proposed improvements in order that we may review our conclusions and recommendations that we present in this report and, if necessary, modify them to satisfy the proposed changes.

This report should always be read and/or distributed in its entirety (including all figures, exploration logs, appendices, etc.) so that all of the pertinent information contained within is effectively disseminated. Otherwise, an incomplete or misinterpreted understanding of the site conditions and/or our engineering recommendations may occur. Our recommended best practice is to make this report accessible, in its entirety, to any design professional and/or contractor working on the project. Any part of this report (e.g., exploration logs, calculations, material values, etc.) which is presented in the design/construction plans and/or specifications for the project should have an adequate reference which clearly identifies where the report can be obtained for further review.

Due to the natural variability of earth materials, variations in the subsurface conditions across the project site may exist other than those we identified during the course of our geotechnical assessment. Therefore, a qualified geotechnical engineer, geologist, and/or special inspector be on-site during construction activities to provide corrective recommendations for any unexpected conditions revealed during construction (see our discussion of the Observational Method in Section 9.0 of this report for more detail). Furthermore, the construction budget should allow for any unanticipated conditions that may be encountered during construction activities.

We conducted this evaluation following the standard of care expected of professionals undertaking similar work in the State of Alaska under similar conditions. No warranty, expressed or implied, is made.

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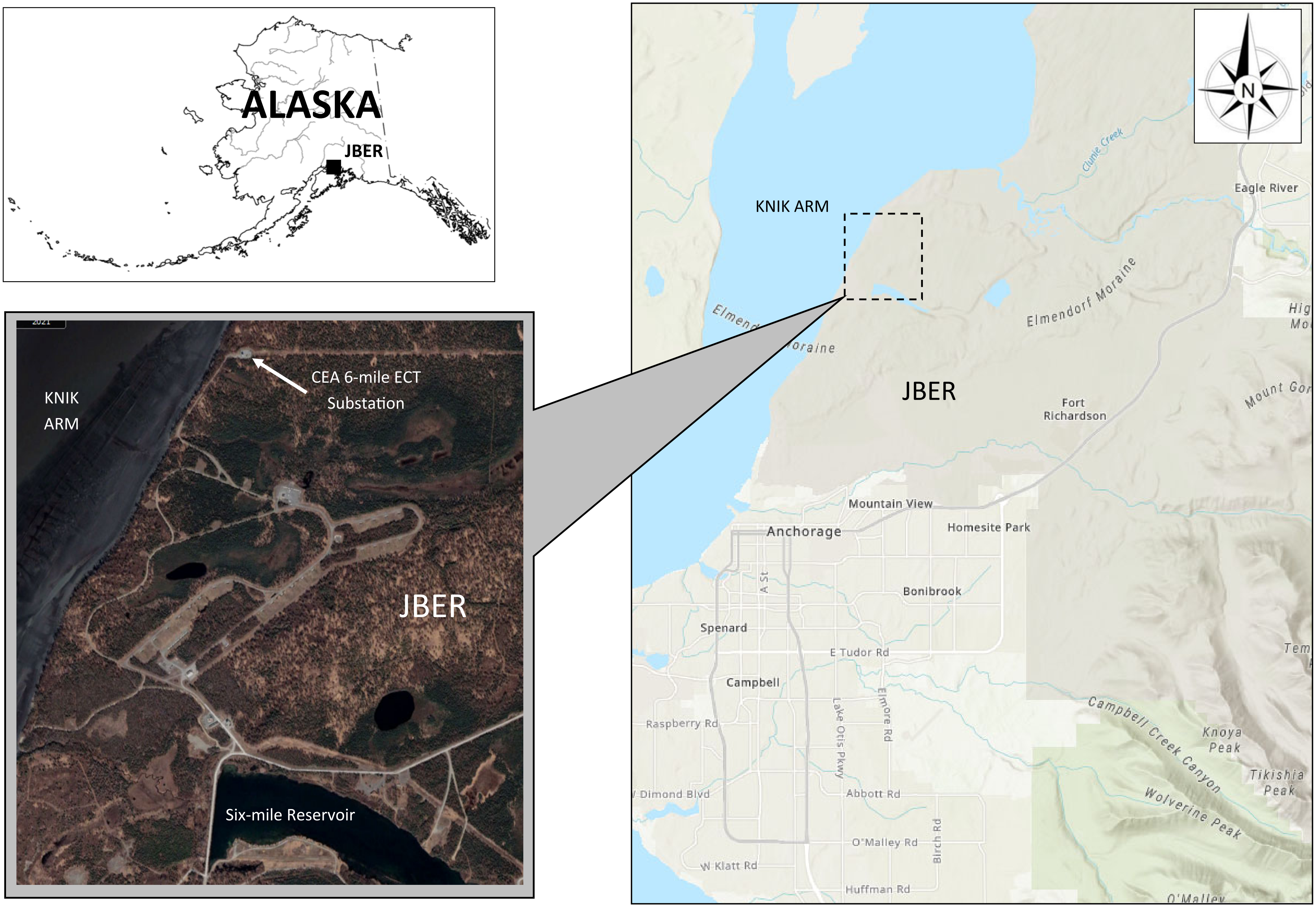
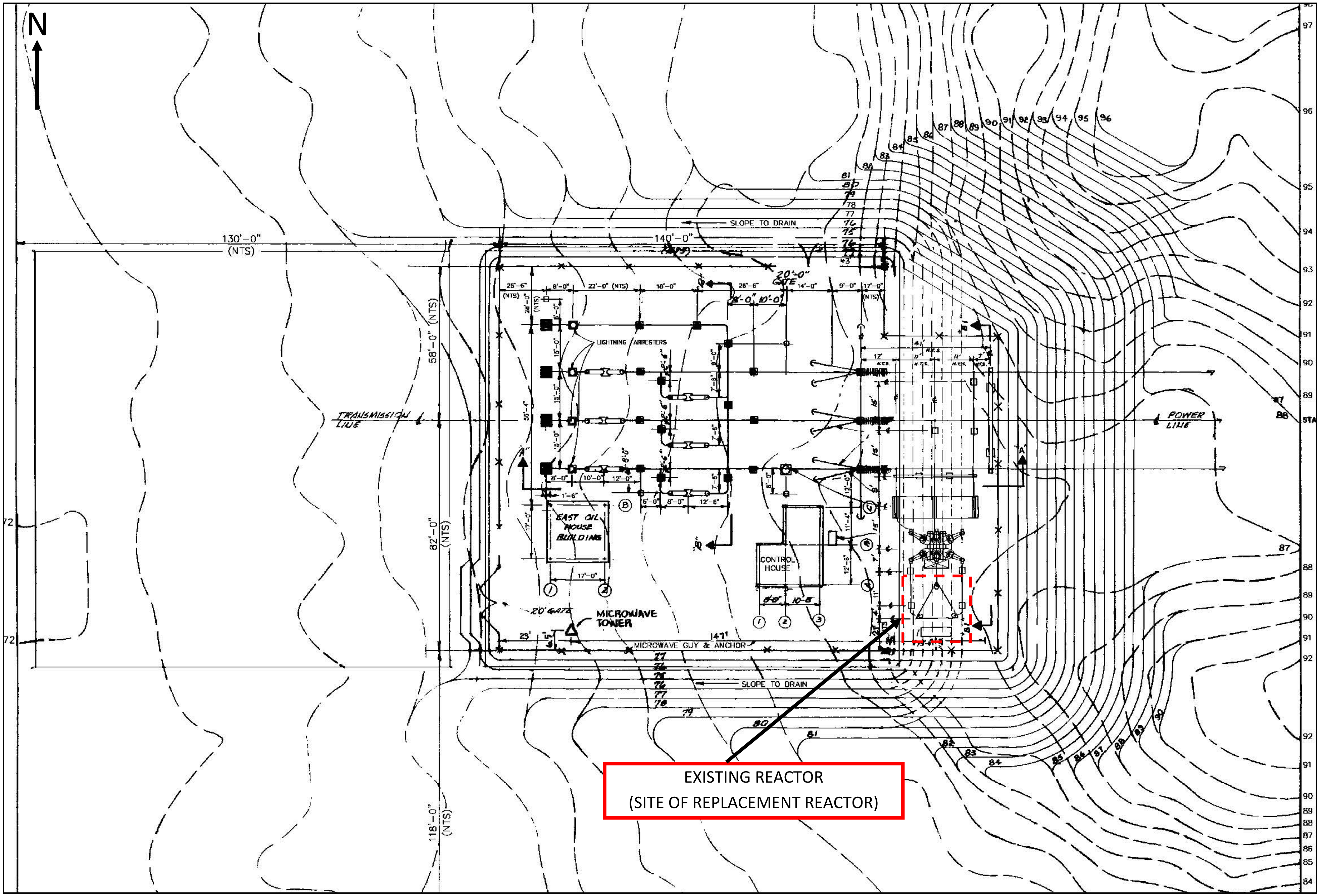
REPORT FIGURES



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROJECT SITE LOCATION	
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 1



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

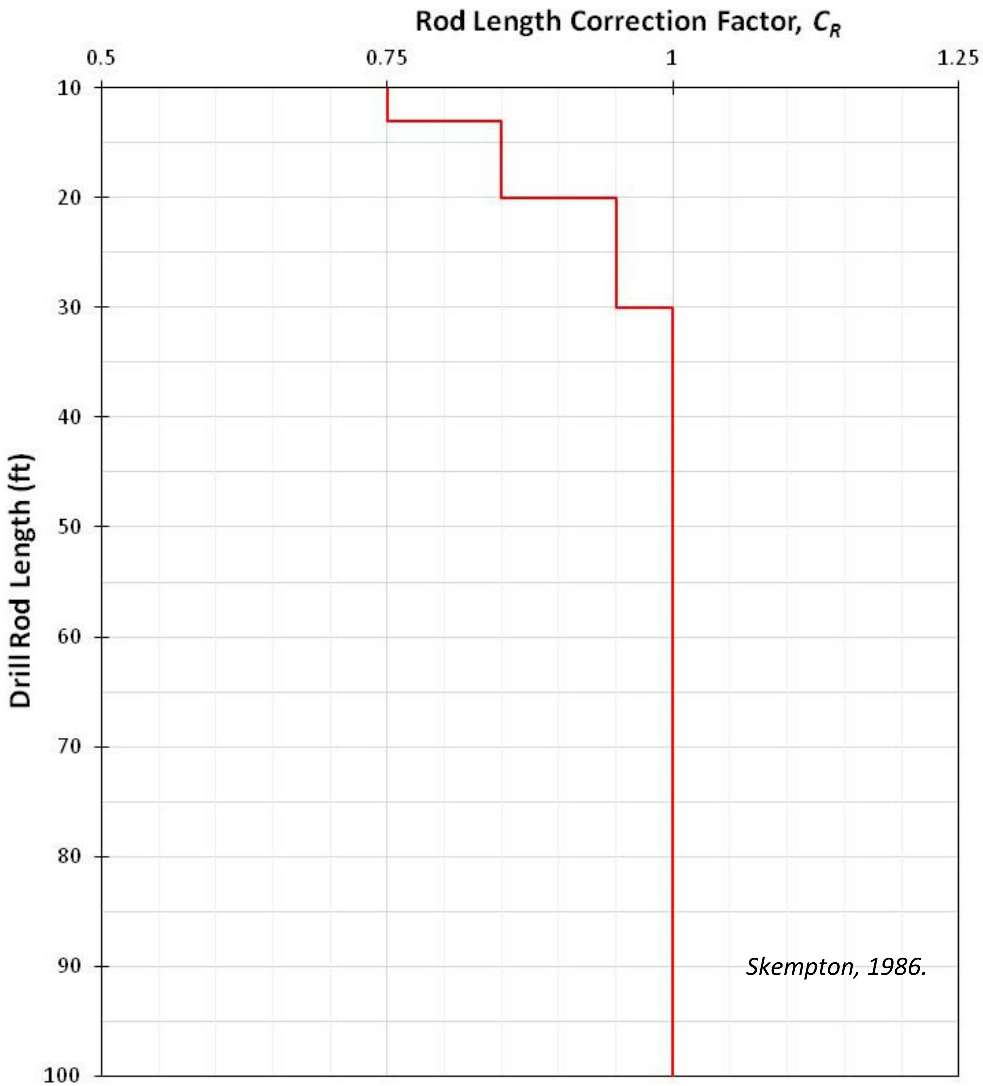
FIGURE TITLE: PLOT PLAN OF EXISTING SIX-MILE ECT SUBSTATION	
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 2

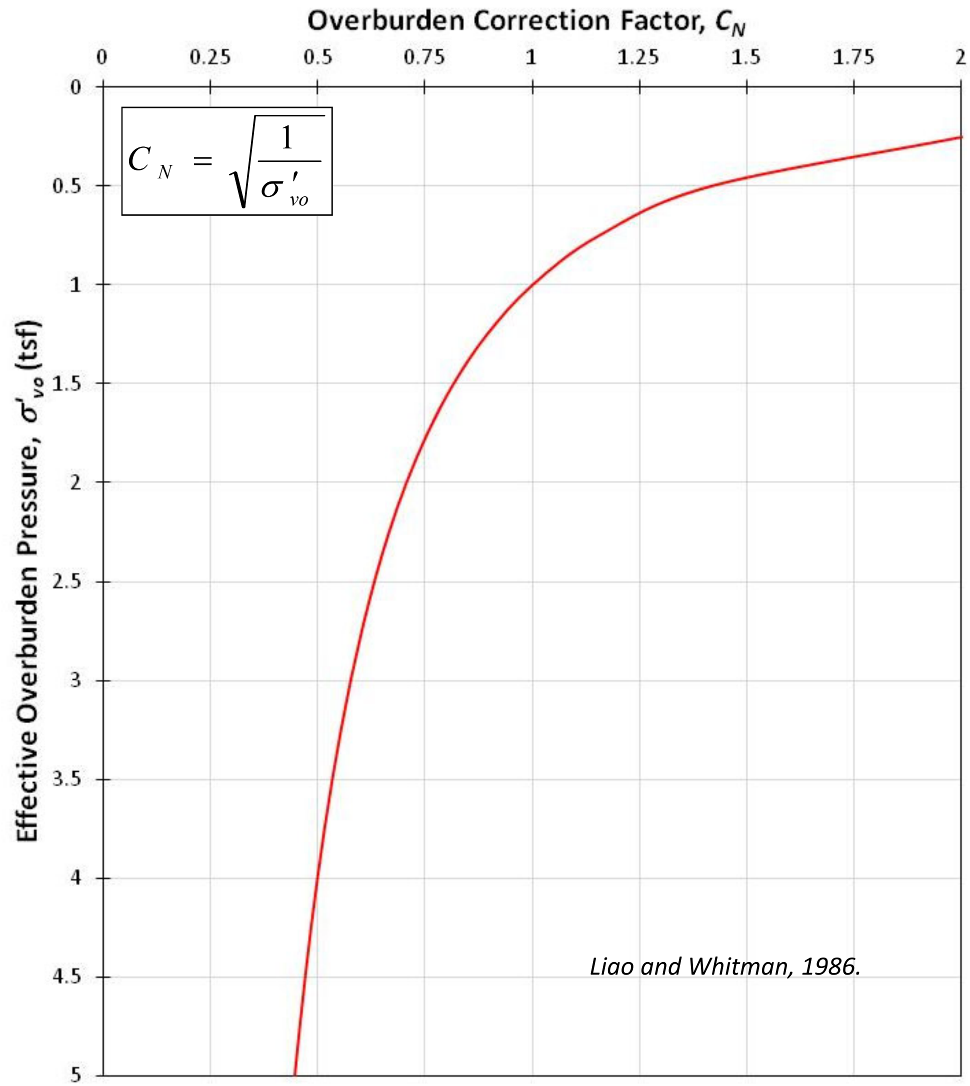
⊕ = APPROX. LOCATION OF SOIL BORING

2014 Aerial Imagery obtained from GoogleEarth 2022)

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: SOIL BORING LOCATIONS		
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21	
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 3	





Notes:

- OVERBURDEN CORRECTION FACTOR IS USED ONLY FOR COHESIONLESS SOILS
- C_N IS THE RATIO OF THE MEASURED BLOW COUNT TO WHAT THE BLOW COUNT WOULD BE AT AN OVERBURDEN PRESSURE OF 1 TON/FT^2
- Σ'_{VO} IS THE EFFECTIVE OVERBURDEN PRESSURE AT THE POINT OF MEASUREMENT (TON/FT^2)

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: BLOW COUNT CORRECTIONS	
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 4

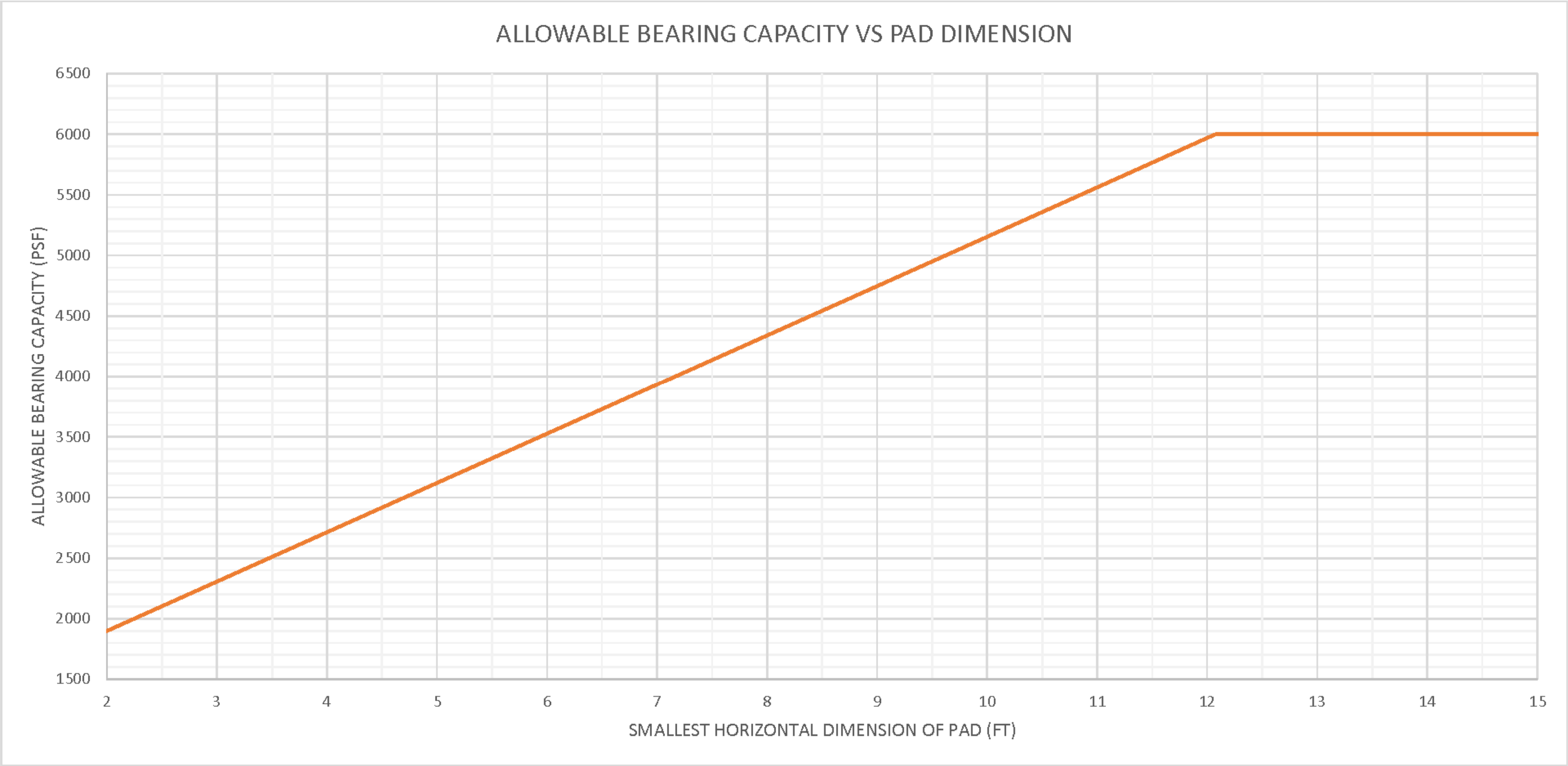


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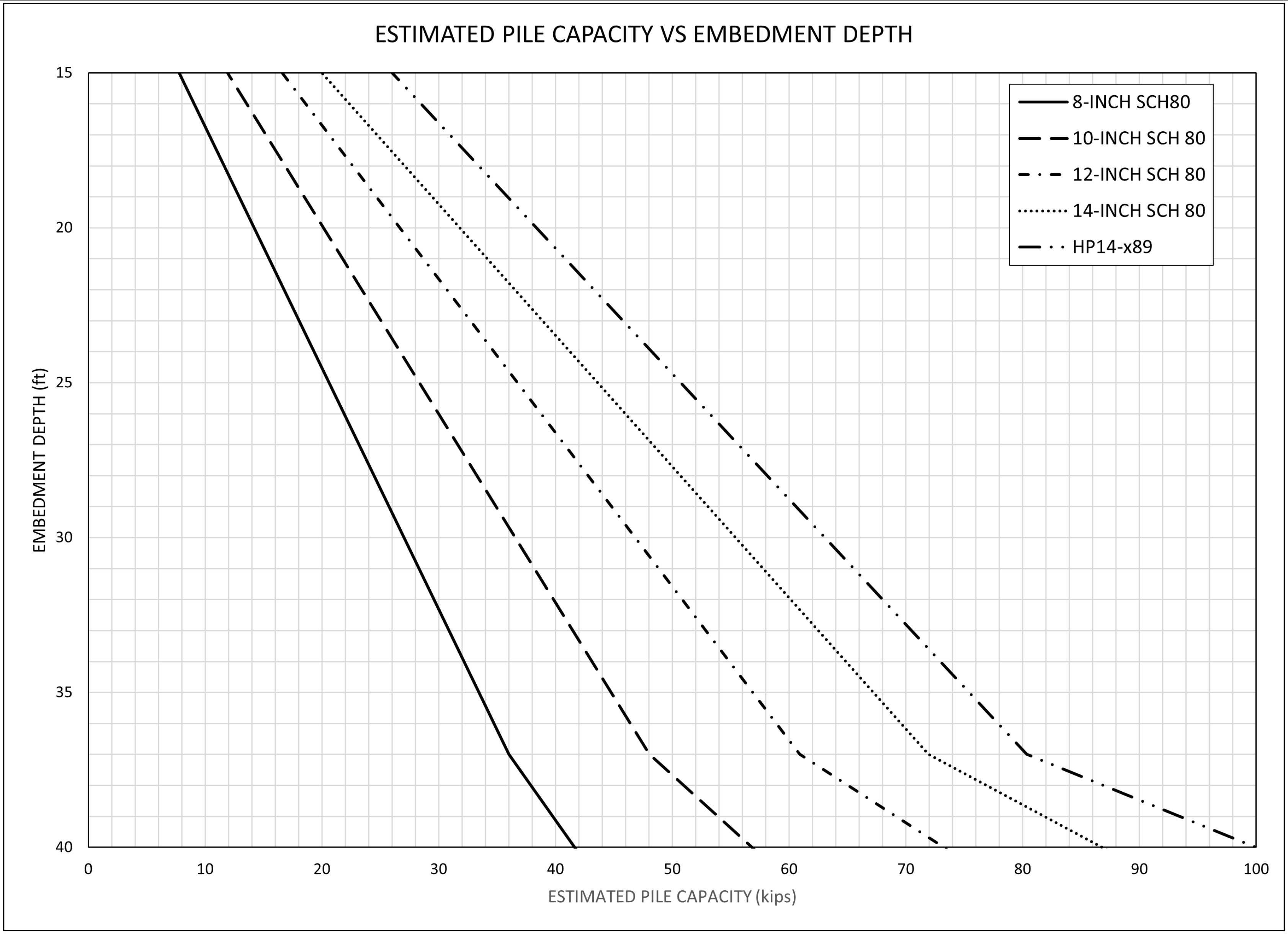
FIGURE TITLE: ALLOWABLE BEARING CAPACITY FOR SLABS	
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 5



NOTES:

1) BEARING CAPACITY SHOWN IS FOR EACH PILE WITHOUT GROUP EFFICIENCY APPLIED.
2) MIN. 15 FEET EMBEDMENT INTO SUBGRADE.
3) ALLOWABLE BEARING CAPACITY MAY BE INCREASED BY 1/3 FOR SHOER-TERM WIND AND SEISMIC LOADS.
4) ALLOWABLE UPLIFT CAPACITY IS 1/2 OF THE ALLOWABLE BEARING CAPACITY.
5) DO NOT ADD 1/3 FOR SHORT-TERM WIND AND SEISMIC LOADS FOR UPLIFT CONDITION.
6) A FINAL DRIVING RATE SHOULD BE USED TO VERIFY THE ESTIMATED ALLOWABLE BEARING CAPACITY HAS BEEN OBTAINED.

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FIGURE TITLE: ALLOWABLE PILE BEARING CAPACITIES	
PROJECT NAME: CEA SIX-MILE ECT SUBSTATION REACTOR REPLACEMENT	PROJECT ID: 6238-21
PROJECT LOCATION: JBER, ALASKA	FIGURE NUMBER: 6

APPENDIX A

GRAPHICAL BOREHOLE LOGS

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EXPLORATION B1

NGE-TFT PROJECT NAME: 230 kV 30 MVAR Shunt Reactor
NGE-TFT PROJECT NUMBER: 6238-21
PROJECT LOCATION: CEA 6-Mile ECT SS - JBER, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 1/7/2022 @ 8:30:00 AM
DATE/TIME COMPLETED: 1/7/2022 @ 10:30:00 AM
EXPLORATION LOCATION: Apx. 15 ft SW of existing reactor (See report Figure 3)
GROUND ELEVATION: Not Known
GROUNDWATER (ATD): Approx. 6.0 ft bgs
GROUNDWATER (): N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Fog, light wind, -5°F

DEPTH (ft bgs)	MATERIAL DESCRIPTION	SAMPLE TYPE	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**WELL GRADED GRAVEL WITH SILT AND SAND** (GW-GM), olive brown, gravel up to 3" in diameter with some cobbles up to 4" in diameter. Upper 1-2 feet may be re-worked as fill								12" diameter pilot hole excavated to approx. 5.75 ft bgs using vac truck on 1/6/22.
5			S1	0	3 3 3	N/A	S1		No recovery - pushing cobble.
			S2	16	9 11 15	N/A	S2A S2B	S2A MC = 24.2% S2B MC = 16.8% P200 = 6.9%	
10	**POORLY GRADED SAND WITH SILT AND GRAVEL** (SP-SM), medium dense, olive brown, saturated, gravel up to 1" in diameter		S3	17	7 9 11	23	S3	S3 MC = 12.4% 30.1% gravel, 64.6% sand, 5.3% silt	
15	**SILT** (ML), medium stiff, medium gray, damp, medium plasticity		S4	13	4 4 6	9	S4	S4 MC = 24.9% P200 = 94.5% LL = 33 PL = 25 PI = 8.6	
20			S5	18	4 4 6	10	S5	S5 MC = 23.3%	PP = 3.5 tsf (H). PP = 4.5 tsf (V).
25									

Always refer to our complete geotechnical report for this project for a more detailed explanation of the subsurface conditions at the project site and how they may affect any existing and/or prospective project site development.

(Continued Next Page)

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EXPLORATION B1

NGE-TFT PROJECT NAME: 230 kV 30 MVAR Shunt Reactor
NGE-TFT PROJECT NUMBER: 6238-21
PROJECT LOCATION: CEA 6-Mile ECT SS - JBER, AK
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: MPT w/ 340lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 1/7/2022 @ 8:30:00 AM
DATE/TIME COMPLETED: 1/7/2022 @ 10:30:00 AM
EXPLORATION LOCATION: Apx. 15 ft SW of existing reactor (See report Figure 3)
GROUND ELEVATION: Not Known
GROUNDWATER (ATD): Approx. 6.0 ft bgs
GROUNDWATER (): N/A
EXPLORATION COMPLETION: See comments at end of log
WEATHER CONDITIONS: Fog, light wind, -5°F

DEPTH (ft bgs)	GRAPHIC LOG	FROZEN SOILS	MATERIAL DESCRIPTION	SAMPLE TYPE	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	SAMPLE INT. COLLECT	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
25			**SILT** (ML), medium stiff, medium gray, damp, medium plasticity *(continued)*		S6	18	3 4 4	8		S6	S6 MC = 25.4% P200 = 92.7% LL = 31 PL = 24 PI = 7.1	PP = 3.5 tsf (H). PP = 4.0 tsf (V).
30					S7	20	3 5 6	11		S7	S7 MC = 20.3%	PP = 3.5 tsf (H). PP = 4.5 tsf (V).
35												
			SILTY GRAVEL WITH SAND (GM), dense, medium gray, damp, gravel up to 3" in diameter									Increased drilling resistance at approx. 37 ft bgs.
40					S8	20	8 10 14	24		S8		

Bottom of borehole at 41.5 ft bgs.
Backfilled with cuttings to approx. 4 ft bgs. P-gravel from 3-4 ft bgs.
Bentonite chips from 2-3 ft bgs. P-gravel from 0-2 ft bgs.

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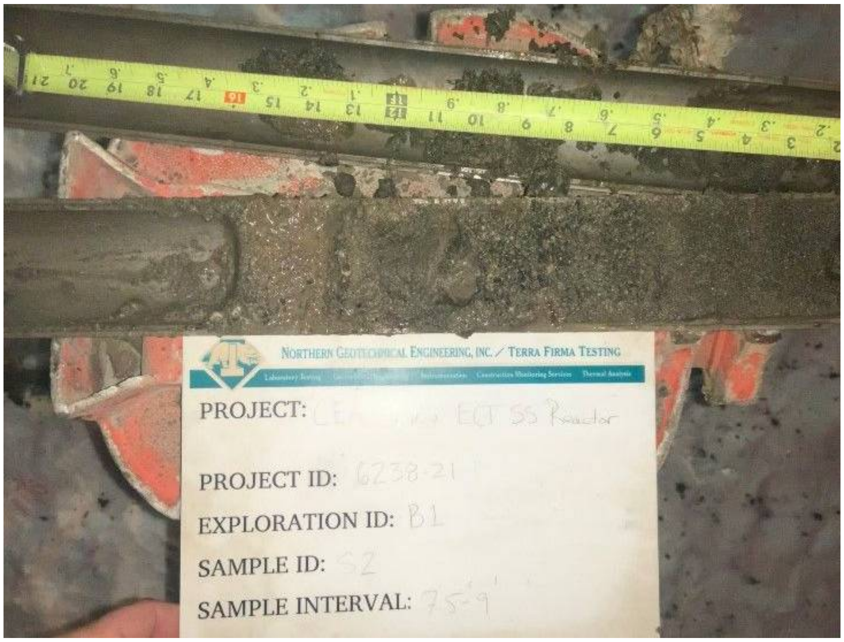
PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.

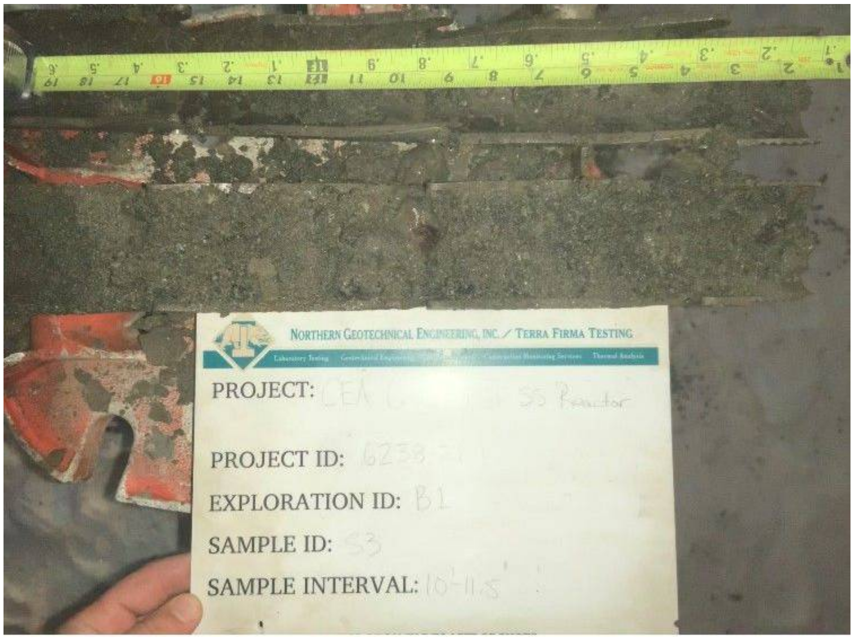
PROJECT NAME 230 kV 30 MVAR Shunt Reactor

PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK



Exploration B1 Sample S2
Sample Interval 7.5 - 9 ft bgs



Exploration B1 Sample S3
Sample Interval 10 - 11.5 ft bgs

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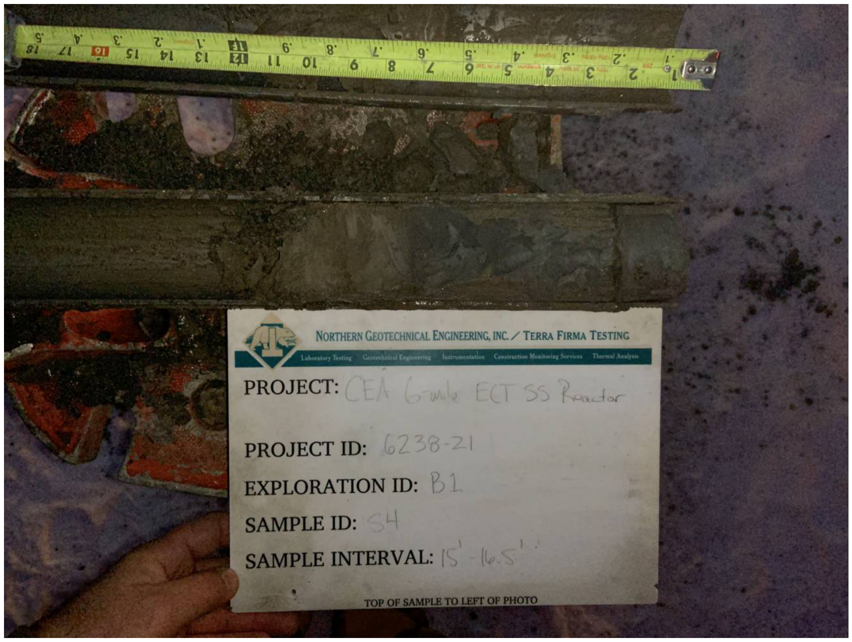
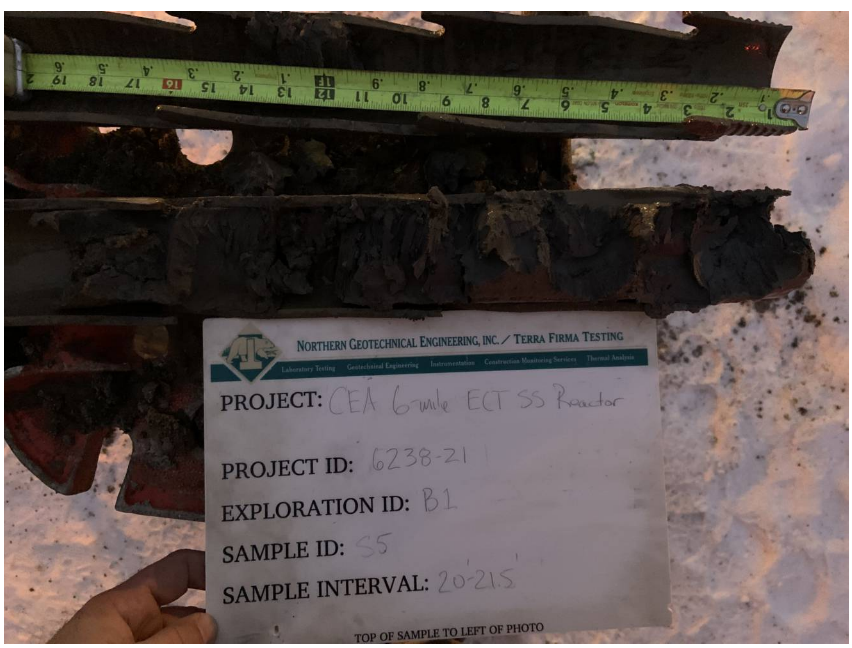
PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.
PROJECT NAME 230 kV 30 MVAR Shunt Reactor
PROJECT NUMBER 6238-21
PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK



Exploration B1 Sample S4
Sample Interval 15 - 16.5 ft bgs



Exploration B1 Sample S5
Sample Interval 20 - 21.5 ft bgs

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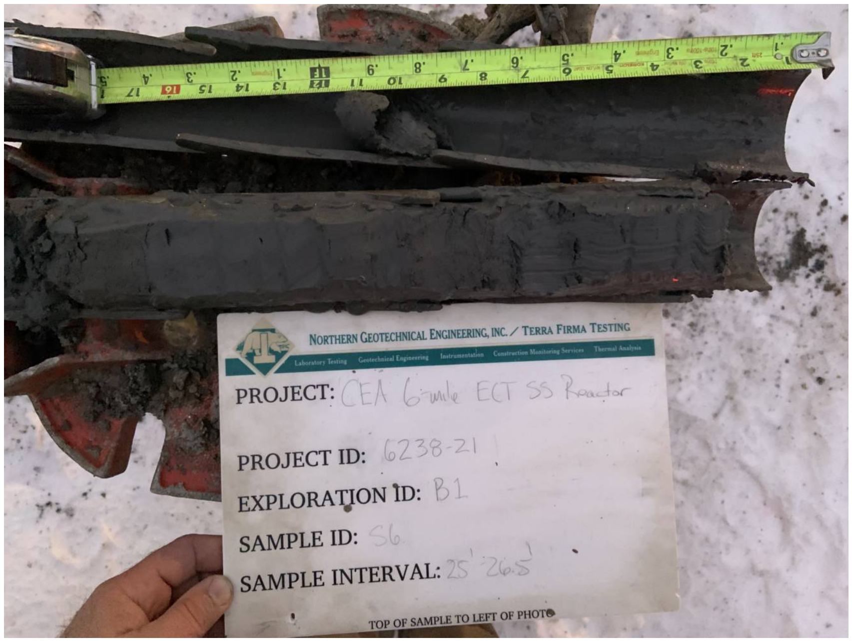
PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.
PROJECT NAME 230 kV 30 MVAR Shunt Reactor
PROJECT NUMBER 6238-21
PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK



Exploration B1 Sample S6
Sample Interval 25 - 26.5 ft bgs

Exploration B1 Sample S7
Sample Interval 30 - 31.5 ft bgs

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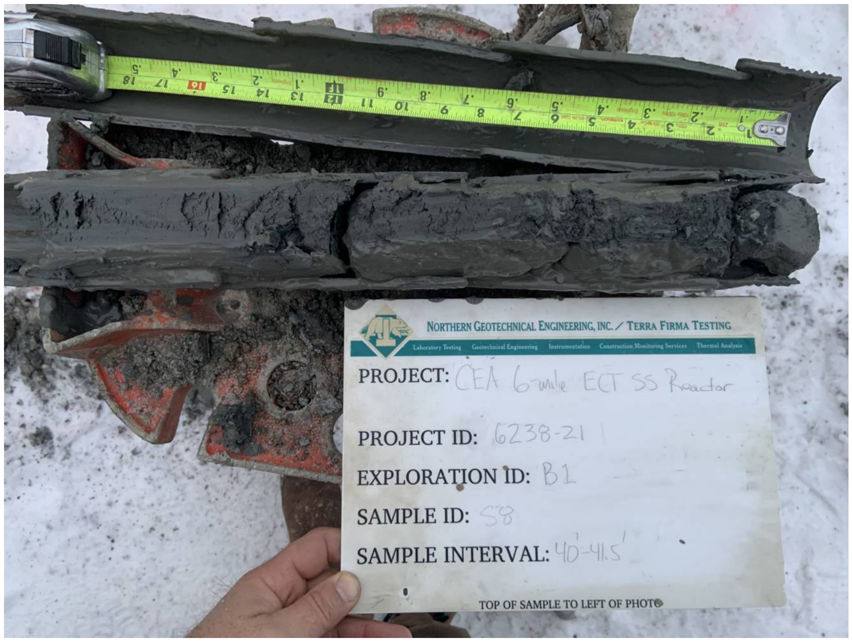
PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.

PROJECT NAME 230 kV 30 MVAR Shunt Reactor

PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK



Exploration B1 Sample S8
Sample Interval 40 - 41.5 ft bgs

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EXPLORATION B2

NGE-TFT PROJECT NAME: 230 kV 30 MVAR Shunt Reactor

NGE-TFT PROJECT NUMBER: 6238-21

PROJECT LOCATION: CEA 6-Mile ECT SS - JBER, AK

EXPLORATION CONTRACTOR: Discovery Drilling, Inc.

EXPLORATION EQUIPMENT: CME 85

EXPLORATION METHOD: Hollow Stem Auger

SAMPLING METHOD: MPT w/ 340lb autohammer

LOGGED BY: A. Smith

DATE/TIME STARTED: 1/7/2022 @ 10:55:00 AM

DATE/TIME COMPLETED: 1/7/2022 @ 12:15:00 PM

EXPLORATION LOCATION: Apx. 18 ft W of existing reactor (See report Figure 3)

GROUND ELEVATION: Not Known

▽ **GROUNDWATER (ATD):** Approx. 6.0 ft bgs

▼ **GROUNDWATER (1/18/2022):** Approx. 3.8 ft bgs

EXPLORATION COMPLETION: See comments at end of log

WEATHER CONDITIONS: Fog, light wind, -5°F

DEPTH (ft bgs)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**WELL GRADED GRAVEL WITH SILT AND SAND** (GW-GM), olive brown, damp to saturated, gravel up to 2" in diameter. Upper 1-2 feet may be re-worked as fill							12" diameter pilot hole excavated to approx. 5.5 ft bgs using vac truck on 1/6/22.
5		S1	14	5 7 9	22	S1	S1 MC = 6.1% 53.5% gravel, 38.4% sand, 8.1% silt P0.02 = 5.1% FC = F1	
		S2	13	8 9 8	21	S2	S2 MC = 8.3%	
10	**POORLY GRADED SAND** (SP), olive brown, saturated, trace gravel up to 0.25" in diameter	S3	20	6 9 14	27	S3	S3 MC = 20.2% P200 = 3.9%	
15	**SILT** (ML), medium stiff, medium gray, damp, medium plasticity	S4	19	4 6 7	12	S4	S4 MC = 20.1%	
20	**SILTY SAND** (SM), dense, dark gray, saturated	S5	18	8 10 10	23	S5A	S5A MC = 22.5% P200 = 25.9%	

Bottom of borehole at 21.5 ft bgs.
Set 1" PVC casing to BOH. Hand slotted bottom 15' of casing. Backfill with cuttings to approx. 5 ft bgs. P-gravel from 3-5 ft bgs. Bentonite chips from 2-3 ft bgs. P-gravel from 0-2 ft bgs.

Always refer to our complete geotechnical report for this project for a more detailed explanation of the subsurface conditions at the project site and how they may affect any existing and/or prospective project site development.

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PHOTO LOG
EXPLORATION B2

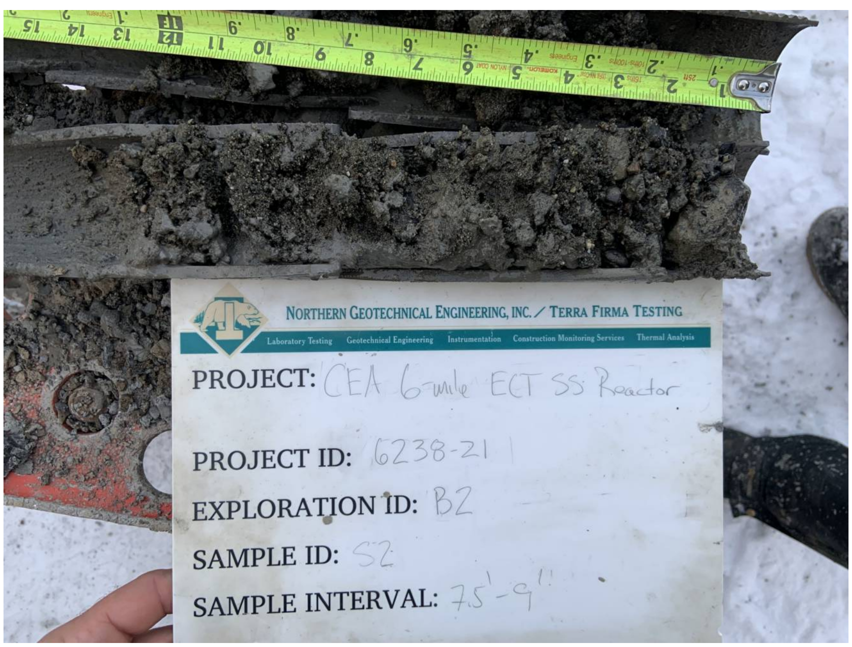
CLIENT Electric Power Systems, Inc.

PROJECT NAME 230 kV 30 MVAR Shunt Reactor

PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK

Exploration B2 Sample S1
Sample Interval 5.5 - 7 ft bgs



Exploration B2 Sample S2
Sample Interval 7.5 - 9 ft bgs

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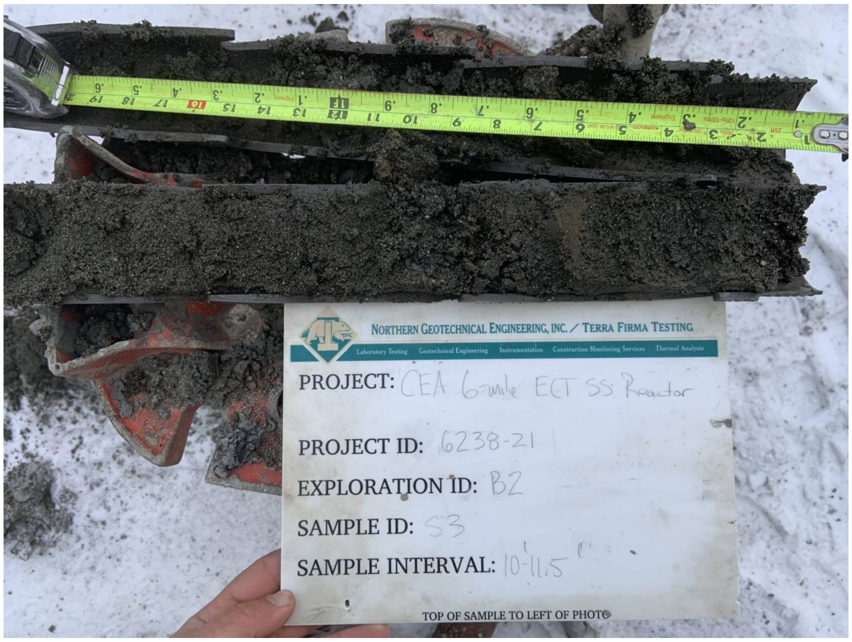
PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc.
PROJECT NAME 230 kV 30 MVAR Shunt Reactor
PROJECT NUMBER 6238-21
PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK



Exploration B2 Sample S3
Sample Interval 10 - 11.5 ft bgs

Exploration B2 Sample S4
Sample Interval 15 - 16.5 ft bgs

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PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc. **PROJECT NAME** 230 kV 30 MVAR Shunt Reactor

PROJECT NUMBER 6238-21 **PROJECT LOCATION** CEA 6-Mile ECT SS - JBER, AK

Exploration B2 Sample S5
Sample Interval 20 - 21.5 ft bgs

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EXPLORATION LEGEND

CLIENT Electric Power Systems, Inc.

NGE-TFT PROJECT NAME 230 kV 30 MVAR Shunt Reactor

NGE-TFT PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK

LITHOLOGIC SYMBOLS
(Unified Soil Classification System)

GM: USCS Silty Gravel

GW-GM: USCS Well-graded Gravel with Silt

ML: USCS Silt

SM: USCS Silty Sand

SP: USCS Poorly-graded Sand

SP-SM: USCS Poorly-graded Sand with Silt

SAMPLER SYMBOLS

Modified Penetration Test

No Recovery

WELL CONSTRUCTION SYMBOLS

Bentonite Seal

Pipe backfilled with pea gravel

Slough Backfill

Slotted Pipe Backfilled with Slough

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
MC - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
P200 - PERCENT PASSING NO. 200 SIEVE
P0.02- PERCENT PASSING 0.02mm SIEVE
PP - POCKET PENETROMETER (tons/ft^2)
S/U - CASING STICK-UP

▽ Water Level at Time Drilling, or as Shown

▼ Water Level After 24 Hours, or as Shown

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION
N/E - NOT ENCOUNTERED
N/R - NOT REPRESENTATIVE
N/A - NOT APPLICABLE

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SOIL CLASSIFICATION CHART

CLIENT Electric Power Systems, Inc.

PROJECT NAME 230 kV 30 MVAR Shunt Reactor

NGE-TFT PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.
DIAGONAL LINES INDICATE UNKNOWN DEPTH OF SOIL TRANSITION.

Northern Geotechnical Engineering, Inc.
and Terra Firma Testing
11301 Olive Lane
Anchorage, AK 99515
Telephone: 907-344-5934

EXPLORATION LOG KEY

CLIENT Electric Power Systems, Inc.
PROJECT NAME 230 kV 30 MVAR Shunt Reactor
NGE-TFT PROJECT NUMBER 6238-21
PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK

SAMPLER SYMBOLS

SPT w/ 140# Hammer
30" Drop and 2.0" O.D. Sampler

Modified SPT w/ 340# Hammer
30" Drop and 3.0 O.D. Sampler

Grab Sample

Shelby Tube Sample

Rock Core Sample

Direct Push Sample

No Recovery

N/E Not Encountered

WELL SYMBOLS

1" Slotted Pipe
Backfilled with Silica Sand

1" PVC Pipe
Backfilled with Auger Cuttings

1" PVC Pipe
with Bentonite Seal

Capped Riser

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No. 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No. 4 (4.5 mm)
Sand	No. 4 (4.5 mm) to No. 200
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace	1-5%
Few	5-10%
Little	10-20%
Some	20-35%
And	35-50%

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch
DAMP	Some perceptible moisture; below optimum
MOIST	No visible water; near optimum moisture content
WET	Visible free water, usually soil is below water table

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
DENSITY	N (BLOWS/FT)	APPROXIMATE RELATIVE DENSITY (%)	CONSISTENCY	N (BLOWS/FT)	APPROXIMATE UNDRAINED SHEAR STRENGTH (PSF)
VERY LOOSE	0-4	0-15	VERY SOFT	0-1	< 250
LOOSE	5-10	15-35	SOFT	2-4	250-500
MEDIUM DENSE	11-25	35-65	MEDIUM STIFF	5-8	500-1000
DENSE	26-50	65-85	STIFF	9-15	1000-2000
VERY DENSE	> 50	85-100	VERY STIFF	16-30	2000-4000
			HARD	> 30	> 4000

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EXPLORATION LOG KEY

CLIENT Electric Power Systems, Inc.

PROJECT NAME 230 kV 30 MVAR Shunt Reactor

NGE-TFT PROJECT NUMBER 6238-21

PROJECT LOCATION CEA 6-Mile ECT SS - JBER, AK

FROST DESIGN SOIL CLASSIFICATION

FROST GROUP (USACOE)	FROST GROUP (M.O.A.)	SOIL TYPE	% FINER THAN 0.02mm BY MASS	TYPICAL SOIL TYPES UNDER UNIFIED SOIL CLASSIFICATION SYSTEM
NFS*	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK (B) SANDS	0 - 1.5 0 - 3	GW, GP SW, SP
PFS+	NFS*	(A) GRAVELS CRUSHED STONE CRUSHED ROCK	1.5 - 3	GW, GP
	F2	(B) SANDS	3 - 10	SW, SP
S1	F1	GRAVELLY SOILS	3 - 6	GW, GP, GW-GM, GP-GM
S2	F2	SANDY SOILS	3 - 6	SW, SP, SW-SM, SP-SM
F1	F1	GRAVELLY SOILS	6 - 10	GM, GW-GM, GP-GM
F2	F2	(A) GRAVELLY SOILS (B) SANDS	10 - 20 6 - 15	GM, GW-GM, GP-GM SM, SW-SM, SP-SM
F3	F3	(A) GRAVELLY SOILS (B) SANDS, EXCEPT VERY FINE SILTY SANDS (C) CLAYS, PI>12	Over 20 Over 15 - - - - - -	GM, GC SM, SC CL, CH
F4	F4	(A) ALL SILTS (B) VERY FINE SILTY SANDS (C) CLAYS, PI<12 (D) VARVED CLAYS AND OTHER FINE GRAINED, BANDED SEDIMENTS	- - - - - - Over 15 - - - - - - - - - - - -	ML, MH SM CL, CL-ML CL & ML; CL, ML, & SM; CL, CH, & ML; CL, CH, ML, & SM

*Non-frost susceptible
+Possibly frost susceptible, but requires lab testing to determine frost design soils classification.

ICE CLASSIFICATION SYSTEM

GROUP	ICE VISIBILITY	DESCRIPTION		SYMBOL	
N	SEGREGATED ICE NOT VISIBLE BY EYE	POORLY BONDED OR FRIABLE		Nf	
		WELL BONDED	NO EXCESS ICE	Nb	Nbn
			EXCESS MICROSCOPIC ICE		Nbe
V	SEGREGATED ICE IS VISIBLE BY EYE AND IS ONE INCH OR LESS IN THICKNESS	INDIVIDUAL ICE CRYSTALS OR INCLUSIONS		Vx	
		ICE COATINGS ON PARTICLES		Vc	
		RANDOM OR IRREGULARY ORIENTED ICE		Vr	
		STRATIFIED OR DISTINCTLY ORIENTED ICE		Vs	
		UNIFORMLY DISTRIBUTED ICE		Vu	
ICE	ICE IS GREATER THAN ONE INCH IN THICKNESS	ICE WITH SOILS INCLUSIONS		ICE + Soil Type	
		ICE WITHOUT SOILS INCLUSIONS		ICE	

APPENDIX B

LABORATORY TEST RESULTS

Summary of Laboratory Test Results

CEA 6-mile ECT SS Reactor

NGE-TFT Project #:6238-21

Exploration ID	Sample Number	Depth Interval		Moisture Content ASTM D2216 (% By Dry Mass)	Atterberg Limits ASTM D4318			Particle Size Analysis ASTM C136/D7928/D6913 (% By Mass)			Passing #200 ASTM D1140 (% By Mass)	Passing 0.02mm ASTM D7928 (% By Mass)	Frost Class. (MOA)	Unified Soil Classification ASTM D2487
		(ft) Top	(ft) Bottom		LL	PL	PI	Gravel	Sand	Silt/Clay				
B1	S2A	7.5	8.5	24.2										
B1	S2B	8.5	9.0	16.8							6.9			
B1	S3	10.0	11.5	12.4				30.1	64.6	5.3		N/A	N/A	(SP-SM) Poorly-graded sand w/ silt and gravel
B1	S4	15.0	16.5	24.9	33.4	24.8	8.6				94.5			
B1	S5	20.0	21.5	23.3										
B1	S6	25.0	26.5	25.4	30.8	23.7	7.1				92.7			
B1	S7	30.0	31.5	20.3										
B1	S8	40.0	41.5	12.7										
B2	S1	5.5	7.0	6.1				53.5	38.4	8.1		5.1	F1	(GW-GM) Well-graded gravel w/ silt and sand
B2	S2	7.5	9.0	8.3										
B2	S3	10.0	11.5	20.2							3.9			
B2	S4	15.0	16.5	20.1										
B2	S5A	20.5	21.5	22.5							25.9			

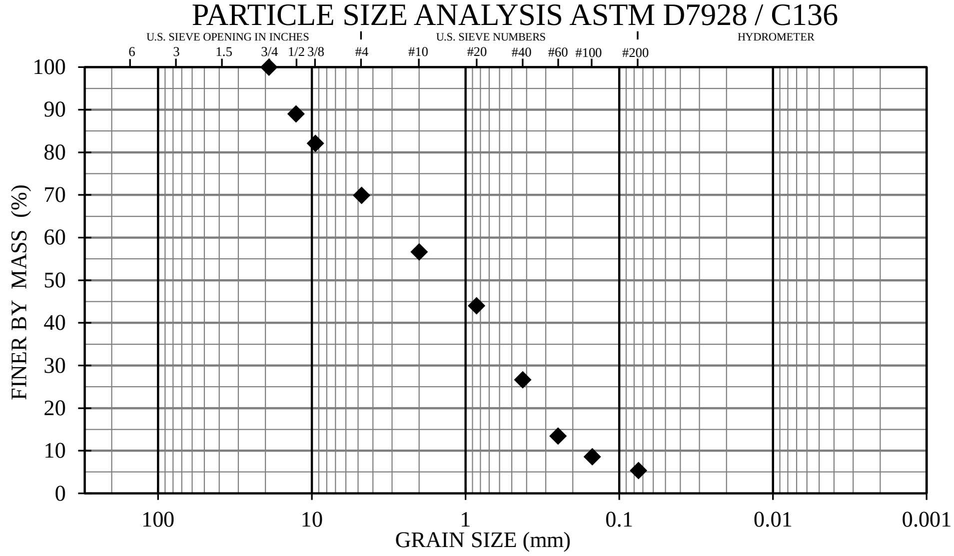
NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

Laboratory Testing | Geotechnical Engineering | Instrumentation | Construction Monitoring Services | Thermal Analysis

PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA 6-mile ECT SS Reactor**
PROJECT NO.:	**6238-21**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S3 / 10 - 11.5'**
DESCRIPTION:	**Poorly-graded sand w/ silt and gravel**
DATE RECEIVED:	**1/7/2022**
TESTED BY:	**EB**
REVIEWED BY:	**ACS**

% GRAVEL	**30.1**	USCS	**SP-SM**
% SAND	**64.6**	MOA FC	**N/A**
% SILT/CLAY	**5.3**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**12.4**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)	**15.0**		
COEFFICIENT OF GRADATION (C_c)	**0.5**		
ASTM D1557 (uncorrected)	**N/A**		
ASTM D4718 (corrected)	**N/A**		
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**		

PARTICLE SIZE ANALYSIS ASTM D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

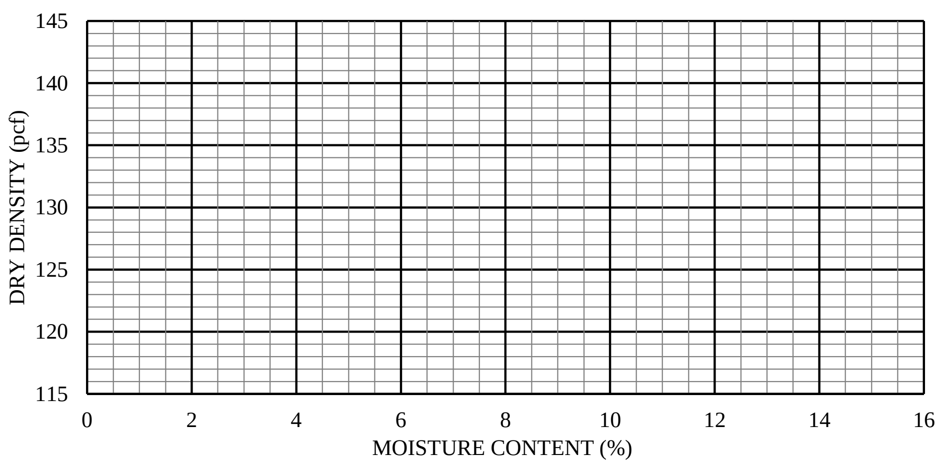
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"		
19.00	3/4"	**100**	
12.70	1/2"	**89**	
9.50	3/8"	**82**	
4.75	#4	**70**	
2.00	#10	**57**	
0.85	#20	**44**	
0.43	#40	**27**	
0.25	#60	**13**	
0.15	#100	**9**	
0.075	#200	**5.3**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1		
2		
5		
8		
15		
30		
60		
250		
1440		

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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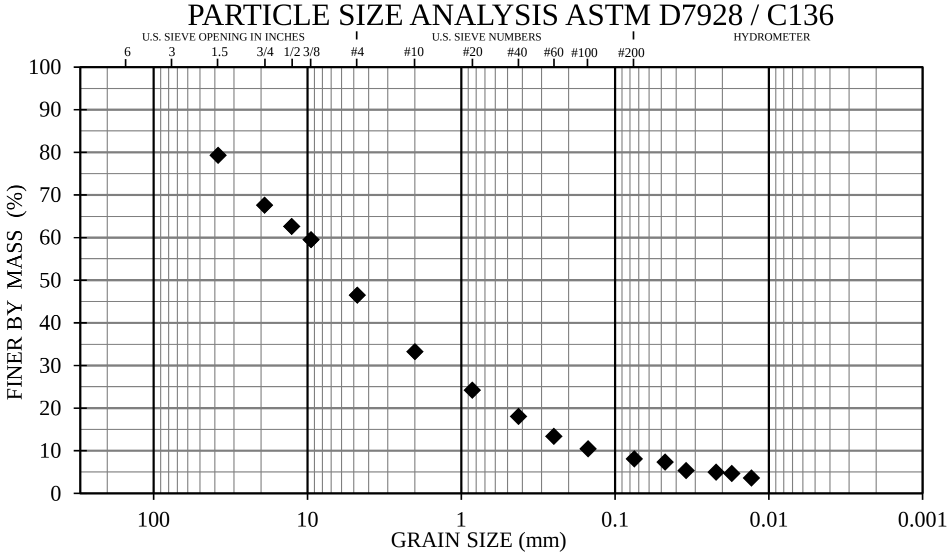
PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA 6-mile ECT SS Reactor**
PROJECT NO.:	**6238-21**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S1 / 5.5 - 7'**
DESCRIPTION:	**Well-graded gravel w/ silt and sand**
DATE RECEIVED:	**1/7/2022**
TESTED BY:	**EB**
REVIEWED BY:	**ACS**

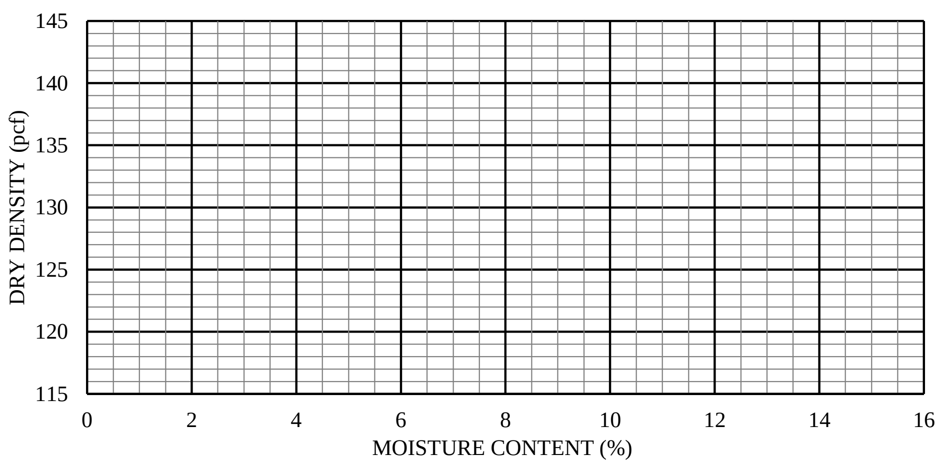
% GRAVEL	**53.5**	USCS	**GW-GM**
% SAND	**38.4**	MOA FC	**F1**
% SILT/CLAY	**8.1**	% PASS. 0.02 mm	**5.1**
% MOIST. CONTENT	**6.1**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**73.8**	
COEFFICIENT OF GRADATION (C_c)		**1.9**	
ASTM D1557 (uncorrected)		**N/A**	
ASTM D4718 (corrected)		**N/A**	
OPTIMUM MOIST. CONTENT. (corrected)		**N/A**	

PARTICLE SIZE ANALYSIS ASTM D7928 / C136



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**79**	
19.00	3/4"	**68**	
12.70	1/2"	**63**	
9.50	3/8"	**60**	
4.75	#4	**46**	
2.00	#10	**33**	
0.85	#20	**24**	
0.43	#40	**18**	
0.25	#60	**13**	
0.15	#100	**10**	
0.075	#200	**8.1**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0473	7.4
2	0.0345	5.3
5	0.0221	5.0
8	0.0174	4.6
15	0.0130	3.6
30		
60		
250		
1440		

HYDRAULIC COND. (ASTM D2434)	**N/A**
DEGRADATION (ATM T-313)	**N/A**
PLASTICITY INDEX ASTM 4318	**N/A**

The testing services reported herein have been performed to recognized industry standards, unless otherwise noted. No other warranty is made. Should engineering interpretation or opinion be required, NGE-TFT will provide upon written request.

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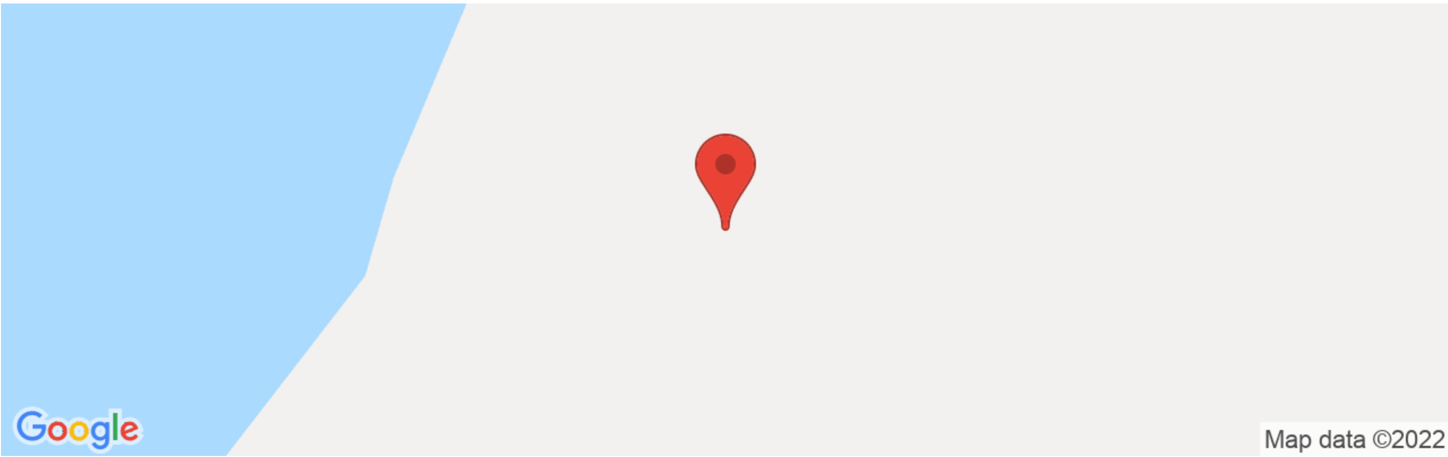
APPENDIX C

SEAOC SITE SPECIFIC SEISMIC LOADS

CEA Six-Mile ECT Substation Reactor Replacement

Latitude, Longitude: 61.311295, -149.812699



Date	1/18/2022, 2:42:53 PM
Design Code Reference Document	IBC-2015
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.678	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	1.017	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.678	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.6	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration
T_L	16	Long-period transition period in seconds
SsRT	1.912	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.708	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.837	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.806	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.678	Factored deterministic acceleration value. (1.0 second)
PGAd	0.6	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	1.12	Mapped value of the risk coefficient at short periods
C_{R1}	1.038	Mapped value of the risk coefficient at a period of 1 s

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